

12-2010

Investigation of Northeast Extreme Tee (NEXT) D Beam Bridges as an Alternative to Precast Hollow Core Bridges: An Exploration of Appropriate Slab Design Forces

Daniel Deery
Clemson University, ddeery5@gmail.com

Follow this and additional works at: https://tigerprints.clemson.edu/all_theses



Part of the [Civil Engineering Commons](#)

Recommended Citation

Deery, Daniel, "Investigation of Northeast Extreme Tee (NEXT) D Beam Bridges as an Alternative to Precast Hollow Core Bridges: An Exploration of Appropriate Slab Design Forces" (2010). *All Theses*. 1011.
https://tigerprints.clemson.edu/all_theses/1011

This Thesis is brought to you for free and open access by the Theses at TigerPrints. It has been accepted for inclusion in All Theses by an authorized administrator of TigerPrints. For more information, please contact kokeefe@clemson.edu.

INVESTIGATION OF NORTHEAST EXTREME TEE (NEXT) D BEAM BRIDGES
AS AN ALTERNATIVE TO PRECAST HOLLOW CORE BRIDGES:
AN EXPLORATION OF APPROPRIATE SLAB DESIGN FORCES

A Thesis
Presented to
the Graduate School of
Clemson University

In Partial Fulfillment
of the Requirements for the Degree
Master of Science
Civil Engineering

by
Daniel Patrick Deery Jr.
December 2010

Accepted by:
Dr. Bryant Nielson, Committee Chair
Dr. Scott Schiff
Dr. WeiChiang Pang

ABSTRACT

Adjacent precast, prestressed concrete beam bridges have become a popular solution throughout the country because deck forming can be eliminated and construction is rapid. In South Carolina, adjacent beam bridges primarily consist of flat slab or hollow core sections, and they are currently only used on secondary, low-volume, short-span bridges. Durability and load sharing issues stemming from cracking, however, have caused concern with the longevity of these bridge types. Thus, the South Carolina Department of Transportation (SCDOT) has sought an alternative to the flat slab for short span bridges that can be used on high volume roads without an overlay. This research focuses on the selection of the Northeast Extreme Tee (NEXT) D beam as an alternative and later focuses on the deck design for the bridge and appropriate slab design forces for the section.

The NEXT D sections designed for larger spans in the Northeast were scaled down since shorter spans were targeted in this project than in the original concept. Preliminary prestressed design was performed to verify the new section geometry. The deck was designed using the American Association of State Highway and Transportation Officials (AASHTO) Load Resistance and Factor Design (LRFD) Specifications assuming the deck functioned as a continuous beam with infinitely rigid supports. A sensitivity study was completed which involved varying the stiffness of the beam webs and the shear keys and studying the resulting shear and moment responses in the deck in order to determine appropriate slab forces for design.

The NEXT D section proposed in the Northeast was scaled down to six-foot (NEXT D6) and eight-foot (NEXT D8) wide alternatives, both 20 inches deep, and confirmed to meet AASHTO requirements for flexure and limit stresses for a 40-ft. span bridge. Through the sensitivity study, the AASHTO equivalent strip method was found to be conservative for shear but non-conservative for moment. The design positive moment values calculated using the AASHTO equivalent strip method for a 40-ft. span bridge were found to be on-average 2.51 times less than those determined through the sensitivity study which calculated the web stiffness using classical beam theory. Therefore, in order to be conservative, the stiffness of the beam webs should be determined using classical beam theory, instead of assuming infinite rigidity, when designing the NEXT D slab.

The average ratio of positive to negative moment generated in the shear key was found to be approximately 2:1 for the NEXT D6 and 6:1 for the NEXT D8. Therefore, the headed reinforcing bars should be placed one inch below the mid-depth of the shear key in order to optimize the moment capacity of the key by providing more eccentricity for positive moment. In addition, the translational and rotational stiffness of the shear key should be assumed to be fully rigid in order to produce conservative design forces in the key; however further numerical and experimental studies should be performed to determine more appropriate design forces for the shear key.

ACKNOWLEDGEMENTS

I would like to thank my entire graduate committee: Dr. Bryant Nielson, Dr. Scott Schiff, and Dr. WeiChiang Pang, for taking the time to guide and help me through this research project. Their constant availability and expertise in structures made this research possible and I could not have done it without them. I would especially like to thank my committee chair, Dr. Nielson, for finding a position for me in this project and providing the opportunity for me to study bridge design in-depth, which I was not able to do in my other courses.

I would like to thank the steering committee from the South Carolina Department of Transportation, specifically Steve Nanney, for their support of the project and their willingness to provide guidance and assistance to me during all phases of the project.

I would also like to thank my research teammates, Sara Roberts and Armando Flores, for their commitment to the project and their willingness to assist me throughout my time on the project. Although we were working on different assignments, our work frequently depended on one another and their hard work made my job much easier and more enjoyable.

Finally, I would like to thank my wife, Laura, for her constant support and encouragement not only during this research but also throughout my undergraduate and graduate careers. Her support and draft reviewing throughout the writing process was much appreciated and very helpful and I would like to thank her for all that she does.

TABLE OF CONTENTS

	Page
TITLE PAGE	i
ABSTRACT	ii
ACKNOWLEDGEMENTS	iv
LIST OF TABLES	vii
LIST OF FIGURES	viii
CHAPTER	
1. INTRODUCTION	1
2. BRIDGE BACKGROUND	4
Introduction	4
Current South Carolina Details	4
Accelerated Bridge Construction	7
Shear Key and Post-Tensioning	12
Continuity	18
Durability	23
Web Survey	27
DOT Interviews	30
Contractor and Fabricator Interviews	35
3. BRIDGE DETAIL SELECTION	38
Introduction	38
Steering Committee Meeting	38
Workshop	43
Detail Selection	50
4. NEXT D DECK DESIGN	51
Introduction	51
Design Issues	52
NEXT Developers Conference Call	54
Preliminary Beam Design	57
Slab Design	61

Table of Contents (Continued)

	Page
Conclusions and Closure.....	67
5. DECK SENSITIVITY STUDY	68
Introduction.....	68
Stiffness Parameters.....	68
Software	72
Results and Discussion	75
Conclusions and Closure.....	87
6. CONCLUSIONS.....	88
Recommendations for Future Work.....	90
REFERENCES	91
APPENDICES	96
A. Clemson University Web DOT Survey	97
B. DOT, Contractor and Fabricator Phone Interview Summaries.....	105
C. Fabricator Workshop Survey	117
D. Contractor and Fabricator Breakout Session Summaries.....	125
E. Slab Design Sheets.....	132
F. OpenSees Scripts	147
G. MATLAB Script	161
H. Sensitivity Study Plots	167

LIST OF TABLES

Table	Page
2.1 State Usage of Adjacent Beam Bridges	28
2.2 DOT Survey Responses	29
2.3 Bridge Alternatives	29
3.1 System Goal Attainment Chart	40
3.2 Pre-Workshop Fabricator Cost Estimates	45
3.3 Normalized Fabricator Cost Estimates	45
4.1 NEXT D Material Properties	60
4.2 Slab Transverse Reinforcement	65
4.3 Slab Longitudinal Reinforcement	65
5.1 Slab Forces due to Infinitely Rigid Supports	77
5.2 Influence of Span Length on NEXT D8 Responses	77
5.3 Ratio of NEXT D8 Response Value to Infinitely Rigid Responses	77
5.4 Ratio of NEXT D6 Response Values to Infinitely Rigid Responses	78
5.5 Ratio of NEXT D Response Values to Infinitely Rigid Responses	81
5.6 Revised Slab Transverse Reinforcement	83
5.7 Revised Slab Longitudinal Reinforcement	83

LIST OF FIGURES

Figure		Page
2.1	SCDOT Hollow Core Section.....	5
2.2	SCDOT Hollow Core Shear Key.....	5
2.3	SCDOT Hollow Core Transverse Ties.....	6
2.4	Precast Modified Beam in Slab System.....	8
2.5	Poutre Dalle System.....	9
2.6	Minnesota DOT System.....	9
2.7	Two Lane Modular Bridge.....	10
2.8	Grout Filled Splice Sleeve.....	11
2.9	NYSDOT plan shown with five TPT Locations.....	15
2.10	TXDOT plan shown with two TPT Locations.....	16
2.11	Texas Transverse Post-Tensioning Detail.....	17
2.12	Formation of Positive Restraint Moments.....	19
2.13	Grout Filled Splices used in Continuity Connections.....	21
2.14	Standard Continuity Details.....	22
2.15	Lakeview Drive Collapse.....	24
2.16	Lakeview Drive Box Beam Section.....	25
2.17	Texas Robust Shear Key Detail.....	31
2.18	Decked Slab Beam Section (Texas).....	32
2.19	Deck Bulb-T Bridge Girder (Washington).....	32
2.20	Inverset System Unit.....	33

List of Figures (Continued)

Figure	Page
2.21 NEXT F Beam	34
2.22 NEXT D Beam.....	34
3.1 Clemson Section Preliminary Details	39
3.2 Headed Continuity Option	48
3.3 Hooked Continuity Option.....	49
4.1 SCDOT Barrier Detail	53
4.2 New NEXT D Beam Geometry	58
4.3 NEXT D Shear Key Detail	59
4.4 CONSPAN Models.....	59
4.5 Preliminary NEXT D Prestressed Design.....	61
4.6 NEXT D8 Slab: Structural Idealization	62
4.7 Construction Load Case.....	64
4.8 NEXT D Beam Typical Reinforcement.....	66
5.1 Slab Structural Idealization.....	69
5.2 Live Load Placement to Determine Web Stiffness.....	70
5.3 Software Flowchart.....	74
5.4 Influence of Support Stiffness on Maximum Negative Moment for NEXT D6.....	75
5.5 Maximum Negative Moment Response Locations for NEXT D6.....	76
5.6 NEXT D8 – Influence of Support Stiffness on Slab Design Forces.....	79

List of Figures (Continued)

Figure		Page
5.7	Influence of Shear Key Stiffness on Design Forces in Shear Key	84
5.8	Revised NEXT D Beam Typical Reinforcement.....	87

CHAPTER ONE

INTRODUCTION

Adjacent precast, prestressed concrete beam bridges have become a popular solution throughout the country because in using them, deck forming can be eliminated and construction is rapid. The adjacent sections are also highly favored in construction since they provide an immediate working platform for the laborers as they place the remaining beams. In South Carolina, adjacent beam bridges primarily consist of flat slab or hollow core sections, and they are currently only used on secondary, low-volume, short-span bridges. These adjacent beam sections are mainly used on 20 to 70 ft. spans with an asphalt overlay. The sections are butted against each other with grouted cutouts (shear keys) and pulled together with transverse tie rods to facilitate transverse continuity and load sharing between the members.

These adjacent beam bridges have been used in many different variations all over the country; however some states have recently minimized or even ceased their use of the sections. In South Carolina, despite their low cost and ease of installation, the durability and potentially strength issues evidenced by longitudinal cracking in the asphalt along the shear key have caused concern. These cracks are problematic because they allow water and deicing salts to seep between the members and corrode the prestressing and post-tensioning steel. This is especially a concern in sections that include voids, since many states have noticed trapped water in the voids due to clogged drain holes, which can slowly corrode the steel in the sections from the inside-out. The bridges subjected to this type of deterioration mechanism usually do not show any significant visual distress prior

to catastrophic failure. This has been particularly problematic in the northern and coastal states which have more of a corrosive environment, but has also concerned the South Carolina Department of Transportation (SCDOT).

The longitudinal cracks also signify the possible break down of the shear key. In addition to causing durability issues with water and salts seeping into the sections, there is a reduced ability to distribute load to adjacent beams with a degraded shear key. The shear key is designed to transfer load from beam to beam, and although some load can still be transferred in a cracked key due to frictional resistance of the grout, the ability to share is compromised greatly with a cracked keyway. Since the hollow core beams are designed to take only a fraction of the wheel line load, a degraded shear key could cause overloading of one of the beams and thus failure. Also, with a degraded shear key, the amplitude of stress cycles increases and reduces the fatigue life of the bridges (Roberts 2010).

For these reasons, the SCDOT has sought a new alternative to be used on short span bridge projects. The following guidelines were defined by the SCDOT for the bridge alternative:

- Eliminate or minimize longitudinal and transverse cracking
- Lower, if possible, the initial price and maintenance costs
- Have a shorter erection time than cast-in-place (CIP) slabs
- Provide a longer service life than the current precast hollow-core slabs
- Be available for use on all routes (no Annual Daily Traffic (ADT) or National Highway System (NHS) restriction)
- Be designed so it does not require an asphalt overlay

The goal of this project is to select the alternative system through various forms of research and also to identify and address some of the design issues associated with the selected alternative. The objectives of this research include:

- Study the use of adjacent beam bridges nationwide and the behavior of the sections through surveys, interviews, and literature reviews.
- Select an alternative section for use on future bridge projects by meeting with local designers, fabricators, and contractor and discussing the findings.
- Adapt the section geometry to meet a targeted span length of 22 to 40 feet and perform preliminary design of the modified section.
- Identify design concerns with the section, specifically the deck slab.
- Perform sensitivity study on the influence of support and shear key stiffness on the deck design and provide recommendations for stiffness parameters in the design and compare these values to AASHTO recommendations.

The remaining chapters of this thesis focus on the various bridge sections considered, the future South Carolina alternative selection, and the design of the new section, specifically the slab and determining appropriate design forces for the slab. Chapter 2 discusses previous research pertaining to adjacent beam bridges and accelerated bridge construction and the designer, contractor, and fabricator interviews performed in order to learn more about the sections used nationally on short span bridges. Chapter 3 explains the process the research team and the SCDOT steering committee went through to narrow down potential sections and ultimately select a section for further consideration. Chapter 4 focuses on the design of the section, with a specific focus on the slab design. It also addresses the design concerns with the section that must be investigated before the section can be implemented confidently in South Carolina. Chapter 5 addresses one of the design concerns for the slab, the influence of support and shear key stiffness on the design forces for the slab. Finally, Chapter 6 states the conclusions and recommendations for future research regarding the NEXT D section.

CHAPTER TWO

BRIDGE BACKGROUND

Introduction

In order to ensure the alternative section would meet the objectives set by the SCDOT, a myriad of topics were studied in depth by reviewing literature from completed research, conducting surveys and by directly contacting designers, contractors, and fabricators nationwide. Accelerated bridge construction, shear key performance, continuity, durability, and transverse post-tensioning studies associated with adjacent beam bridges were reviewed in order to target specific details that could be considered for use in South Carolina. States that have used or currently use adjacent beam bridges or other intriguing sections with favorable results for short span, rapid construction bridges were identified through web searches and an online survey. These states were then targeted for further information through phone interviews. States of particular interest were asked for names of contractors and fabricators associated with these bridges and these groups were contacted for their perspective on the details.

Current South Carolina Details

The current adjacent beam bridge section used by the SCDOT is a three-foot wide precast, prestressed solid or hollow core slab 21 inches or 24 inches deep with 11 to 24 prestressing strands depending on the span length. A 21-inch deep hollow core section with 14 one-half inch diameter prestressing strands is the primary solution used for 40-ft. spans which is the targeted span length for this research (see Figure 2.1). The sections

are butted against each other with grouted cutouts (shear keys, see Figure 2.2) and pulled together with transverse tie rods to facilitate transverse continuity and load sharing between the members.

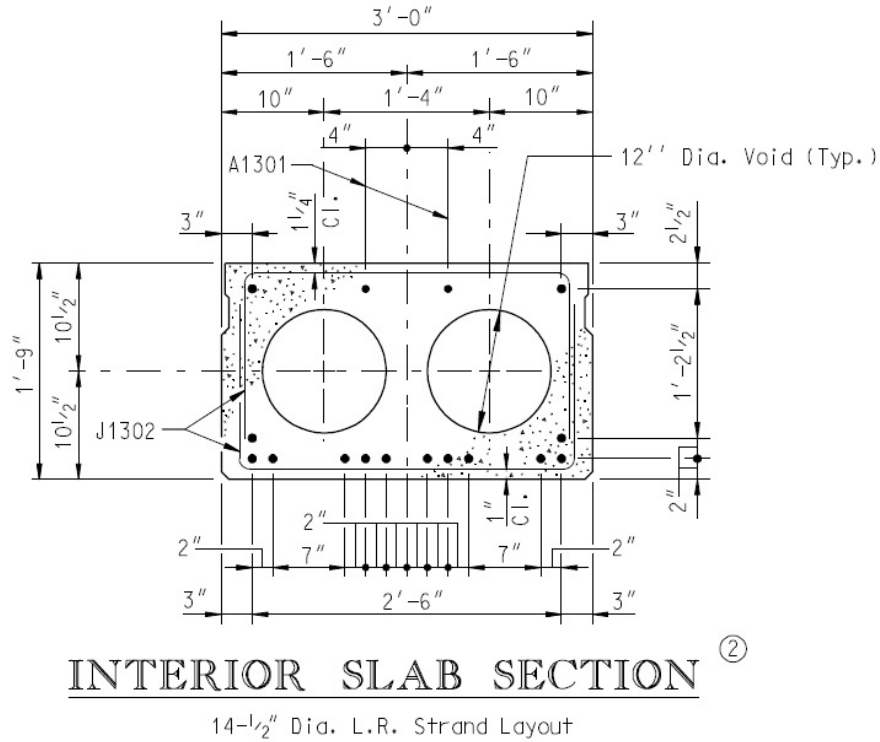


Figure 2.1: SCDOT Hollow Core Section (SCDOT 2010)

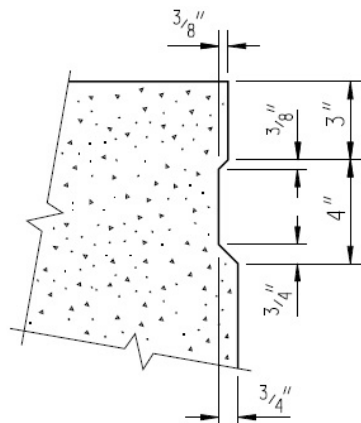
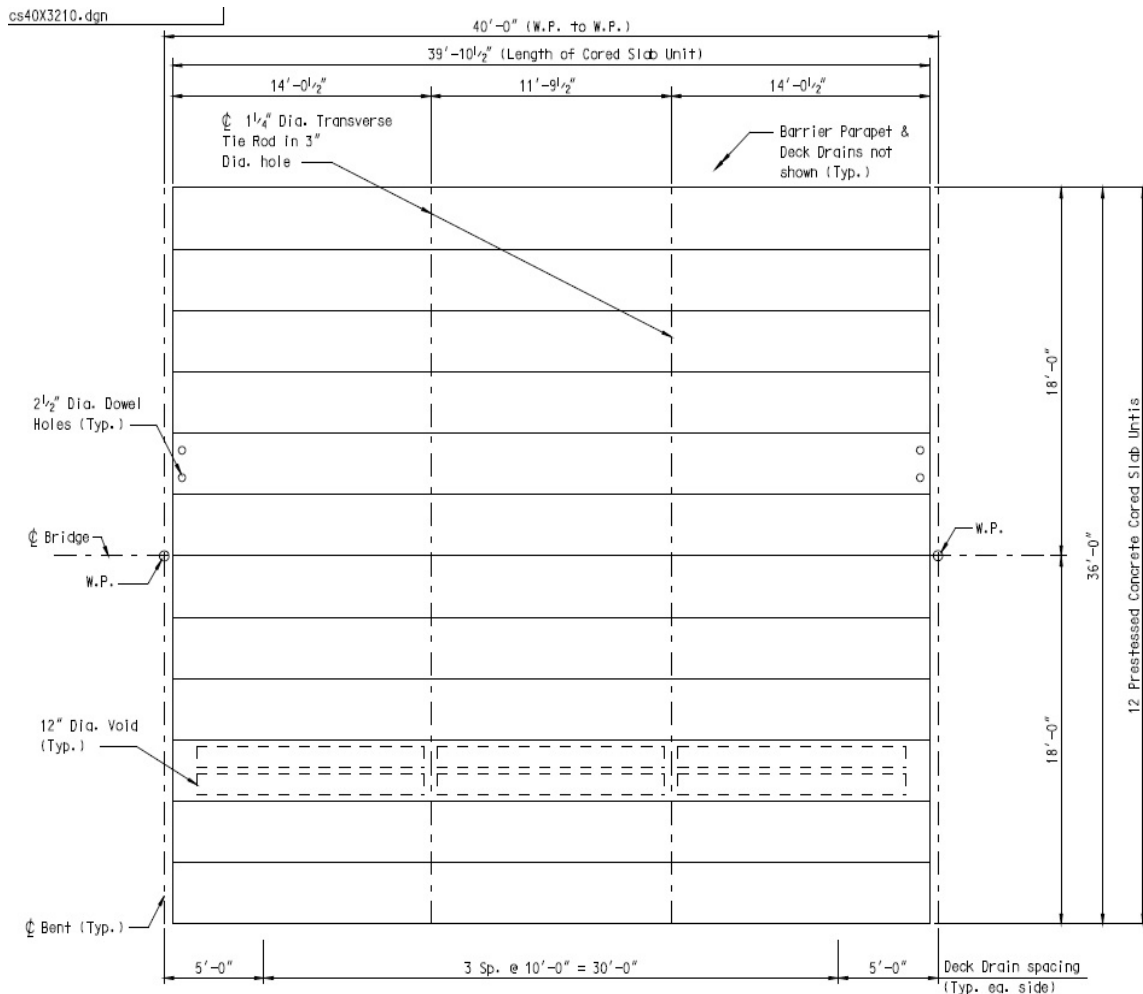


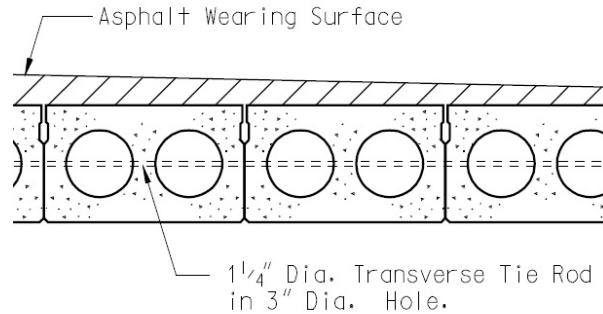
Figure 2.2: SCDOT Hollow Core Shear Key (SCDOT 2010)

The transverse ties are one-and-one-quarter inch mild steel rods located at the third points of the span and they are tensioned to ensure the adjacent beams touch (see Figure 2.3). The ties are not tensioned to a particular force level, which was found to be common in many states, since the AASHTO LRFD Specifications provide no design parameters for the lateral ties (Culmo 2009).



(a) SCDOT Hollow Core Plan showing Transverse Tie Rod Locations

Figure 2.3: SCDOT Hollow Core Transverse Ties (SCDOT 2010)



(b) SCDOT Hollow Core Section showing Transverse Tie Rods

Figure 2.3 (Continued)

The current details include an asphalt wearing surface (see Figure 2.3b), thus there is no reinforced concrete overlay to help with load sharing; the shear key and ties must fully distribute the wheel-line loads to adjacent beams. These bridges have frequently formed longitudinal reflective cracks in the asphalt overlay along the joints between adjacent sections (above the shear keys). Transverse cracks have also developed at the abutments and interior bents where no continuity is provided between adjacent spans. The SCDOT desires to resolve these issues without compromising the construction time in the new alternative. The following literature reviews and designer, contractor, and fabricator interviews were performed to target alternatives and design and construction practices that could be the solution to this problem.

Accelerated Bridge Construction

A search of publications relating to accelerated bridge construction techniques and designs was conducted to learn of the recent research in the area. Several publications were found relating to new systems, new materials, and new construction practices to reduce the erection time of bridges.

One of the systems found was the beam-in-slab system. The most recent system consists of rolled wide flange steel sections precast into concrete with transverse arching (see Figure 2.4).

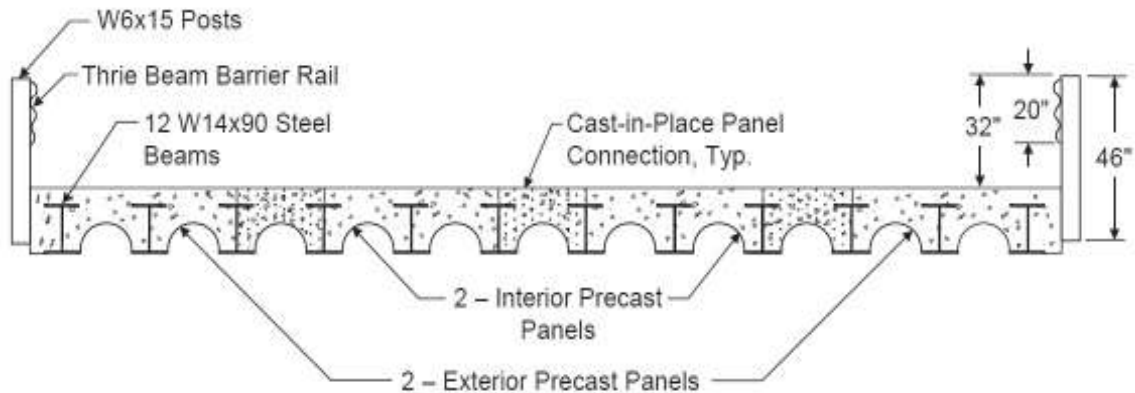


Figure 2.4: Precast Modified Beam in Slab System (Konda et al. 2007)

This system is an adaptation from the Beam in Slab System and Modified Beam in Slab System and was designed to target bridge spans of 40 to 80 ft. and designed to serve low volume roads in Iowa (Konda et al. 2007). The Precast Modified Beam in Slab Bridge (PMBISB) would ensure rapid construction and eliminate longitudinal reflective cracks, however more research would be required to ensure it could be used on high volume roads. Another drawback is the large, cast-in-place concrete pours between precast sections which would inherently prolong construction.

Another system found was the Poutre Dalle system which is used in France and consists of a shallow, precast inverted tee beam which serves as stay-in-place formwork for the bridge (see Figure 2.5).



Figure 2.5: Poutre Dalle System (Ralls et al. 2005)

The members are placed adjacently and a cast-in-place deck is then poured over them. This system is typically used for span lengths of 20 to 82 ft. The Poutre Dalle system retains the clearance advantage of the South Carolina hollow core slabs and decreases the occurrence of cracking because of the elimination of a cold-joint through the depth of the slab. The Minnesota Department of Transportation (MnDOT) has created their own modified version of the Poutre Dalle system and has implemented it successfully on several bridges (see Figure 2.6) (Culmo 2009).



Figure 2.6: Minnesota DOT System (Culmo 2009)

Another short span bridge option that has become popular in the Midwest is the modular steel bridge system. These bridges are made in strips of steel grid that can be placed and fastened together using bolted diaphragms (see Figure 2.7). These bridges are mainly used for very low volume roads or temporary purposes, but can be fitted with precast deck panels to make them permanent vehicular bridges. Big R Bridge (Big R Bridge) and Roscoe Bridge (Roscoe Bridge) manufacture these types of modular structures. If this system could be tested and authorized for high ADT routes, this may be a high speed alternative for short span bridges with the use of precast deck panels.

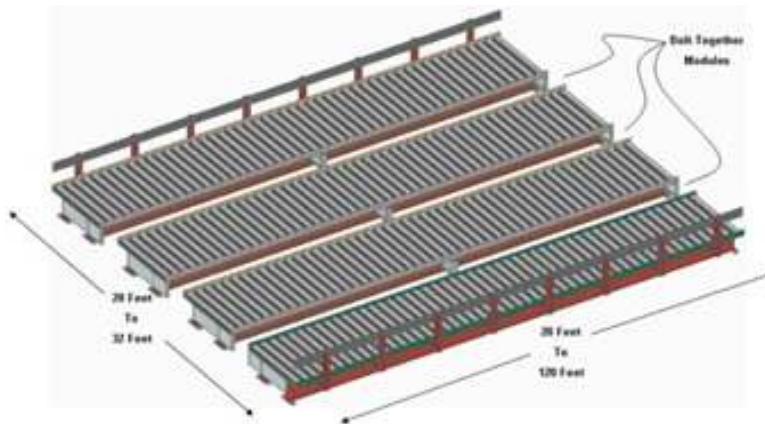


Figure 2.7: Two Lane Modular Bridge (Roscoe Bridge)

One new construction practice found involved using grout filled splice sleeves to make the transverse connections between adjacent beams in lieu of cast-in-place concrete details. The Michigan Department of Transportation has conducted a research study on the use of grout filled splice sleeves used in precast construction (Jansson 2008). These sleeves are proprietary products manufactured by Lenton and NBM. The sleeves come in different sizes for specific rebar (see Figure 2.8). These sleeves can be placed in members during precasting and rebar jutting out from an adjoining member can be fitted

into the sleeve during erection. These sleeves are listed in ACI 550.1R-09 as acceptable means of emulating cast-in-place concrete details (ACI 2009).

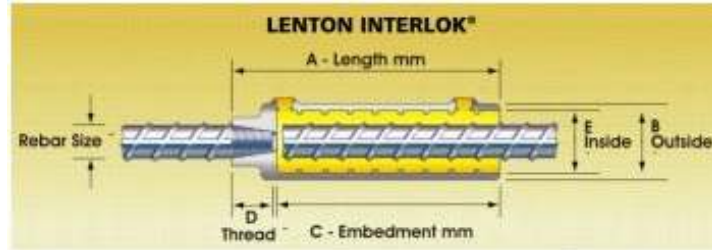


Figure 2.8: Grout Filled Splice Sleeve (ERICO)

These sleeves are mainly used in vertical connections such as connecting piers to pier caps and beams because of the difficulty of grouting normal connections in these situations. There is not much information on using these to make horizontal connections such as adjoining adjacent beams. The tolerances would need to be well controlled well to ensure match up when erecting the beams. However, if these systems could be implemented successfully in adjacent construction, the load sharing ability could be enhanced between members.

Another construction practice was discovered through articles promoting the use of self-propelled modular transports (SPMTs) and barges as an accelerated construction technique (Bergeron 2008). Large, full-width sections of bridges are built nearby and rolled or floated into place using the SPMTs. This technique is excellent in reducing road closure time and reducing construction time and costs. However, given the terrain in South Carolina and the short spans this research deals with, these solutions do not appear to be a feasible way to achieve this project's objectives.

Finally, an article from *Construction and Building Materials* describes a practical case of using rapid hardening concrete on a short span bridge (Cangiano et al. 2009). The experiment was conducted in Italy using precast elements as stay-in-place forms for the early age strength concrete. The concrete was self-compacting and achieved a compressive strength of about 80 MPa (11.6 ksi) within 24 hours. Although one of the targets of this research is to eliminate topping when constructing these short spans, cast-in-place concrete may be required in a desirable design. The use of this rapid hardening concrete could be used to shorten the construction time greatly and should be taken into consideration.

Shear Key and Post-Tensioning

In addition to accelerated bridge construction research, specific attention was given to shear key and post-tensioning research in order to study the practices used nationwide to improve the load sharing between members and minimize longitudinal reflective cracks. The AASHTO LRFD Specifications provide no design parameters for the common method of shear transfer of using the combination of grouted shear keys and lateral ties (Culmo 2009). Also, AASHTO's transverse post-tensioning recommendation of 250 psi applied over a keyway depth of 7 inches is rarely met by states since it is very conservative and difficult to reach (Russell 2009). Therefore, the size and type of shear key and amount of post-tensioning has evolved by trial and error and varies widely between State departments of transportation (DOTs) (Culmo 2009).

The most common shear key placement is at the top flange of the beam and they are usually very small keyways similar to the SCDOT detail (see Figure 2.2). Due to the

narrowness of the keyway it can be very difficult to place the grout correctly. The West Virginia DOT investigated several high volume bridges and concluded the shear key failures were due to inadequate grouting procedures during construction (El-Remaily et al. 1996). The moment transferred between the beams creates a hinging action about the shear key and can possibly lead to opening and closing of the grout at the top face of the beams (Miller et al. 1999). Furthermore, the application of post-tensioning force at mid-depth after the curing of a partial-depth shear key may create moment and opening of the grout at the top of the beams (Russell 2009). To improve the performance and durability of the shear keys, tensile moment action at the shear key face needs to be reduced.

The most common mitigation strategy for cracks along the shear key, which many state DOTs implement, is to require five to six inches of reinforced concrete overlay on the beams. However, approximately 65 percent of states that responded to Henry Russell's 2009 survey on adjacent beam bridges still see reflective cracking through the concrete overlay (Russell 2009). The use of transverse post-tensioning has also been considered a viable solution to minimize the development of the longitudinal cracks in adjacent box beam bridges (Grace and Jensen 2008). Therefore, in addition to the overlay requirement, some state DOTs also increase the amount of transverse post-tensioning force. Transverse ties, grouted or ungrouted, vary from a limited number of nontensioned, threaded rods to several high-strength strands post-tensioned in multiple stages (Russell 2009). The amount of transverse post-tensioning can be varied by the number of transverse diaphragms, the number of strands at each diaphragm, and the amount of post-tensioning force applied at each strand. States with transverse post-

tensioning requirements use one or all of these methods to adjust their transverse post-tensioning levels.

At the time of Russell's survey, only 19 percent of states that responded made design calculations to determine the amount of post-tensioning necessary (Russell 2009). Two of these states were New York and Texas, both of which use full depth shear keys. Both of these states' standards and bridge design manuals include their TPT recommendations, and these states are satisfied with their current details (Losee and Deery 2010).

Section 9.2.6 of the NYSDOT Bridge Manual states the size, number, force, and location required for the strands based on the span length. Each tendon is composed of three ½"-diameter low-relaxation strands tensioned to 28 kips per strand and these are tensioned after the shear keys have been grouted. For span lengths less than 50 ft., three TPT force locations are required and all other span lengths require five TPT force locations (see Figure 2.9) (NYSDOT 2010a).

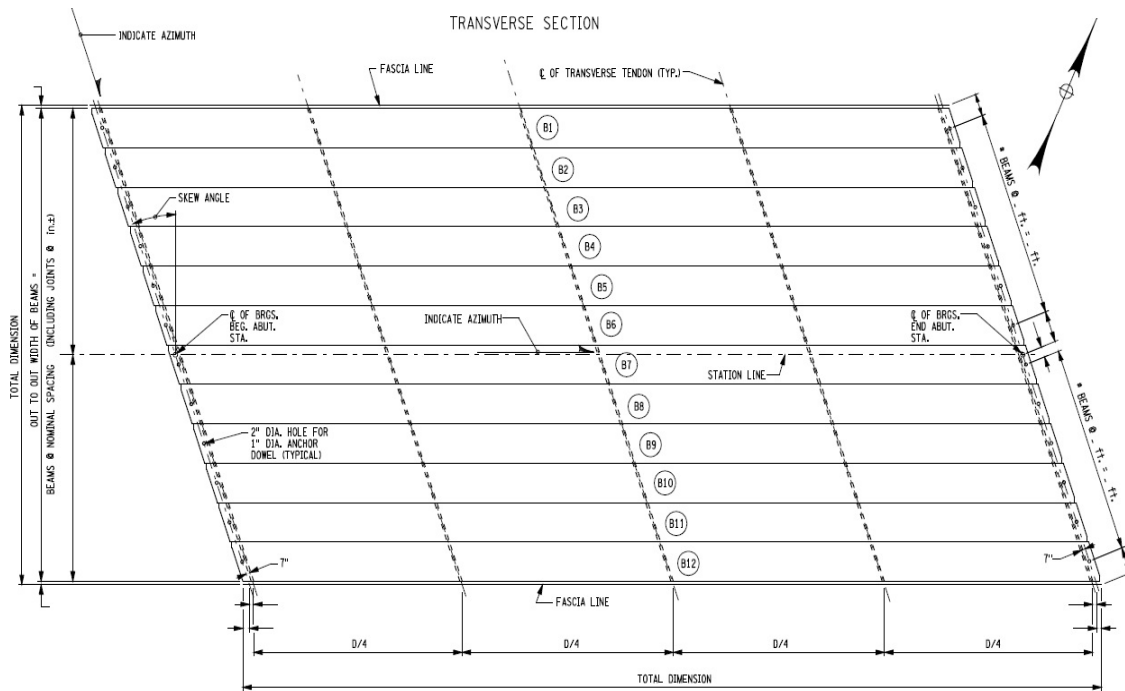


Figure 2.9: NYSDOT plan shown with five TPT Locations (NYSDOT 2010b)

These requirements have been consistent since 1992, when New York revised their transverse post-tensioning recommendations. At the time, they were experiencing frequent longitudinal cracking issues and increased the post-tensioning requirements and enlarged the shear key, which led to the details they still use today (Russell 2009).

Chapter 3, Section 9 of the Texas DOT Bridge Design Manual requires transverse post-tensioning for all box beam bridges topped with an ACP overlay applied directly to the top of beams (TXDOT 2009). The majority of the time, however, Texas uses a five-inch concrete overlay without post-tensioning. The tendons are limited to a maximum spacing of ten feet and although Texas does not have a target post-tensioning force, over the years the force has been increasing and is currently around 30 kips per strand, which is higher than ever before (TXDOT 2009). The Texas DOT plan is shown with two

transverse post-tensioning locations in Figure 2.10. The tendons for a prestressed concrete box beam with ACP overlay are located at about the mid-depth of the shear keys, and are shown in Figure 2.11.

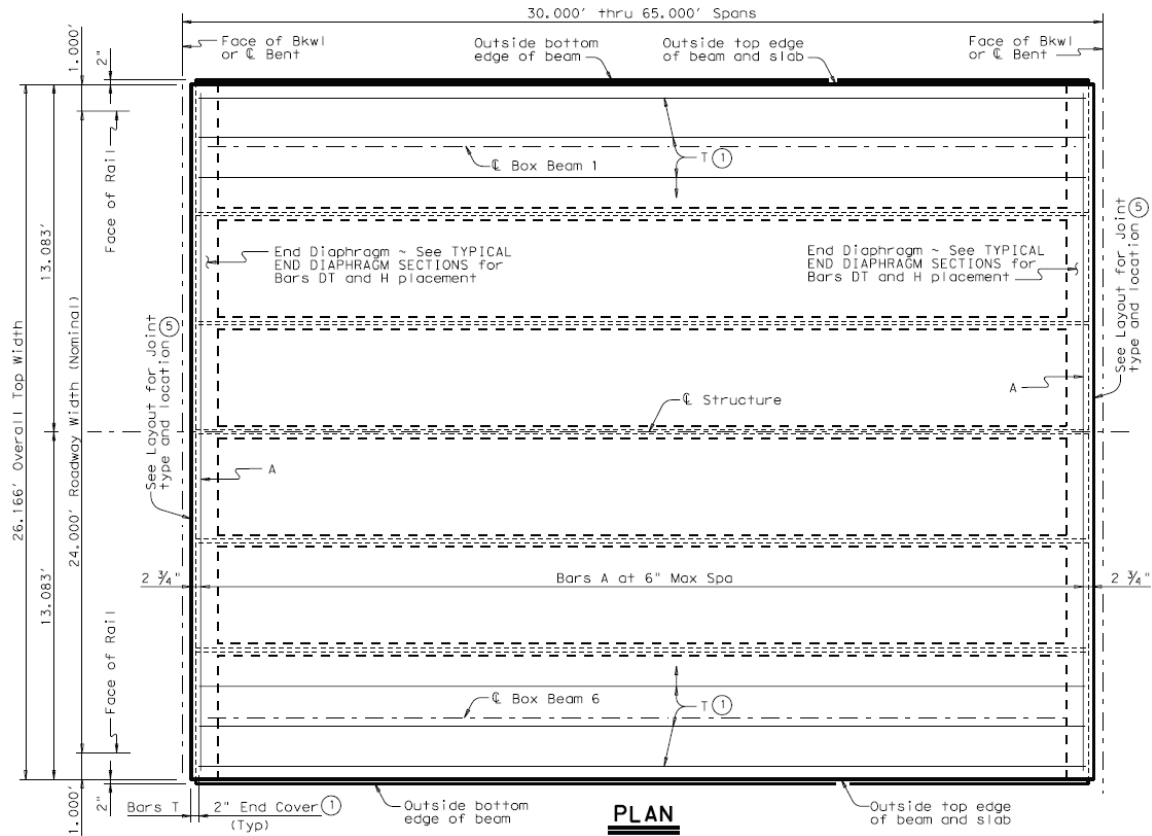


Figure 2.10: TXDOT plan shown with two TPT Locations (TxDOT 2010)

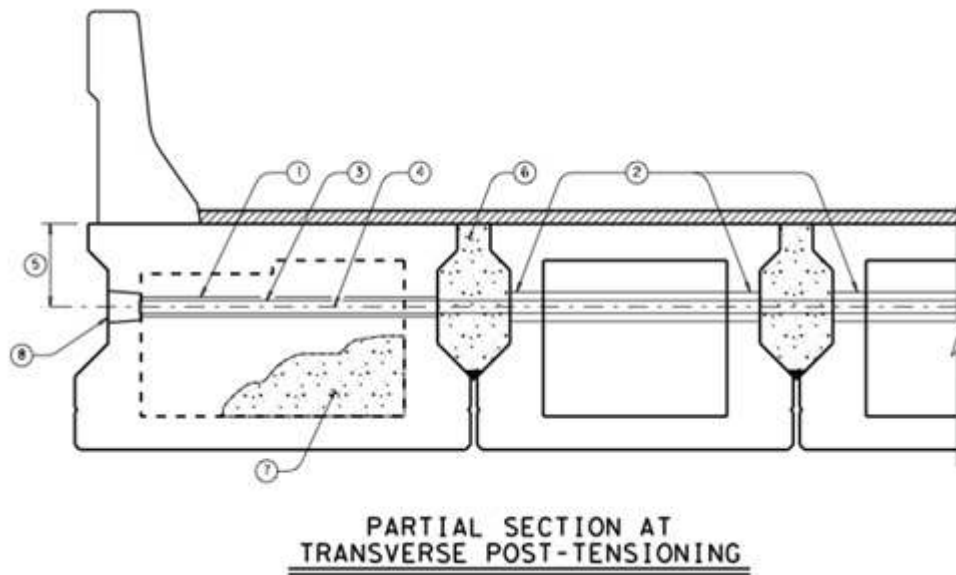


Figure 2.11: Texas Transverse Post-Tensioning Detail (TxDOT 2010)

Various experimental and numerical studies have been performed worldwide regarding shear key and transverse post-tensioning practices. The published recommendations for dealing with reflective cracking in longitudinal shear keys were documented by Roberts (Roberts 2010) and listed below. The following recommendations have shown to decrease the amount of cracking, but not eliminate the cracking completely:

- Move the shear key to the neutral axis of the member (Miller et al. 1999).
- Use a full depth shear key that can be grouted easily (Miller et al. 1999).
- Provide post-tensioning in the top and bottom of the beam (Lall et al. 1998).
- Transversely post-tension after grouting the keys if it will not cause moment about the shear key (Russell 2009).
- Use a grout material with high bond strength (Miller et al. 1999).
- Provide a target post-tensioning force developed for the individual bridge's span, width and member depth (Grace and Jensen 2008)

Continuity

In addition to longitudinal reflective cracks, the current SCDOT details have experienced transverse cracking at the abutments and intermediate bents. Therefore, a longitudinal continuity detail is desired for the alternative section to help minimize these cracks. In addition to minimizing transverse cracking, prestressed concrete bridges are commonly made continuous in order to improve the structural efficiency by reducing the maximum positive moment in the spans of the bridge. Continuous-span bridges are also advantageous since they reduce deflections and result in a better riding surface. Finally, this type of design is also beneficial from a maintenance point of view compared to a simply supported design since it eliminates open joints at intermediate supports. Eliminating open joints and transverse cracks in a bridge helps to minimize water drainage onto the substructure that can cause rebar corrosion and concrete delaminating.

Existing continuity connections have their own shortcomings, however, including the development of positive restraint moments which cause diaphragm cracking at interior piers. Other shortcomings include potentially prolonged construction and time consuming and expensive joint construction due to reinforcement congestion (Saadeghvaziri et al. 2006). These shortcomings must be addressed in order to justify the use of continuous spans.

Bridges composed of simple-span precast prestressed concrete girders made continuous through cast-in-place decks and diaphragms develop positive restraint moments over the internal supports due to time dependent properties such as creep and shrinkage. The girders tend to camber upward due to creep of the concrete and continuity

keeps the girder ends from rotating, which results in positive restraint moments in the girders over the interior bents (see Figure 2.12). Cracks usually develop at the bottom of the diaphragms due to the positive moment, which can cause corrosion of the reinforcement in the diaphragms as well as simply damaging the bridge aesthetics (Saadeghvaziri et al. 2006). These effects are accounted for in the continuity diaphragm with either mild steel reinforcement or prestressed strands that typically continue from the bottom flanges of the precast girders (AASHTO 2007).

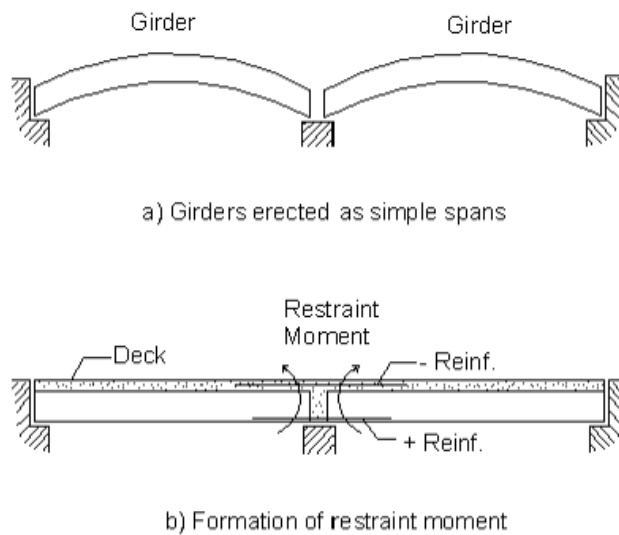


Figure 2.12: Formation of Positive Restraint Moments (Saadeghvaziri et al. 2006)

Studies and field experience indicate that waiting to establish continuity until girders are at least 90 days old will significantly reduce or eliminate the development of positive restraint moments at the internal piers (AASHTO 2007). The girders perform much better at this age; therefore the restraint moments can be ignored in design. Depending on the ability of the fabricator to stockpile, this age can usually be accounted for off-site in the precast yards, thus not impacting the construction time. However, if

this time was accounted for on the job site it would significantly prolong construction. Also, the cast-in-place continuity diaphragm, even with quick setting and consolidating concrete, cannot keep up with the construction pace of the rest of the prestressed, precast bridge components.

One new strategy to improve on current continuity practices involves using Carbon Fiber Reinforced Polymer (CFRP) sheets, instead of mild steel or prestressed strands, to create the continuity. CFRP reinforcement is attached to the top of the girders over the cast in place diaphragm. The negative moment over the supports caused by the deck weight balances the positive restraint moment caused by creep in the prestressed girders. The proposed design eliminates positive moment cracking while increasing structural efficiency. Furthermore, there is no need for positive moment reinforcement in the diaphragm under gravity loads, thus, reducing reinforcement congestion and facilitating construction. Laboratory tests and finite element analyses performed support the notion that CFRP sheets are an ideal material for continuity connections (Saadeghvaziri et al. 2006). However, CFRP sheets are typically more expensive in dollars per square foot than conventional mild reinforcing bars and their use has been minimal in continuity diaphragms thus far.

Mechanical splice sleeves, which have been used to provide continuity over joints (Jansson 2008), are another potential solution. Grout filled splices can be used to create live load continuity as negative moment reinforcement over a pier (see Figure 2.13).

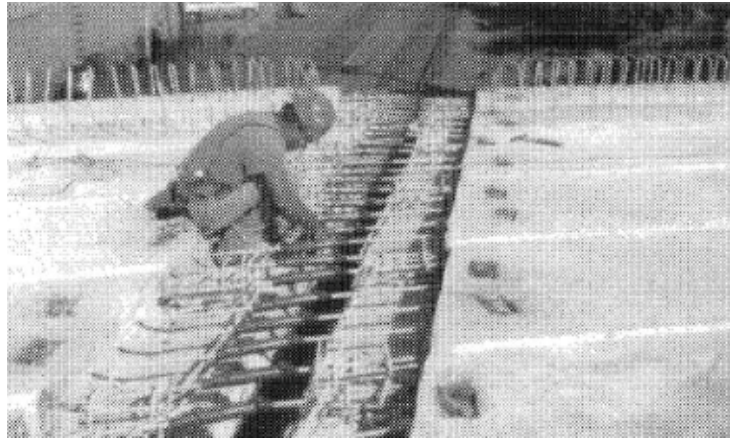


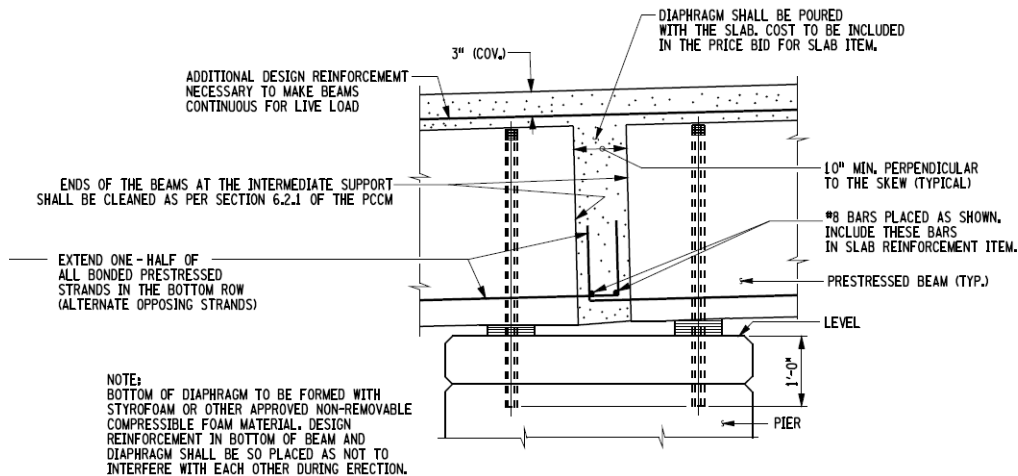
Figure 2.13: Grout Filled Splices used in Continuity Connections (Jansson 2008)

This detail was used for the North Street Bridge in Medford, Massachusetts. The negative moment reinforcement was cast into the top of the precast prestressed deck-beams and connected with splices over each pier joint (Jansson 2008).

Two proprietary grout-filled mechanical reinforcement splices, the Lenton Interlok and the NMB Splice Sleeve, have been evaluated for suitability in connecting precast concrete structural elements. The testing conducted by Jansson included slip, fatigue, ultimate load and creep tests. Both splices met the AASHTO LRFD provisions for slip and fatigue and neither showed susceptibility to significant creep displacements. The ultimate load of the two splices demonstrated that they are capable of exceeding 125 percent of the reinforcing bar's yield strength and in most cases 150 percent. The limited data did suggest epoxy coating might lower the ultimate load capacity after sustained loading (Jansson 2008). Despite this, these grout filled splices may have potential as options for longitudinal continuity in the adjacent beam bridges.

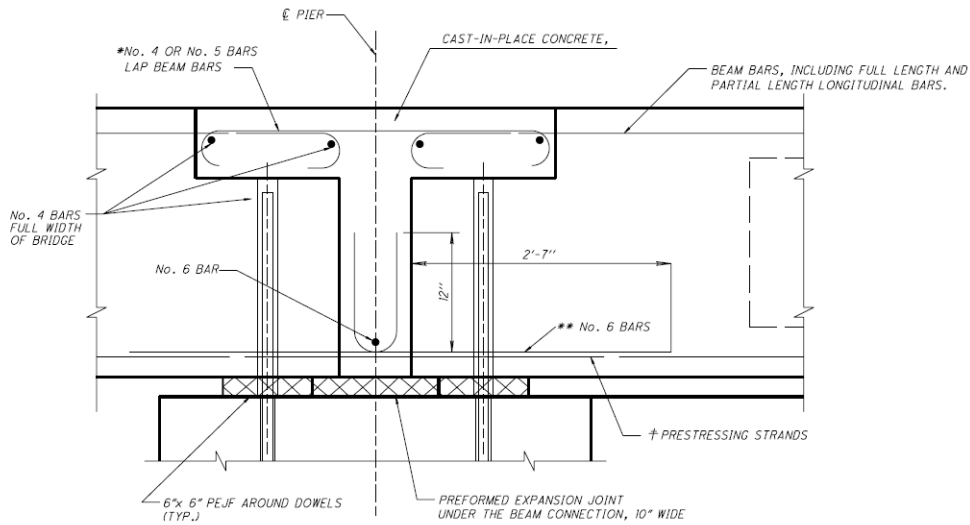
New York and Ohio DOTs both provide details for continuity diaphragms between spans of adjacent box beams (see Figures 2.14a and 2.14b). Texas provides a

detail which only includes the notch in the precast members to make way for a small pour that connects the top reinforcement in the two beams (see Figure 2.14c). This detail may not provide moment restraint, but it may help reduce the cracking at the bents. Texas is one of a few states that construct bridges with continuity diaphragms, but for design calculation purposes the spans are considered as simply supported, thus not taking advantage of the reduced positive moment but still benefiting from the lack of open joints (Saadeghvaziri et al. 2006). The details from New York and Ohio are full continuity diaphragms, where Texas' detail is not a full diaphragm; however all of these states are pleased with the performance. Therefore, a variation of one of these details could be implemented on the alternative section in South Carolina.

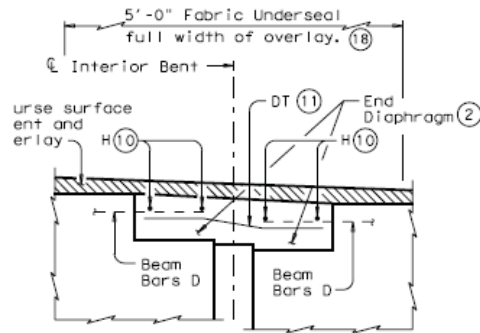


(a) New York Continuity Detail (NYSDOT 2010b)

Figure 2.14: Standard Continuity Details



(b) Ohio Continuity Detail (Ohio DOT)



(c) Texas Continuity Detail (TxDOT 2010)

Figure 2.14 (Continued)

Durability

One problem common to these adjacent beam bridges that experience cracking is strand corrosion. Water and deicing salts leaking through open joints can infiltrate the beams and corrode the prestressing steel in the sections. The decreased area of steel leads to loss of strength and without maintenance can eventually lead to failure and collapse of the bridge. Voided sections are particularly problematic, since they can accumulate water

and salts in the voids which corrode the interior strands that cannot be detected through visual bridge inspections. As a result, some states have abandoned the use of voided adjacent beam bridges.

In 2005, undetected strand deterioration led to the collapse of SR 1014 over I-70 (Lakeview Drive Bridge) in Pennsylvania (see Figure 2.15). The bridge was a non-composite prestressed concrete adjacent box beam bridge with a bituminous wearing surface without waterproofing. The fascia beam was overloaded after the grout in the shear key failed and prevented load sharing between the adjacent members.



Figure 2.15: Lakeview Drive Collapse (Scott 2006)

The box beams were 48 inches x 42 inches with 60 prestressing strands each and were connected with 1 inch diameter steel tie rods and non-shrink grout in the shear keys (see Figure 2.16).

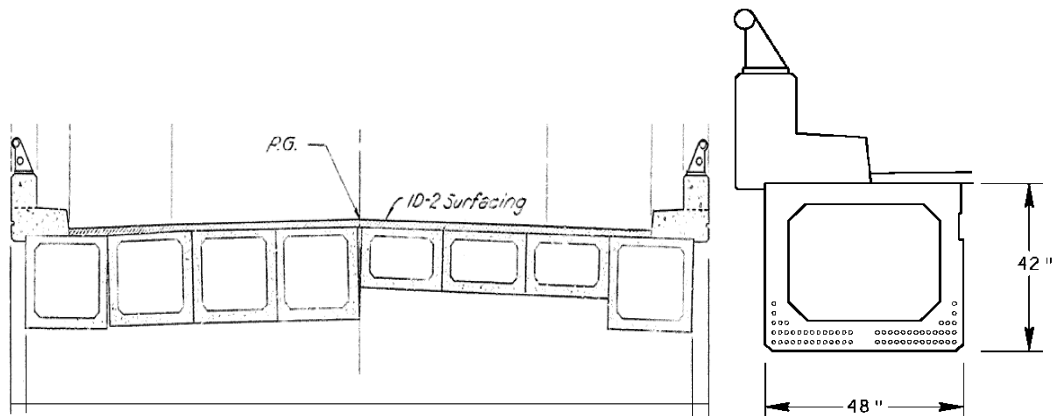


Figure 2.16: Lakeview Drive Box Beam Section (Scott 2006)

Corrosion of the hidden strands was determined to be the largest factor contributing to the failure of the bridge (Hartle 2009). Other factors contributing to the failure include: insufficient strand size, minimal cover, truck collision damage, and fabrication quality control (Scott 2006). The bridge was not considered structurally deficient before collapse largely because only 20 of the 39 failed strands could be visually inspected. Strand losses extended past what inspectors could visually assess which was contrary to conventional wisdom that reinforcement encased in concrete does not corrode. Lab assessment of the girder showed that 39 of the 60 strands had failed which is 95 percent more than that determined in the original field inspection one year before the collapse (Scott 2006). The corrosion was due to an open joint at the barrier that allowed deck leakage through the joint and allowed water and years of salt spray to infiltrate into the sections. These box beams used cardboard forms which have been found to soak up water and eventually slip and clog up drain holes in the box beams, thus allowing water and salts to reside in the voids and corrode the steel. Although cardboard

forms are less common today, drain holes can still clog and allow salt water to corrode the steel.

As a result of this event, Pennsylvania surveyed surrounding states and found at least six states with similar problems with these bridge types (Naito 2010). PennDOT revised their rating guidelines to account for hidden strands and then re-inspected all adjacent box bridges in the state within eight months of the Lakeview Drive failure. The number of structurally deficient bridges more than doubled as a result of the revised guidelines. Pennsylvania issued a moratorium on non-composite adjacent box beam bridges as a result of the collapse since no proven methods to determine deterioration of hidden strands were known and no cost effective rehabilitation and repair schemes had been identified for these initial low cost bridges as they near the end of their useful life (Scott 2006).

Research by the University of Toledo and Lehigh University has commenced to evaluate various nondestructive tests could be used in the field to inspect these bridges for hidden strand corrosion. Dr. Douglas K. Nims of the University of Toledo has been working to develop a prototype magnetic sensor that can reliably estimate the remaining cross sectional area of exposed or hidden corroded prestressing strands (Nims 2010). Dr. Clay J. Naito of Lehigh University has been working with PennDOT since December 2007 to determine inspection methods and techniques for non-visible corrosion of prestressing strands. Currently, destructive evaluation tests have been completed, all nondestructive evaluation vendor reports have been studied, and an extension was granted so that three NDT technologies can be studied more in-depth (Naito 2010).

These projects should provide progress towards accurate inspections to determine corrosion in non-visible strands in precast bridges.

No other similar adjacent box beam failures have been identified through literature searching. A nondestructive test that can determine the corrosion in hidden strands will surely improve the quality of inspections and maintenance on these bridges and extend their useful life and improve the safety of the systems. Higher strength, less permeable concretes and improved casting procedures have improved the structural integrity of box beam bridges over the past 50 years and the durability will only improve as these bridge types are used in future designs (Nims 2010). However, until accurate inspection procedures for non-visible strands are identified, maintenance and inspection procedures of the alternative section must be accounted for in the design to ensure safety and a long service life for the bridge.

Web Survey

In order to learn more about the practices and performance of bridges similar to the hollow core system and to study current alternatives to the hollow core system, the Departments of Transportation (DOTs) nationwide were surveyed. The web survey was created using SurveyMonkey and sent out through the SCDOT to DOTs nationwide. The results of the web survey can be found in Appendix A. Twenty-two different DOTs submitted complete responses to the survey, which helped to form a more targeted follow-up to selected DOTs. This also helped to identify practices used by other states that had not yet been noted.

The survey was split into two sections: low-profile adjacent beam bridges and high-profile adjacent beam bridges, and the states were questioned on a variety of subjects associated with these bridges. In Table 2.1 the states are organized based on the type of bridge they use.

Table 2.1: State Usage of Adjacent Beam Bridges

Types Used	States
Low Profile	AL, ME, MS, WA
High Profile	IN
Both	CA, IL, MA, MO, NM, OH, OR, TX, UT, VA
Neither	FL, KS, MT, ND, OK, PA, TN

Many of the responses gathered indicated that less than 20% of the states' bridges experience longitudinal reflective cracking. Most states did indicate that they were very concerned with these cracks, leading to the assumption that they are aware of them and have been putting forth some effort to try and prevent them. Since the shear key and amount of post-tensioning largely influence longitudinal cracking, the shear key and post-tensioning responses are included in Table 2.2 along with the percentage of bridges that experience cracking.

Table 2.2: DOT Survey Responses

State	% Bridges With Long. Cracking	Shear Key			Post Tensioning	
		Depth	Location	Grout	Before/After Key Grout	Type
Alabama	0-20	Partial	Top Face	CIP	Before	Rods
California	0-20	Partial	Centroid	Non-shrink	After	Both
Illinois	20-40	Partial	Top Face	Non-shrink	Before	Rods
Indiana	-	-	-	-	-	-
Maine	-	Full	Full	Non-shrink	-	Strands
Massachusetts	40-60	Full	Full	Epoxy	After	Strands
Mississippi	0-20	Partial	Top Face	Non-shrink	After	Rods
Missouri	0-20	Partial	Top Face	Non-shrink	After	Rods
New Mexico	0-20	Partial	Top Face	Non-shrink	Before	Rods
Ohio	80-100	Partial	Top Face	Non-shrink	After	Rods
Oregon	0-20	Partial	Top Face	Non-shrink	-	-
Texas	-	Partial	Centroid	CIP	After	Both
Utah	40-60	Partial	Top Face	Non-shrink	Before	Both
Virginia	20-40	Both	Top Face	Non-shrink	Before	Both
Washington	0-20	Partial	Top Face	CIP	Before	Strands

The only states that indicated in their survey responses that they make these adjacent beam bridges longitudinally continuous for multi-spans were California, Massachusetts, Missouri, Ohio, and Washington. All of these states indicated they account for positive restraint moments when designing the continuity diaphragm in order to minimize cracking and potential bridge durability and aesthetic issues but their details vary. A few states listed alternatives to adjacent beam bridges at the end of the survey and these responses are summarized in Table 2.3.

Table 2.3: Bridge Alternatives

Alabama	Precast Concrete Deck Channels
California	Spliced Precast Girder Systems
Tennessee	CONSPAN Arch, Single T Girder
Washington	Deck Bulb T, Precast Concrete Deck Form Panels

Many states consider longitudinal cracking along the joints of their adjacent beam bridges an issue. Due to the many differences in practices and crack prevalence, it is difficult to determine which methods best remedy the longitudinal cracking problem. Also, there is not a consensus on how to create continuity between the box beams. This shows that the use of longitudinal continuity depends on the individual DOT's common practice and not a universally accepted superior detail.

DOT Interviews

To obtain more detail on the practices and performance of bridges similar to the hollow core system, learn more about alternative systems, and to follow-up on the survey responses, phone calls were made to target DOTs across the country that either filled out the survey or were a state of interest based on their online bridge standards. Twelve states were targeted and unique questions were posed for each state that was contacted based on the website searching and web survey responses. The complete interview summaries can be found in Appendix B.

Practices that were confirmed to improve bridge performance based on these conversations included: full-depth shear keys, larger shear keys (see Texas detail, Figure 2.17), reinforced concrete topping, post-tensioning after grouting the shear key, and more post-tensioning/higher post-tensioning stress.

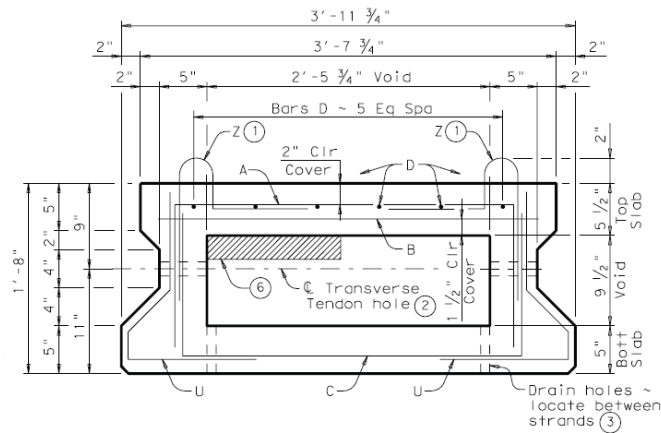


Figure 2.17: Texas Robust Shear Key Detail (TxDOT 2010)

Factors confirmed to degrade bridge performance based on these conversations included: reinforcing steel corrosion, partial depth shear keys, little/no post-tensioning, and post-tensioning before grouting the shear key. In addition, the NYSDOT mentioned corrosion of steel in these bridges was one of the main reasons New York recently had limited the usage of box beams.

Alternatives that were discovered to be of interest include:

- Decked Slab Beam Bridges (Texas)
- Deck Bulb-T Bridges (Washington)
- NEXT Beam (Precast/Prestressed Concrete Institute Northeast)
- Inverset System (New York).

Decked Slab Beams for the same depth usually span farther, use fewer beam lines to haul out to a jobsite, and install quicker than standard adjacent beams since there is no field placed concrete. They have only been used since 2007 but thus far have shown no signs of longitudinal reflective cracking although they have been used primarily on low-volume roads so far. The shear key detail is different from common details seen across the country since they use welded connector plates in a “V” like detail (see Figure 2.18b).

One concern with this detail is fatigue, since welds are depended upon so heavily in the shear key. However, the detail has not been used long enough to confirm the long-term performance of this shear key (see Decked Slab Beam detail, Figure 2.18).

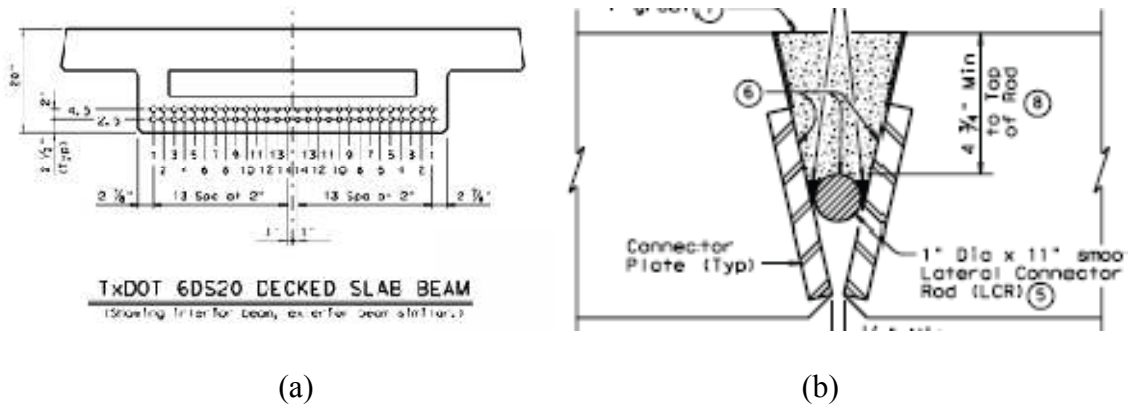


Figure 2.18: Decked Slab Beam Section (Texas) (TxDOT 2010)

The Deck Bulb-T bridges use a reinforced concrete topping but the Washington DOT states they permit faster construction than adjacent beam bridges and thus are used when faster construction is needed (see Figure 2.19).

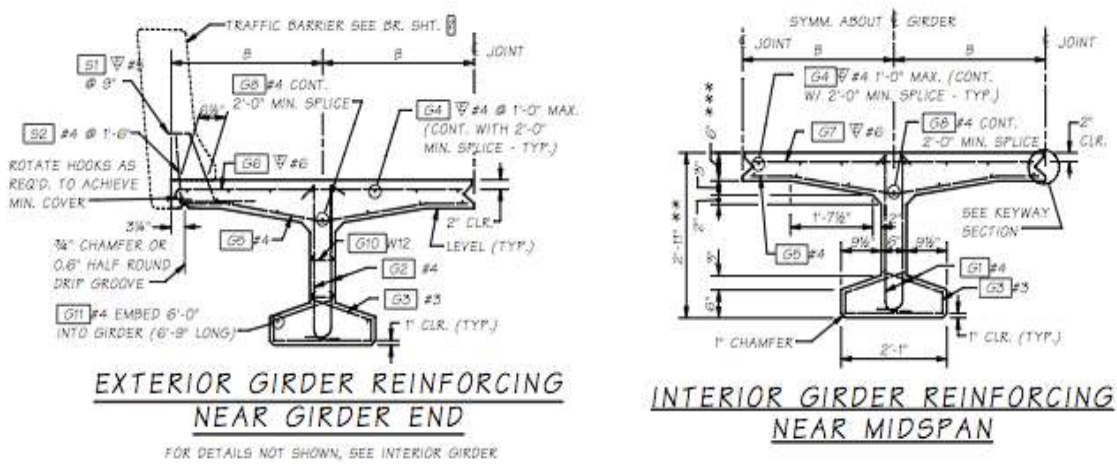


Figure 2.19: Deck Bulb-T Bridge Girder (Washington) (WSDOT)

The Inverset system, which was developed in Oklahoma in the 1980s, is used frequently by the NYSDOT. The system consists of two steel stringers supporting a composite concrete deck (see Figure 2.20). The main connection to adjacent units is accomplished using bolted diaphragm plates, which would prolong the construction. The NYSDOT has used this system at length to replace aging bridge superstructures and to increase vertical clearance at highway overpasses.



Figure 2.20: Inverset System Unit (New York) (Culmo 2009)

The Precast Concrete Institute Northeast has been working on a replacement system for adjacent box beams. It is called the Northeast Extreme Tee beam (PCI Northeast). This section is a squat double tee beam that ranges from 28 to 46 inches in depth. Two different versions of this beam are provided: the NEXT F, which requires an eight inch cast-in-place overlay (see Figure 2.21), and the NEXT D, which requires an eight inch wide shear key between adjacent sections (see Figure 2.22). These sections can be up to ten feet wide, therefore reducing the number of sections required and amount of joints that must be filled. The NEXT F provides stay-in-place forms, but does not require the same amount of rebar work needed for the Minnesota Inverse Tee beam mentioned previously. These sections are also much easier to inspect compared to the

box beams and hollow core beams. The Pennsylvania DOT (PennDOT) has recently adopted this system to replace their adjacent box beam bridges.

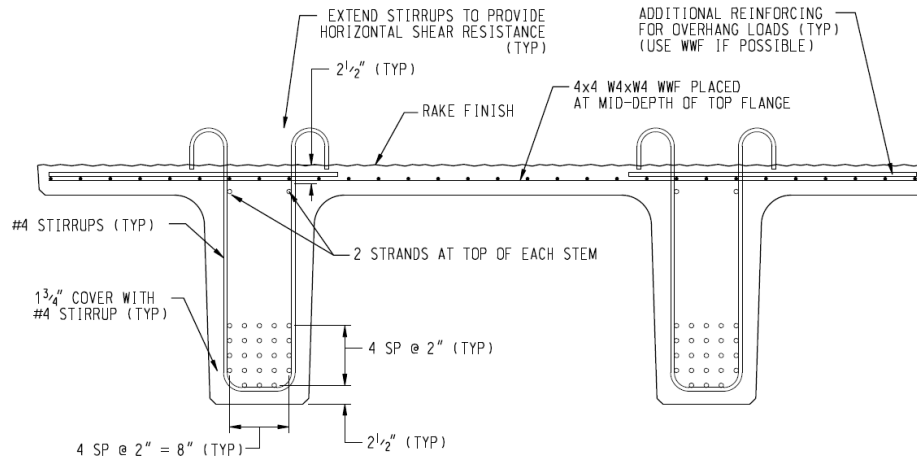


Figure 2.21: NEXT F Beam (PCI Northeast)

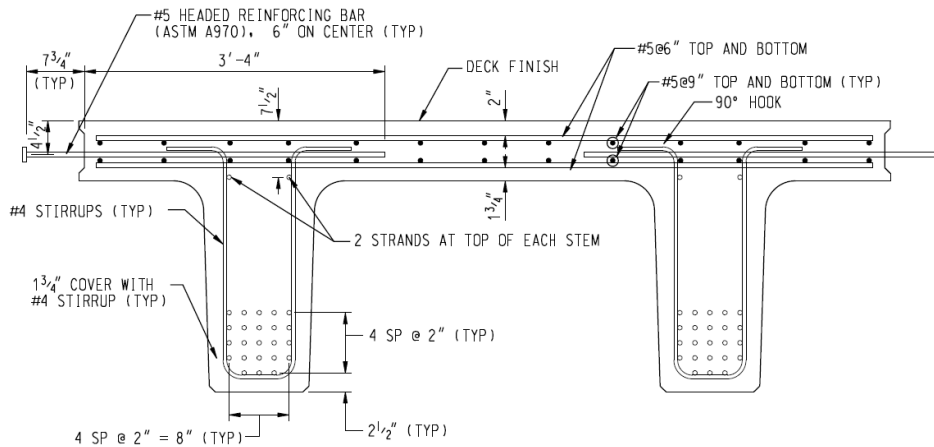


Figure 2.22: NEXT D Beam (PCI Northeast)

Another detail of interest is the cored slab bridges in North Carolina, which appear to be performing well and are similar to the SCDOT details, except transverse post-tensioning is used instead of tie rods. It was also discovered that the North Carolina DOT (NCDOT) has constructed cored slab bridges without any topping, although the

NCDOT did mention that on high ADT and NHS roads an overlay is required (Perfetti and Roberts 2010). The NCDOT is pleased with the performance of the un-topped cored slab bridges, but their use of un-topped cored slab bridges is relatively new thus there have not been many performance reviews on the bridges (Perfetti and Roberts 2010).

Systems that could be potential solutions or could lead to a better detail include:

- New York – Box Beams, Slab Beams, Inverset System
- North Carolina – Box Beams, Cored Slabs (with and without topping)
- Oregon – Box Beams, Slab Beams
- Texas – Box Beams with robust shear key, Slab Beams, Decked Slab Beams
- Washington – Slab Beams, Deck Bulb-T

Through conversations with the DOTs these systems seem to minimize the reflective cracking issues, ensure rapid construction, and in some cases allow for longitudinal continuity. These states, for the most part, are very pleased with the performance of these systems.

Contractor and Fabricator Interviews

To gain more detail on the practices and performance of bridges similar to the hollow core system, learn more about alternative systems, and to follow-up on the phone conversations with the DOTs, contractors and fabricators provided by target DOTs across the country were contacted by phone and email. Unique targeted questions were created for each contractor and fabricator provided by each state in order to learn more about the specific projects those contractors and producers work on. The complete interview summaries can be found in Appendix B.

Summarized findings from the fabricator interviews:

- NYSDOT has been using fewer adjacent box beams recently due to corrosion issues and has explored the use of the double-T sections (New York)
- Most bridges experiencing corrosion problems today were designed many years ago with different standards and the newer designs cannot be accurately compared to those older bridges in terms of susceptibility to corrosion (New York)
- Durability issues in northern states are no reason to abandon the use of these bridges in southern states (North Carolina)
- North Carolina uses double ducts occasionally to get more post-tensioning to limit cracking and this is not difficult for fabricators to handle (North Carolina)
- Transverse rods should be about five feet on-center maximum to provide a tensile tie across the keyway to minimize cracking (Washington)
- Potential fatigue issues associated with welded plates in the deck slab beam bridge shear keys should be investigated once bridges have been in service longer (Texas)

Summarized findings for the contractor interviews:

- Key width is too narrow and causes significant grout wasting and too much time to ensure the keys are properly filled (North Carolina)
- Concrete is the most expensive and least preferred topping (North Carolina)
- On a cubic yard basis, a concrete wearing surface on a cored slab bridge is much more expensive than a CIP deck on a prestressed girder bridge (North Carolina)
- Deck slab member can be installed in one day and the entire construction process is about one week for a deck slab bridge (Texas)
- Crane required for deck slab beam bridges is expensive since section is large (Texas)
- Open space in work area is the most important aspect when constructing deck slab bridges due to the large crane required (Texas)
- Certified welder is required to work on the welded plates in the shear key for the deck slab beam bridges (Texas)

Through all of the interviews with fabricators and contractors it was discovered that all of the targeted bridge types are sitting well with DOTs, contractors, and producers. Most producers seem to not prefer one adjacent type to the other, but they all tend to like the adjacent beams as a whole. The favoritism is mainly due to the construction speed, which is the main determining factor for contractor approval.

Durability is still a concern in these bridges, but the fabricators interviewed strongly believe that it is a maintenance issue and an issue due to outdated methods in the 40 to 50 year-old structures that are currently experiencing durability problems. Some of the newer details, like the Texas deck slab beams, need time in order to prove their durability and performance over their intended life but so far these newer bridges seem to be producing good results.

The literature reviews, web survey, and phone interviews have provided many different options that could be implemented and, in some cases, slightly modified to fit the objectives set by the SCDOT for this project. This information was presented to the SCDOT for review and an alternative section was selected for further study. This selection process is covered in Chapter 3.

CHAPTER THREE

BRIDGE DETAIL SELECTION

Introduction

The information compiled from literature searching, nationwide bridge standards, web surveys and designer, contractor and fabricator interviews was presented to the SCDOT by the Clemson research team in an interim report in June 2010 (Nielson et al. 2010). A joint meeting between the Clemson research team and the SCDOT steering committee was then planned to discuss the options and their ability to meet the desired project objectives. At this meeting, the three most intriguing alternative sections, selected by the steering committee, were identified and the actual implementation of these sections in South Carolina was investigated further by the research team. In August 2010, the research team and steering committee hosted a workshop and invited local contractors and fabricators that may eventually work on these bridges to attend and provide their perspective on the three options for the alternative section. After the workshop, the steering committee compiled all of the feedback from fabricators and contractors and selected the flat slab alternative section to be used in South Carolina on short span, rapid construction bridge projects.

Steering Committee Meeting

The information compiled by the research team was analyzed and the systems and practices that met the SCDOT's goals for the alternative section were identified. The following systems were presented by the research team to the SCDOT at the meeting:

Precast Modified Beam-in-Slab Bridge (see Figure 2.4), MnDOT Inverted Tee System (see Figure 2.6), Texas Decked Slab Beams (see Figure 2.18), Washington Decked Bulb Tees (see Figure 2.19), Inverset System (see Figure 2.20), and the NEXT F and D Beams (see Figures 2.21 and 2.22). In addition to these sections, at the steering committee meeting, a new system was proposed by the Clemson University research team. The Clemson section incorporated the Texas robust shear key and the headed reinforcing bars used in the NEXT beam shear keys into a section with a similar geometry to the current hollow core sections (see Figure 3.1). This section maintains the advantages of the NEXT beams but reduces the size of the sections, thus allowing for more manageable construction. It also eliminates the need to use forms for the shear key which is a disadvantage with the NEXT D section.

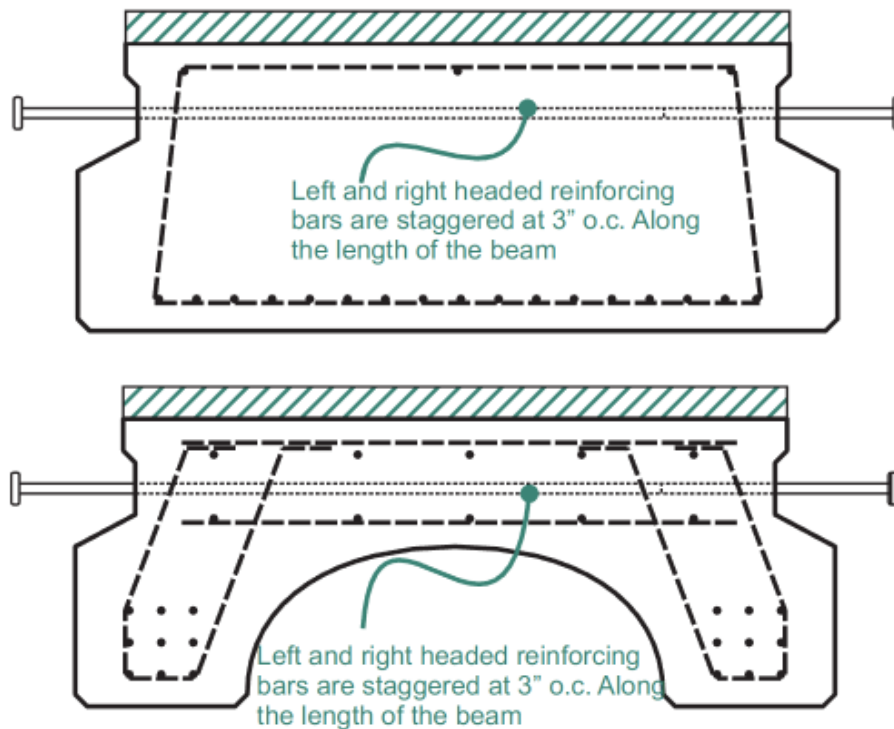


Figure 3.1: Clemson Section Preliminary Details

In addition, the robust shear key detail (see Figure 2.17) and the grout filled splice sleeves (see Figure 2.8) were presented as details that could be incorporated in the current SCDOT systems or the new system to improve them and meet the project objectives. Also, two potential continuity solutions were presented: the Texas detail (see Figure 2.14c) and the New York detail (see Figure 2.14a). Table 3.1 includes each of these systems or practices and their ability to meet the project goals set by the SCDOT.

Table 3.1: System Goal Attainment Chart

System	No Overlay	No Post-Tensioning	Minimizes Long. Cracks	Eliminates Long. Cracks	Faster than C.I.P. slabs	No AADT or NHS Restrictions	Initial Cost More than CIP Slabs
Texas/New York Robust Shear Key							
With overlay			✓		✓	✓	
Without overlay	✓		✓		✓		
Modified Beam-in-Slab System	✓	✓	✓	✓	✓		
MnDOT Inverted Tee System		✓	✓	✓	✓	✓	✓
Grout Filled Splice Sleeves	✓	✓			✓		✓
Texas Decked Slab Beams	✓	✓			✓		
Washington Decked Bulb Tee	✓	✓			✓		
Inverset System	✓	✓			✓		✓
NEXT F beam		✓	✓	unknown	✓	✓	
NEXT D beam	✓	✓	✓	unknown	✓	✓	
Clemson Section	✓	✓	✓	unknown	✓	✓	

At the meeting, the SCDOT steering committee indicated that the most important factors were having no AADT restrictions on the bridge and allowing for top-down construction. According to the SCDOT, voided sections were to be avoided if possible due to their tendency to allow water and deicing salts to corrode the prestressing steel from the inside-out. Since there is no proven inspection technique to determine hidden strand corrosion, the steering committee would like to move away from voided sections with the new alternative. The SCDOT also mentioned that wider sections are advantageous since they would require fewer joints and thus less pours and also fewer opportunities for water and salts to infiltrate the sections. Finally, the steering committee was wary of using the Texas Decked Slab beams or the Washington Deck Bulb Tee beams due to the dependence on welded connections which could cause fatigue problems over the service life of the bridge.

As a result, the Minnesota Inverted Tee System, the NEXT D beam, and the Clemson section were the three alternatives selected by the steering committee for further research and consideration. The NEXT D beam was preferred over the NEXT F since it did not require an overlay. All three of these systems have no perceived AADT restrictions and thus can be used on high volume roads. The comments from the steering committee on the three sections were as follows:

NEXT D Beam:

- No void material to hold down (fabricator advantage)
- Shear studs are not problematic for fabricators
- For low volume roads: fill keys and grind surface; for high volume roads: use overlay
- Barrier detail must be determined – mentioned cast-in-place barriers not preferred
- Heavy section (contractor disadvantage)

MnDOT Inverted Tee:

- Top-down construction may not work (contractor disadvantage)
- Could widen workable surface before casting for possible top-down construction
- Increase width of sections to minimize the number of joints
- Essentially this section still has an overlay, which is a disadvantage

Clemson Section:

- Concrete/grout in large key could slow down construction
- Section without voids would be preferred by fabricator (also mentioned solid is cheaper)
- Geometry must be refined and the various options explored
- Recommended grinding on this section; grinding is preferred due to cost

These issues were to be considered by the research team moving forward in addition to refining the geometry of all of the sections to meet the project objectives. The NEXT D section, which was designed for longer spans in the Northeast, must be scaled down in order to be reasonably compared with the other sections in consideration for use on short span bridge projects. The geometry of the Clemson section also must be investigated further in order to produce an economical section that still meets the project objectives and is fabricator friendly. The approximate weight of all of the sections should also be calculated so that the contractors at the workshop can make a reasonable comparison between sections.

In addition to narrowing down to three alternative sections, the steering committee also selected the Texas longitudinal continuity detail for further consideration as a solution to the transverse cracks common with the current system. Although it would not allow the bridge to achieve full continuity and the benefit of reduced positive moment in the spans, this detail was chosen since girder age does not have to be accounted for and cracking should be reduced. Moving forward, the longitudinal

continuity details should be adapted to fit the sections under consideration and they should be presented at the workshop for review by the SCDOT, contractors, and fabricators in order to make an informed final decision.

Workshop

The objective of the workshop was to combine the research team, steering committee, contractors and fabricators into one place and openly discuss the positive and negative aspects of each option in order to determine the best solution for all parties that would be impacted by the new alternative. Before the workshop, a web survey was created using SurveyMonkey and sent to local fabricators in order to gain some perspective on the relative fabrication costs of each section since prices could not be discussed publicly at the workshop. The web survey can be found in Appendix C. At the workshop, the results of this survey were presented (keeping all participants anonymous) and a brief review of the project progress and sections under consideration was given by the research team. Then, the contractors and fabricators were split into separate sessions and asked a series of questions on each section specific to their interests. These break-out sessions included discussion on the three options as well as two variations of the Texas continuity detail that could be used to make these bridges longitudinally continuous. The summary from the contractor and fabricator breakout sessions is provided in Appendix D. Finally, the group met together one last time to discuss the results from the break-out sessions and provide some final recommendations to the steering committee.

The survey sent to the fabricators included questions regarding fabrication cost, geometry, reinforcement, and surface roughening. The survey included three variations

of a four-foot wide and four variations of a six-foot wide Clemson section for the fabricators to review in addition to the NEXT D, NEXT F, and MnDOT Inverted Tee system. The Clemson section geometries were developed by the research team in order to provide a variety of options for the fabricators to review and determine which would best fit their needs (see Appendix C).

Four local fabricators participated in the pre-workshop fabrication survey and this allowed the research team to incorporate their comments into the presentation at the workshop for the benefit of all fabricators, contractors, and designers. According to the fabricators, the most relevant factors regarding cost when fabricating precast members were standardization, contract inspection of fabrication, labor intensive detailing, bed efficiency, and material shipping cost. There was no consensus among fabricators as to whether voided members are more expensive to fabricate than solid members. However, some fabricators did feel that voids were more expensive and also posed significant problems since they are hard to maintain during casting and they complicate concrete consolidation. All fabricators agreed that headed reinforcing bars for shear transfer between members would present problems during fabrication since a splice would be required and a two part form may be required. However, all fabricators seemed to agree that the headed bars could be implemented if necessary.

The normalized cost estimates of each section provided by the fabricators relative to the cost of a hollow core section are shown below in Table 3.2.

Table 3.2: Pre-Workshop Fabricator Cost Estimates

Section	Cost relative to Hollow Core				Average
	Fabricator #1	Fabricator #2	Fabricator #3	Fabricator #4	
Hollow Core	1.00	1.00	1.00	1.00	1.00
MnDOT Inverted T	1.28	1.46	1.59	1.14	1.37
NEXT D	1.96	-	3.12	1.87	2.31
NEXT F	1.42	-	2.24	1.46	1.71
CU 4' A	1.24	1.45	1.94	1.16	1.45
CU 4' B	1.24	1.46	1.71	1.14	1.39
CU 4' C	1.17	1.47	1.59	1.13	1.34
CU 6' A	1.57	-	2.35	1.66	1.86
CU 6' B	1.57	-	2.24	1.65	1.82
CU 6' C	1.70	-	3.35	1.58	2.21
CU 6' D	1.49	-	2.29	1.44	1.74

These values reflect the ratio of the cost per linear foot estimate for the section to the cost per linear foot estimate for the hollow core section. These values are not adjusted based on the width of the sections, which would affect the comparison since the sections vary in width. The cost relative to the hollow core section normalized to a standard width is shown below in Table 3.3.

Table 3.3: Normalized Fabricator Cost Estimates

Section	Cost relative to Hollow Core
NEXT D	0.87
Clemson 6'	0.87
Clemson 4'	1.00
MnDOT Inverted T	0.68

Since there are fewer sections required when using these sections (compared to the current hollow core details), although initially these systems appeared to be more expensive they all have been estimated by the fabricators to be equal to or less expensive per linear foot than the hollow core system. The Clemson six-foot section is preferred over the four-foot section since fewer joints are required and it is less expensive to fabricate. The NEXT D estimates were based on an eight-foot wide section with a 28-inch depth, and this geometry should be scaled down for use in South Carolina since it was intended for much longer spans in the Northeast. Therefore, the NEXT D section may actually be the most cost-effective system from a fabrication perspective once it is scaled down to a reasonable geometry for the target span length.

The fabricators thought all three alternatives were more difficult to fabricate than the current hollow core system, although they indicated the NEXT D system would be the easiest of the three alternatives. The Minnesota DOT detail had a low center of gravity, which was an advantage to the fabricators; but it included projected steel, required a difficult side finish, would be difficult to screed, and removing the forms would be challenging. The fabricators did not find many advantages with the Clemson section, and were especially concerned with using the section since the width would be difficult to adjust and thus it would be difficult to utilize the same forms for different sized sections. The fabricators seemed to prefer the NEXT D beam since the side forms could be reused for multiple depths and widths, it is quite versatile, and the fabrication difficulty is most similar to that of the hollow core system. One disadvantage the fabricators found with the NEXT D was they would have to purchase new forms to begin

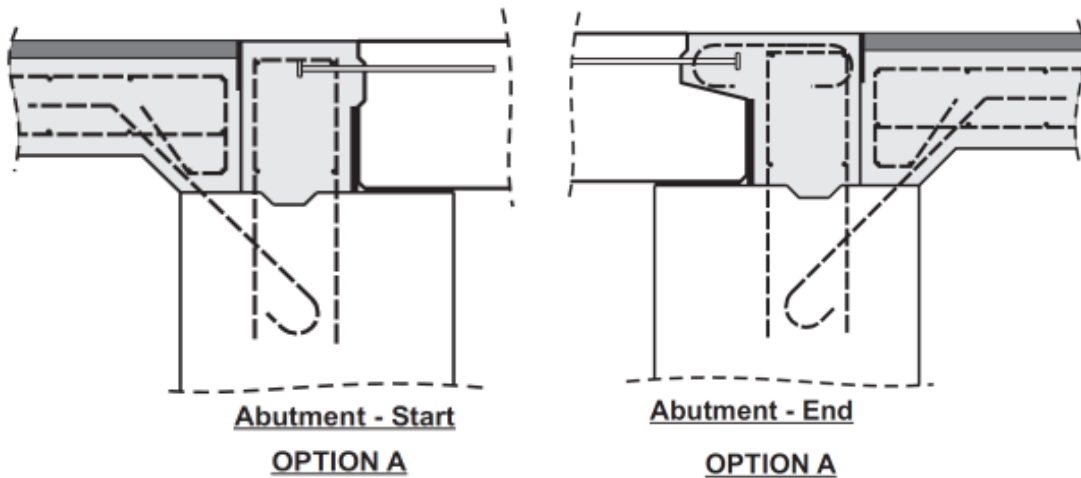
casting these sections. The fabricators also felt that the projected headed reinforcing bars were too frequent and that could be a challenge in fabrication.

The contractors felt both the NEXT D and Clemson sections would take relatively the same amount of time to construct as the hollow core but felt the MnDOT section would take much longer. The contractors also felt the MnDOT section would cost more to construct than the other two sections. Some advantages to the MnDOT section from a contractor perspective include: the bottom flange can be used as a form, the key would be filled properly, and no post-tensioning is required. In addition to cost and time to construct, disadvantages include setting these up for multiple span bridges and the safety issues related to the transverse hooks. The Clemson section was attractive to the contractors since it was a good width, had fewer joints to grout, did not require formwork, and required minimal rebar use. However, the Clemson section required headed reinforcing projections that would be difficult to deal with if the bars were not aligned properly. This was also a disadvantage with the NEXT D beam. The NEXT D beam was also the heaviest section by a large amount, which would require large cranes and increase the construction difficulty. The NEXT D was attractive to the contractors, however, since there was no deck to pour and the key would be simpler to grout.

Overall, through the contractor and fabricator break-out sessions, it appeared the NEXT D was the most preferred section, although there were still clear disadvantages to selecting the section, namely the weight. Both the contractors and fabricators offered some changes that could improve the section. The contractors proposed to use stay-in-place forms for the key detail and to add two inches of cover to the top of the slab for

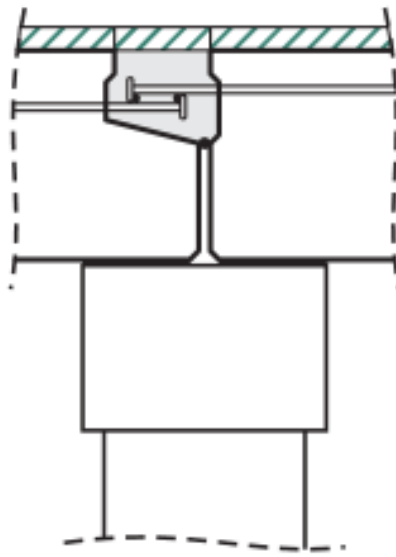
grinding purposes. The contractors also wanted the depth decreased to around twenty inches to decrease the section weight and allow for smaller cranes to be used. The fabricators suggested using sleeves or another alternative to the headed reinforcing bars, and also mentioned that the section width must stay under twelve feet in order to comply with transportation regulations.

Two continuity options were presented for review at the workshop. The headed option (see Figure 3.2) was viewed as problematic by the contractors and fabricators since the ends are not symmetric. The contractors believed there was potential for reinforcement conflict in the detail and the fabricators felt the section was not very flexible, especially for a side form. Both groups favored the hooked continuity option (see Figure 3.3) due to the symmetry and the contractors favored it since it allows for time between pouring the approach slab and setting the members.



(a) Approach Slab Continuity

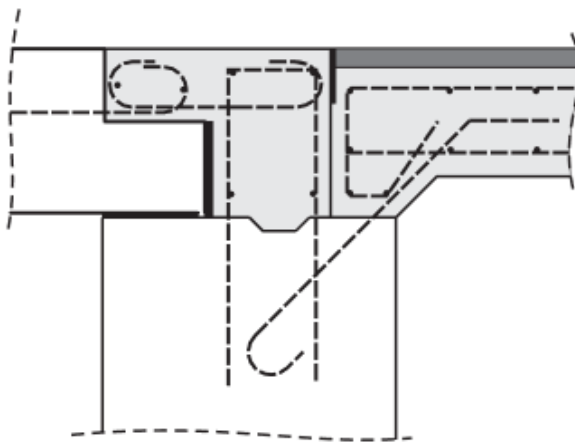
Figure 3.2: Headed Continuity Option



INTERMEDIATE PIER - A

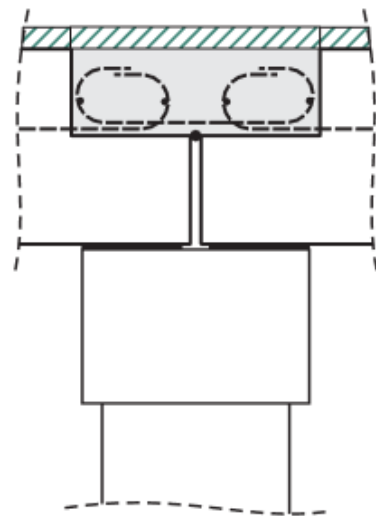
(b) Intermediate Pier Continuity

Figure 3.2 (Continued)



Abutment - Both

OPTION B



INTERMEDIATE PIER - B

Figure 3.3: Hooked Continuity Option

Detail Selection

Overall, it was clear that the hooked continuity option should be investigated further and applied to the final details for the selected alternative. Also, through the break-out sessions and fabrication pre-workshop survey, the NEXT D was determined by the steering committee to be the best solution that would not only meet the project objectives but would also benefit the fabricators and contractors that would work on the bridges. The main concerns with the NEXT D section were weight (contractors) and new forms (fabricators). In the design phase, the research team will seek to improve upon the weight of the section by adjusting the current detail used in the Northeast to fit the spans necessary for this project in South Carolina. The NEXT D was designed for much larger spans than required for this project, thus the research team should be able to decrease the size of the section and improve upon the weight. The SCDOT suggested using out-of-state fabricators during the first few projects in order to prevent the in-state fabricators from investing in forms that they would hardly use. This way, if the section is successful, the fabricators can then invest in new forms since they would be certain they would be used on a myriad of bridge projects.

The following chapter will focus on the design of the NEXT D section, specifically: identifying the design issues associated with the section, adjusting the geometry to meet the target span length in South Carolina, and designing the slab for the section.

CHAPTER FOUR

NEXT D DECK DESIGN

Introduction

The SCDOT project steering committee selected the NEXT D beam to be used as the flat slab alternative in South Carolina. The NEXT D beam, developed by Precast Concrete Institute Northeast (PCINE), was originally designed for medium-span bridges (40 to 80 ft. in length) and therefore the geometry must be adjusted for the section to be economical for the project's target span length of 22 to 40 ft. The design issues associated with this section were identified and the unknowns and limitations of the current design were acknowledged through a phone conversation with the NEXT beam developers (Culmo et al. 2010). All of these issues, which are laid out in the next section, must be addressed before this system can be implemented confidently. However, a preliminary design of the section is needed to be able to work out the approximate geometry of the new section. CONSPAN (Bentley Systems 2010) was used to perform the preliminary design for the section. Once the geometry was defined, the reinforcement in the slab of the section and shear key was selected by performing a deck design in accordance with the AASHTO LRFD Specifications (AASHTO 2007). The AASHTO equivalent strip method was used to determine the design forces for the slab, the appropriateness of which is investigated in Chapter 5.

Design Issues

The research team identified a list of design issues that must be considered when developing the details for the NEXT D system. These items were either brought up during conversations with the SCDOT steering committee or at the workshop by contractors or fabricators. Design issues to be considered for the NEXT D section include:

- Geometry
- Transverse reinforcement in slab
- Shear key
- Construction loads
- Distribution factor
- Longitudinal continuity
- Barriers
- Bearing details
- Approach slab
- Crown
- Handling details
- Vertical curve
- Skew

The immediate issues that must be studied involve the section geometry, the transverse reinforcement in the slab, the shear key headed reinforcing bars, and consideration of the construction loads on the system. The shear key will be assumed to act as a moment connection and will be checked for moment capacity based on the resistance of the concrete and headed reinforcing bars. The construction loads are a major concern with this section since it is a heavy section and thus may require larger cranes than currently used in South Carolina on these short span bridges. Remembering that top-down construction is a desired characteristic of this new section, one must include crane loads in the design process. These loads will be checked during the slab

design phase. An appropriate live load distribution factor will be determined once the actual behavior of the shear key is known, which will be studied in depth by the research team in future phases of the project. For the preliminary design of the section, the live load distribution factors will be calculated using CONSPAN (Bentley Systems 2010), which uses the AASHTO closed form equations, specifically the AASHTO Section I equations for the NEXT D beam (AASHTO 2007). Section I is defined as “Precast Concrete Double Tee Section with Shear keys and with or without Transverse Post-Tensioning” (AASHTO 2007). The longitudinal continuity details will be based on the hooked details presented at the workshop (see Figure 3.3) and these will be incorporated once the entire section design is complete and the bridge layout is known. The barrier will be assumed to be the same cast-in-place slip-form barrier used by the SCDOT for the hollow core systems (see Figure 4.1).

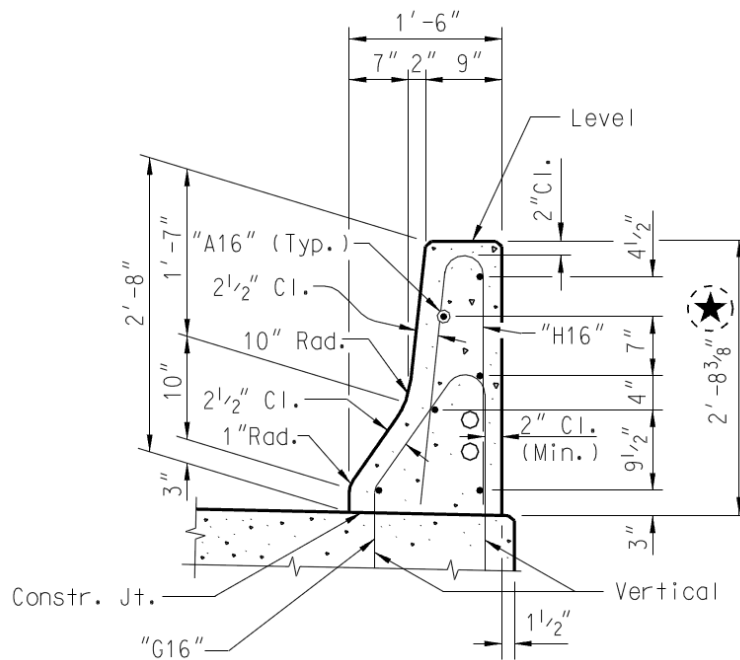


Figure 4.1: SCDOT Barrier Detail (SCDOT 2010)

For the preliminary design, a straight, non-skewed, 48-ft. wide, 40-ft. span bridge will be selected since 40-ft. span bridges represent the upper bound of this current project.

NEXT Developers Conference Call

In order to learn more about the NEXT D development and to learn of any recent research on the section, the developers of the section were contacted and a conference call was held between the lead developers, Michael Culmo and Rita Seraderian, and the research team (Culmo et al. 2010). The conversation shed more light on the history of the NEXT sections and the perceived advantages and disadvantages of the section and the desired future research involving the sections.

They are currently not suggesting using the NEXT D on highways, since there is no rigorous experimental or analytical study to confirm the performance of NEXT D systems. The NEXT D initiative began as a solution for low-volume bridges where a cast-in-place deck was not desirable. The NEXT D was not intended, originally, to be used for high-volume roads although the potential is certainly there. The developers would like to see more research performed on the shear keys before they will confidently promote these sections for use on high ADT roads. Overall, these systems will be cost effective compared to cast-in-place slabs and very durable, so the developers believe once the shear key connection is verified through research these systems will make for very efficient solutions (Culmo et al. 2010).

Since the research team was planning to scale down the geometry of the section for use on short span bridges in South Carolina, the developers were asked if they had considered using the section with a lower profile on short span bridges. The developers

had not seriously considered this, since they believed the sections may not be competitive with flat slab bridges for short spans. However, they believed a shallow NEXT D could certainly provide better performance; they just had not considered it for anything beyond medium span bridges. One potential issue identified with using a shallow NEXT D is the overstressed anchorage blocks in the fabrication yard due to the small strand eccentricity. This should be checked before the section is implemented into a design. The developers believed the SCDOT may eventually want to adapt the section for longer spans if they are pleased with the performance of the short span NEXT D bridges, so fabricators may want to look into adjustable forms that could accommodate different depths and stem spacing (Culmo et al. 2010).

They originally desired a shear key that did not require a form, but contractors informed them they were pleased with the key and could form it easily, so the current detail was selected. According to the developers, research by the University of Tennessee confirmed that #5 headed reinforcing bars lapped six inches provide enough capacity for the AASHTO LRFD moment at the shear key for an adjacent decked bulb tee bridge (Li et al. 2010). Test results were evaluated based on flexural capacity, curvature behavior, cracking, deflection and steel strain. Based on these test results, the longitudinal joint detail was found to be a viable connection system that transfers the forces between the adjacent decked bulb tee girders (Li et al. 2010). Furthermore, they are comfortable with the connection capacity verified by the University of Tennessee and are comfortable using the connection for the NEXT D section, but would like for more

research to be done on the actual design loads in the shear key. This will be covered in later phases of the research project.

The developers concluded that for the NEXT D, the equations for an “adequately connected” Section I in AASHTO should be used to calculate the live load distribution factor (AASHTO 2007). Culmo also believed the AASHTO equivalent strip method for calculating shear and moment at the joint is a very conservative approach, and this assumption will be checked for accuracy in Chapter 5. AASHTO assumes infinitely rigid supports for the slab but semi-flexible supports and continuous beam action is the expected structural behavior (AASHTO 2007).

In addition to the geometry, shear key, and live load distribution factors, the developers also provided some insight on the use of these sections for skewed bridges and their current research on bearing details for the sections. The skew is currently limited to 30 degrees for these sections, which is a problem in the Northeast since many bridges have at least a 30 degree skew (Culmo et al. 2010). The skew is limited due to low torsional stiffness and the resulting twisting which occurs at the time of strand release. They would like for research to be conducted to look into this issue and verify that skews could be safely achieved with these sections. Finally, they would like to create an adjustable bearing detail to account for shimming (since beams do not normally sit properly), and they believe this can be done but want to be careful with camber and matching up of the beams (Culmo et al. 2010).

Overall, much was gathered from this conference call with the NEXT D developers. The concerns and limitations with the NEXT D indentified by the developers

were shared with the SCDOT and the overall project objectives were shifted to ensure sufficient research would be performed on the NEXT D before it was implemented for use in South Carolina. The first steps of the research involve:

- Preliminary beam design of shallow depth NEXT D sections
- Slab transverse reinforcement design (using AASHTO equivalent strip method)
- Shear key headed reinforcing bar design
- Consideration of construction loads on slab design
- Check AASHTO equivalent strip method's slab design forces for appropriateness

These items will be covered in the following sections of Chapter 4 and Chapter 5 and further research will be necessary to verify the other items of interest.

Preliminary Beam Design

The section geometry was scaled down to be comparable to the geometry of the current hollow core details used in South Carolina. The depth was decreased to 20 inches, and the slab was maintained at eight inches deep, leaving a 12-inch depth for the webs. An additional two inches was added to the top of the section for grinding purposes, and this was factored into the design by accounting for the weight of the two-inch grinding surface. The web spacing was set at a constant 36 inches center-to-center, and the beam spacing could be varied from six-feet (NEXT D6) to eight-feet (NEXT D8) in width by sliding the forms and adjusting the length of the slab cantilevers. The web width was maintained at 15 inches at the top with a three-eighths inch per foot slope down the sides (see Figure 4.2).

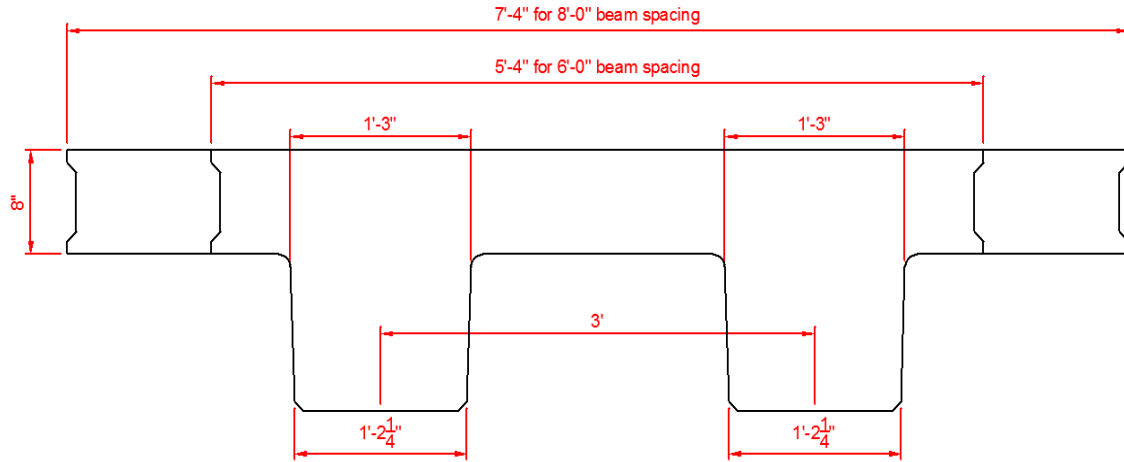


Figure 4.2: New NEXT D Beam Geometry

Each beam section must be spaced eight inches from the adjacent beam sections in order to leave space to pour an eight-inch wide shear key. Therefore, the actual widths of the sections themselves are 64 inches for the NEXT D6 and 88 inches for the NEXT D8 to achieve effective section widths of 72 inches and 96 inches, respectively (see Figure 4.2). This geometry was selected in order to maintain the low-profile advantage of the hollow core and also to ensure that fabricators could use the same adjustable forms to create both the NEXT D6 and NEXT D8. The sloped webs are fabricator friendly because they facilitate the beam removal from the forms. Thus, this cross-section characteristic was maintained from the original NEXT D details. The shear key detail from the original NEXT D was maintained as well (see Figure 4.3).

CONSPAN (Bentley Systems 2010) was used to perform the preliminary beam design for the new section to ensure the necessary prestressing strands would fit and to ensure the design was economical. In the CONSPAN models, the contribution of the shear key to the moment capacity of the beams was not considered. The cross-section of

the beams was assumed to be only the 64 inch and 88 inch wide sections for the NEXT D6 and NEXT D8 respectively. The weight of the shear key was accounted for as an additional dead load (see Figure 4.4).

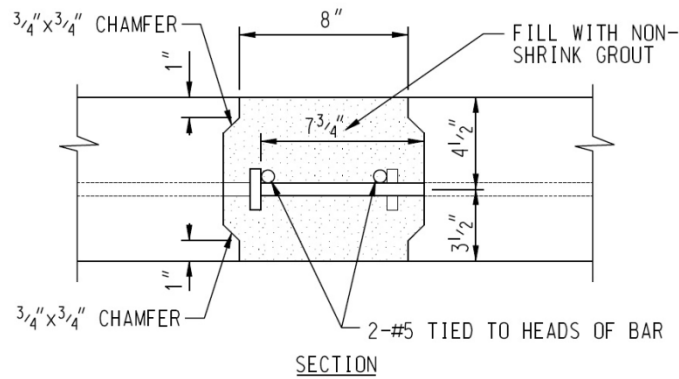
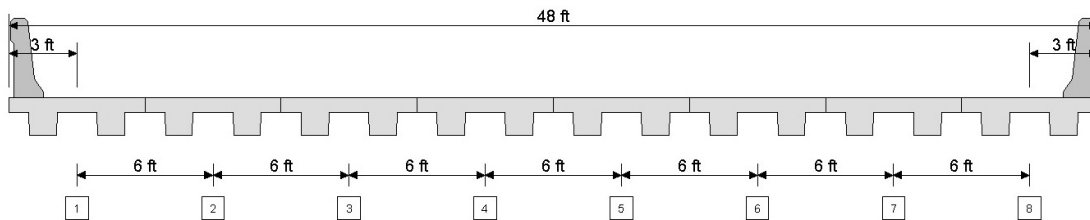
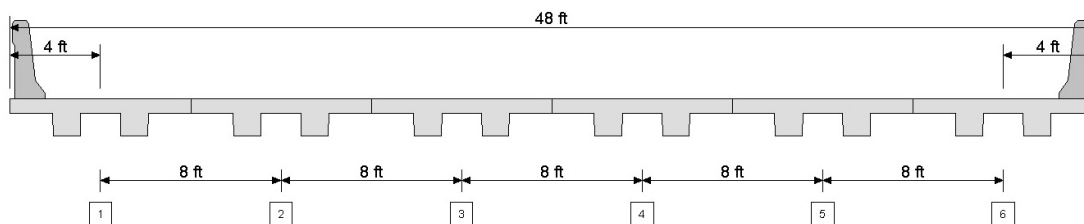


Figure 4.3: NEXT D Shear Key Detail (PCI Northeast)



(a) NEXT D6



(b) NEXT D8

Figure 4.4: CONSPAN Models

The material properties used for the NEXT D are summarized in Table 4.1.

Table 4.1: NEXT D Material Properties

Concrete Compressive Strength (f'_c)	
Girder Release	5.2 ksi
Girder Final	6.5 ksi
Deck Final	6.5 ksi
Concrete Unit Weight (γ_c)	0.150 k/ft ³
Yield Strength of Mild Steel Reinforcement (f_y)	60 ksi
Tensile Strength of Prestressing Steel (f_{pu})	270 ksi

The model was subjected to the HL-93 loading defined in AASHTO (AASHTO 2007) as well as the weight of the sections, the weight of the grinding surface, and the weight of the barriers. The barrier weight was assumed to contribute solely to the exterior beams and was based on the current SCDOT barrier detail for flat slab bridges (see Figure 4.1). The model did not assume continuity between the barrier and the deck. The NEXT D6 beam design required twenty-eight ½”-diameter 270-ksi low-relaxation prestressing strands and the NEXT D8 beam design required forty of these strands. These designs use around five strands per foot width, which is comparable to the current SCDOT hollow core details. Four strands per section are placed 17.5 inches from the bottom of the webs to allow for stirrups to be tied and to account for tensile stresses in the top zones of the sections after release. The remaining strands are evenly distributed in the webs of the section at distances of 2.5”, 4.5”, and 6.5” from the bottom of the section (see Figure 4.5). CONSPAN confirmed the NEXT D6 and NEXT D8 sections conformed with the AASHTO requirements for flexure, final stresses, and release stresses. Therefore, these geometries will be assumed to be sufficient for the section and

these configurations will be used to calculate design forces in the slab and design the transverse slab reinforcement and headed reinforcing bars in the shear key.

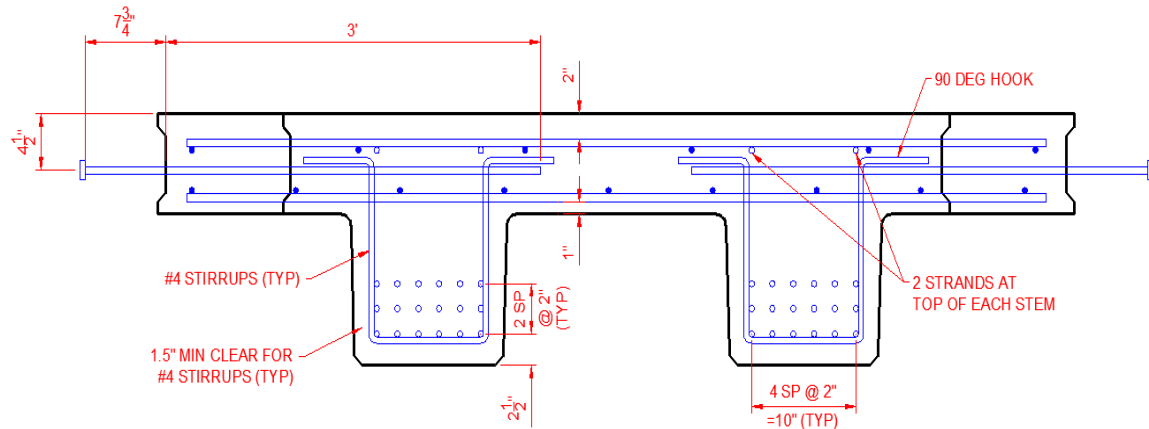


Figure 4.5: Preliminary NEXT D Prestressed Design

Slab Design

The slab of the NEXT D beams was designed using the AASHTO equivalent strip method to determine the design forces (AASHTO 2007). The complete design sheets can be found in Appendix E. In order to use the equivalent strip method, the service loads must be calculated. The service dead loads included the weight of the slab and grinding surface and the weight of the barrier. The service live load was taken as the AASHTO wheel load modeled as a concentrated load per AASHTO 4.6.2.1.6 (AASHTO 2007). The wheel load is a set of 16.0 kip concentrated loads spaced six feet apart that can be placed at any location along the slab up to one foot from the front face of the barriers.

The equivalent strip method calls for classical beam theory to be used, assuming the slab acts as a simply-supported or continuous beam with infinitely rigid supports, to determine the service live loads (AASHTO 2007). SAP2000 was used to perform the

structural analysis and calculate reactions and maximum moment and shear values to be used as the service loads for design (Computers and Structures). The NEXT D8 web spacing was used for the slab design since it produced the maximum force effects and consistent transverse reinforcement was desired for the different sections. The webs provide support for the slab and were modeled as infinitely rigid supports (AASHTO 2007). The slab was modeled as a continuous beam (see Figure 4.6).

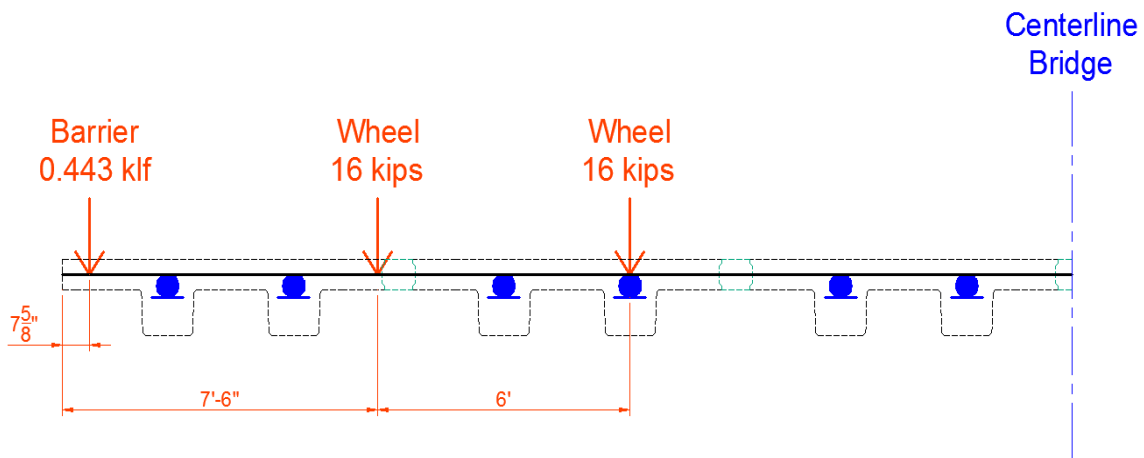


Figure 4.6: NEXT D8 Slab: Structural Idealization

The truck which is modeled by the 72-inch spaced wheel loads was moved across the width of the bridge at midspan in order to determine the maximum force effects. The truck locations producing the maximum positive and negative moment, maximum shear, and maximum moment in the shear key were determined and these values were used for design. The positioning shown in Figure 4.6 creates the maximum positive and negative moment in the slab. Since the 48-ft. wide model allows for three design lanes; one, two, and three trucks were applied to the bridge and the resulting force effects were multiplied by the corresponding AASHTO multiple presence factors defined in AASHTO Table

3.6.1.1.2-1 (AASHTO 2007). One truck controlled all cases, partly since a second truck farther along the span has little effect on the forces under the first truck due to load distribution and partly because the multiple presence factor is highest for the case of a single truck. The multiple presence factor accounts for the probability of the given loading scenario, therefore it is highest with one truck since that scenario is more probable than multiple trucks simultaneously traversing the bridge.

The AASHTO equivalent strip width equations are listed below for positive moment, negative moment, and slab overhang calculations (Equations 4.1-4.3) (AASHTO 2007). The support spacing measured in feet, S , is taken as the minimum spacing between supporting components, which in this case is three feet. The service live load positive moment measured in kip-ft./ft., LL , is calculated by multiplying the load from analysis, Q (kip-ft.), by the multiple presence factor, m , and then dividing by the equivalent strip width (ft) calculated by the equations below. The values for the $+M$, $-M$, and *Overhang* strip widths determined using the empirical equations below are in units of inches.

$$+M: 26.0 + 6.6S \quad (4.1)$$

$$-M: 48.0 + 3.0S \quad (4.2)$$

$$Overhang: 45.0 + 10.0X \quad (4.3)$$

$$LL = \frac{mQ}{strip\ width} \quad (4.4)$$

The design positive and negative moments, shear, and support reactions for this case were compared to the design forces from the construction load case in order to determine the critical load case for design. For the construction load case, the weight of

the crane required to lift the NEXT D8 sections was found using American Crane Corporation's HC 80 crane specifications (American Crane Corporation 2001) which was recommended by a bridge contractor that erects hollow core sections in South Carolina (Geddis and Deery 2010). This load case considers the effects of a crane carrying a NEXT D8 section where the crane would be located on top of the previous span in order to set the next span on a multi-span bridge (see Figure 4.7).

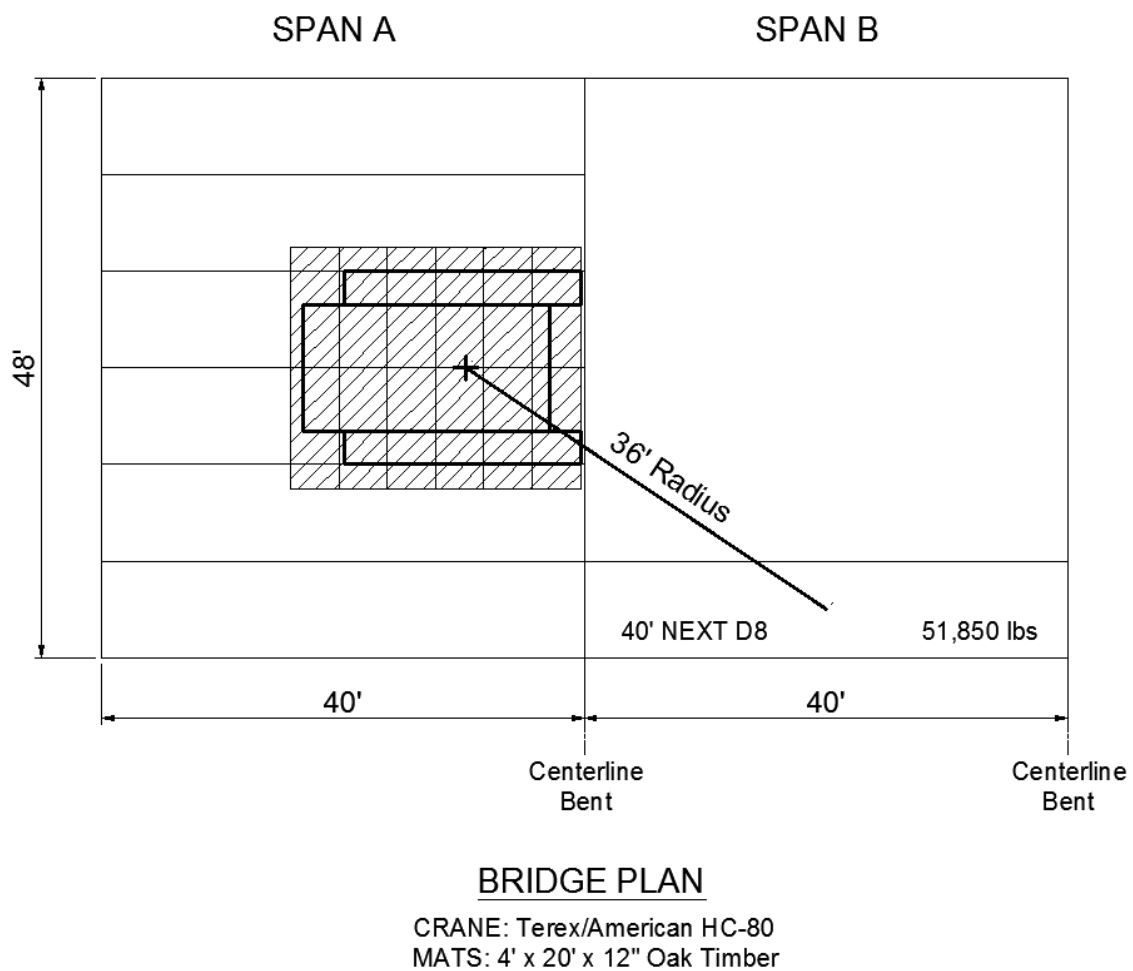


Figure 4.7: Construction Load Case

The American Crane Corporation specifications suggest using 4' x 20' x 12" timber mats in order to distribute the construction loads to numerous sections. Therefore, the NEXT D sections in span A must account for the weight of the crane and all of the attachments, the weight of a NEXT D section to be set in span B, and the weight of the timber mats. This loading scenario was compared to the AASHTO HL-93 load case to determine the design forces.

The AASHTO HL-93 load case controlled the design and these forces were used to check flexure, crack control, distribution reinforcement, and all of the detailing requirements for the slab (AASHTO 2007). The reinforcement required for the slab is given below in Tables 4.2 and 4.3 and compared to the reinforcement used in the original NEXT D details. The typical section reinforcement is shown in Figure 4.8.

Table 4.2: Slab Transverse Reinforcement

Location	NEXT D (South Carolina)	NEXT D (Northeast)
Bottom of Slab	#5 bars at 11" (0.34 in ² per ft)	#5 bars at 6" (0.62 in ² per ft)
Top of Slab	#5 bars at 12" (0.31 in ² per ft)	#5 bars at 6" (0.62 in ² per ft)
Top of Slab (overhang)	#5 bars at 6" (0.62 in ² per ft)	#5 bars at 6" (0.62 in ² per ft)

Table 4.3: Slab Longitudinal Reinforcement

Location	NEXT D (South Carolina)	NEXT D (Northeast)
Bottom of Slab	#4 bars at 10" (0.24 in ² per ft)	#5 bars at 9" (0.41 in ² per ft)
Top of Slab	#4 bars at 18" (0.13 in ² per ft)	#5 bars at 9" (0.41 in ² per ft)

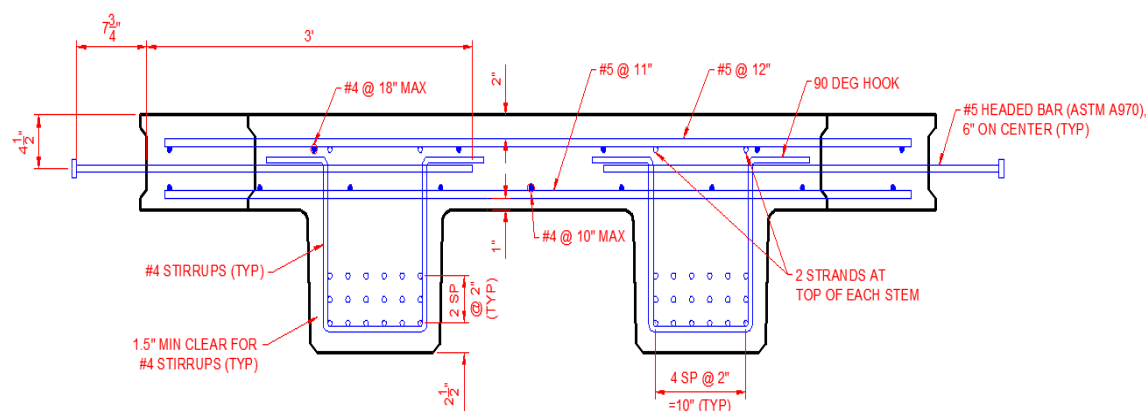


Figure 4.8: NEXT D Beam Typical Reinforcement

The slab design performed based on the AASHTO equivalent strip method required slightly less reinforcement than detailed in the original NEXT D details from the developers. This was expected, since the smaller beam spacing used in the South Carolina details causes smaller moment effects. The slab overhang was designed to resist negative moment effects caused by vehicle collision forces in the barrier. The overhang was found to require more steel than other sections of the slab, but not more than required in the original details.

In addition to the slab reinforcement design, the moment capacity at the shear key was checked. The proposed details for the NEXT D beam call for #5 headed reinforcing bars at six inches on-center through the shear key (see Figure 4.3). Using the AASHTO equivalent strip method to determine the design positive moment, the connection was found to be adequate assuming the headed reinforcing bars were located at the mid-depth of the key. The Northeast details place the reinforcing bars one half inch below the mid-depth of the key in order to increase the eccentricity of the steel and increase the positive moment capacity. Therefore, the check of the bars at mid-depth was conservative.

Conclusions and Closure

This chapter addressed the design concerns with the section that must be investigated before the section can be implemented confidently in South Carolina. It also focused on the preliminary NEXT D beam and slab design. The prestressing strands were found to be comparable to the amount the SCDOT currently uses for flat slab bridges, about five strands per foot width. In addition, the reinforcement in the slab was found to be comparable to the reinforcement used in the original NEXT D details (see Tables 4.2 and 4.3 and Figure 4.8)

The reinforcement in the slab and headed reinforcing bars in the shear key were all designed based on the assumptions in AASHTO for the equivalent strip method that the supporting components are infinitely rigid (AASHTO 2007). In Chapter 5, this assumption is checked for appropriateness with regards to the NEXT D section in order to ensure the design forces calculated by the method are conservative and the reinforcement selected through the AASHTO method is adequate for the section. The optimal depth of the headed reinforcing bars in the shear key is also studied by comparing the maximum positive and negative moment responses in the key and selecting a depth that provides adequate eccentricity for both moments.

CHAPTER FIVE

DECK SENSITIVITY STUDY

Introduction

The AASHTO equivalent strip method was assumed to be conservative when calculating the design forces and selecting the reinforcement for the slab in Chapter 4. The equivalent strip method assumes supporting components are infinitely rigid, and this assumption was checked for appropriateness for the NEXT D section (AASHTO 2007). OpenSees (University of California Berkeley 2009) and MATLAB (The MathWorks 2009) software were used to study the impact of various support (web) stiffness values on the force effects in the slab and to compare the effects when assuming infinite stiffness to the effects due to stiffness values calculated using classical beam theory. In addition to varying the web stiffness, the effects of varying the rotational and translational stiffness in the shear keys were studied. The shear key behavior was studied to gain an understanding of the impact of stiffness on forces in the shear key and to determine the optimal depth of the headed reinforcing bars in the shear key by comparing the maximum positive and negative moment responses in the keyway and selecting a depth that provides adequate eccentricity for both moment capacities.

Stiffness Parameters

The NEXT D slab across the width of the bridge was idealized as a continuous beam with translational springs centered at each web in order to account for the stiffness of the web (see Figure 5.1).

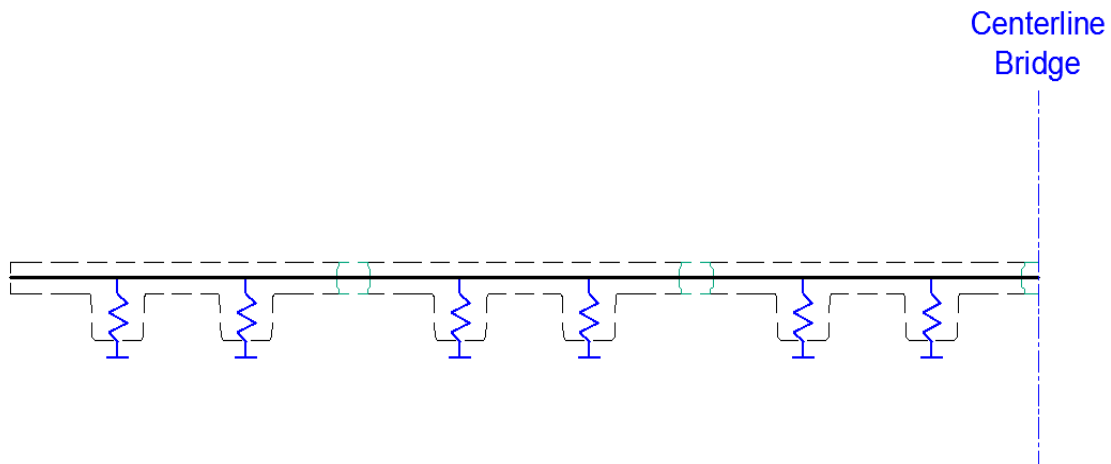
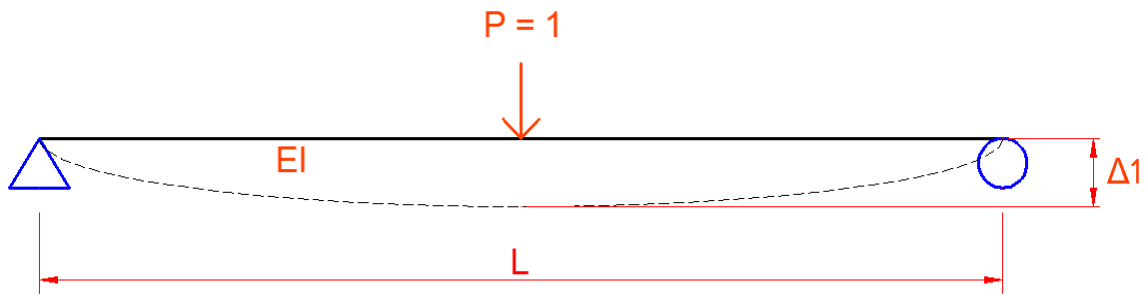


Figure 5.1: Slab Structural Idealization

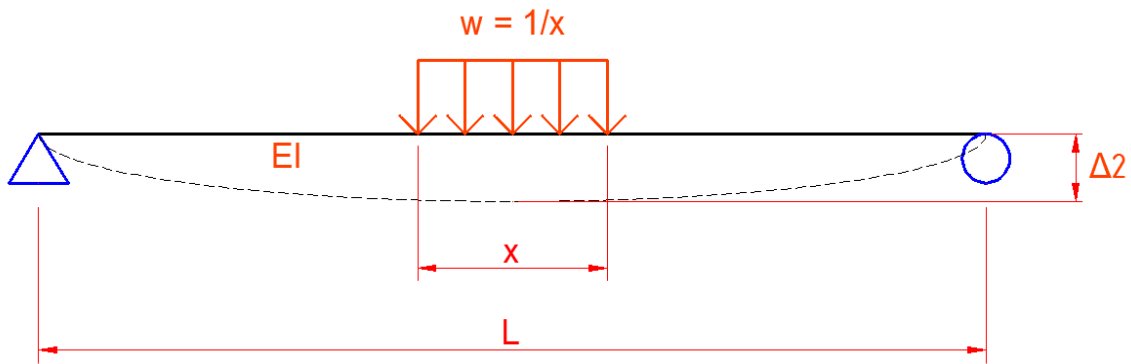
The AASHTO equivalent strip method assumes each of these translational springs has an infinite stiffness. In this portion of the project, the translational springs were assigned various stiffness values (k_{sup}) and the resulting maximum response values in the slab were recorded and compared. In addition, the translational (k_V) and rotational (k_M) stiffnesses of the shear keys were varied to determine the impact on the design forces in the keyways and throughout the slab.

In order to assess the accuracy of the equivalent strip method for the NEXT D beams, reasonable stiffness parameters were determined to compare the design forces using those parameters to the design forces when the supports are assumed to be infinitely rigid. The stiffness calculations are based on applying unit magnitude loads to the bridge and calculating the deflection and then obtaining the stiffness through Equation 5.1. The stiffness parameters were based on placing the truck loads at mid-span of the bridge in order to produce the largest deflection in the bridge and hence the smallest equivalent spring stiffness. In AASHTO, the equivalent strip width is limited to

12 ft., thus unit magnitude loading scenarios ranging from a point load at mid-span to a 12-ft. wide uniformly distributed load at mid-span were considered (see Figure 5.2).



(a) Unit Magnitude Point Load



(b) Unit Magnitude Distributed Load

Figure 5.2: Live Load Placement to Determine Web Stiffness

The relationship between force (F), stiffness (k), and deflection (Δ) is given in Equation 5.1, which was used to calculate the effective stiffness of each of these potential loading scenarios.

$$F = k\Delta \quad (5.1)$$

Virtual work was used to determine the following closed-form equations to determine the deflection (Δ) in each of the scenarios shown in Figure 5.2.

$$\Delta_1 = \frac{PL^3}{48EI} \quad (5.2)$$

$$\Delta_2 = \frac{wx \left(L^3 - \frac{Lx^2}{2} + \frac{x^3}{8} \right)}{48EI} \quad (5.3)$$

Using Equations 5.1 and 5.2, the translational support stiffness for the 40-ft. span bridge due to the point load was found to be 27.57 k/in and the translational support stiffness due to the distributed load, assuming the load is distributed over the maximum permissible 12-ft. width, was found to be 28.77 k/in from using Equations 5.1 and 5.3. Therefore, even with the largest allowable load distribution, the stiffness of the webs was much less than the assumed value of infinity by AASHTO. The effective stiffness values were calculated for the mid-span of the bridge, which are the most flexible values. The effective stiffness of the supports would increase from these calculated values for locations closer to the ends of the spans. However, only the most conservative scenario, the effective stiffness at the mid-span of the bridge, is considered in this study. The design forces resulting from these web stiffnesses were compared to the forces based on infinite web stiffness to check if the equivalent strip method was conservative. In addition to these stiffness values, 60 more values between 0.01 k/in and 1×10^{15} k/in were checked in order to generate sensitivity plots representing the complete relationship between deck support stiffness and the resulting slab forces for the NEXT D system.

Software

In order to calculate the maximum force responses for the system for a number of different stiffness values, OpenSees software was used in conjunction with MATLAB software. The OpenSees *simulation.tcl* and *BridgeDeck.tcl* scripts can be found in Appendix F and the MATLAB *plot_it.m* script can be found in Appendix G. The *simulation.tcl* script ran simulations that varied the transverse location of a single AASHTO HL-93 truck (AASHTO 2007) across the width of the slab. The script then called *BridgeDeck.tcl* which divided up the slab into small frame elements, and calculated member end forces for each element for every truck position. OpenSees then called *plot_it.m* to determine the maximum force responses for each truck position and also determine the response magnitude and location along the slab that was the controlling scenario. The *simulation.tcl* script was used to repeat this process for different stiffness values and at the end of each complete simulation a table of stiffness values and six different maximum force effects and their respective locations along the slab was produced by MATLAB (see Figure 5.3). The stiffness values that were varied in different simulations include: deck support stiffness, shear key translational stiffness, shear key rotational stiffness, and shear key translational and rotational stiffness simultaneously. The responses captured include: maximum positive and negative moments in the slab, maximum shear in the slab, maximum positive and negative moment in a shear key, and maximum shear in a shear key. The relationship between each stiffness value and each of these forces was studied for both the NEXT D6 and NEXT D8 sections.

Only one of the three stiffness parameters (web stiffness, shear key translational stiffness, and shear key rotational stiffness) was varied per simulation in order to properly illustrate the impact of one parameter at a time on the six measured force responses. As one parameter was varied, all other stiffness parameters were assumed to be 1×10^{15} k/in (translational) or 1×10^{15} k-in/rad (rotational), which was taken to be infinite rigidity throughout all of the simulations. Further studies involving other stiffness values for the control variables may be beneficial in order to fully understand the impact of these parameters on the design forces in the slab.

All loads applied to the structure were unfactored service loads and the only loads considered in this analysis were the weight of the barrier (assumed to act at the end of the slab), the weight of slab and grinding surface, and a single HL-93 truck load that was moved along the slab to produce the maximum responses. The set of two, concentrated, 16-kip truck axle loads at 72-inch spacing was checked at four-inch increments along the slab. The software divided the slab into a limited number of short elements and the concentrated live loads were automatically applied to the nearest member end node during the simulations. This range of truck positions was deemed adequate in order to assess the maximum force responses in the slab but further refinement of the position increments may improve the accuracy. The following flowchart in Figure 5.3 illustrates the logic used by the software.

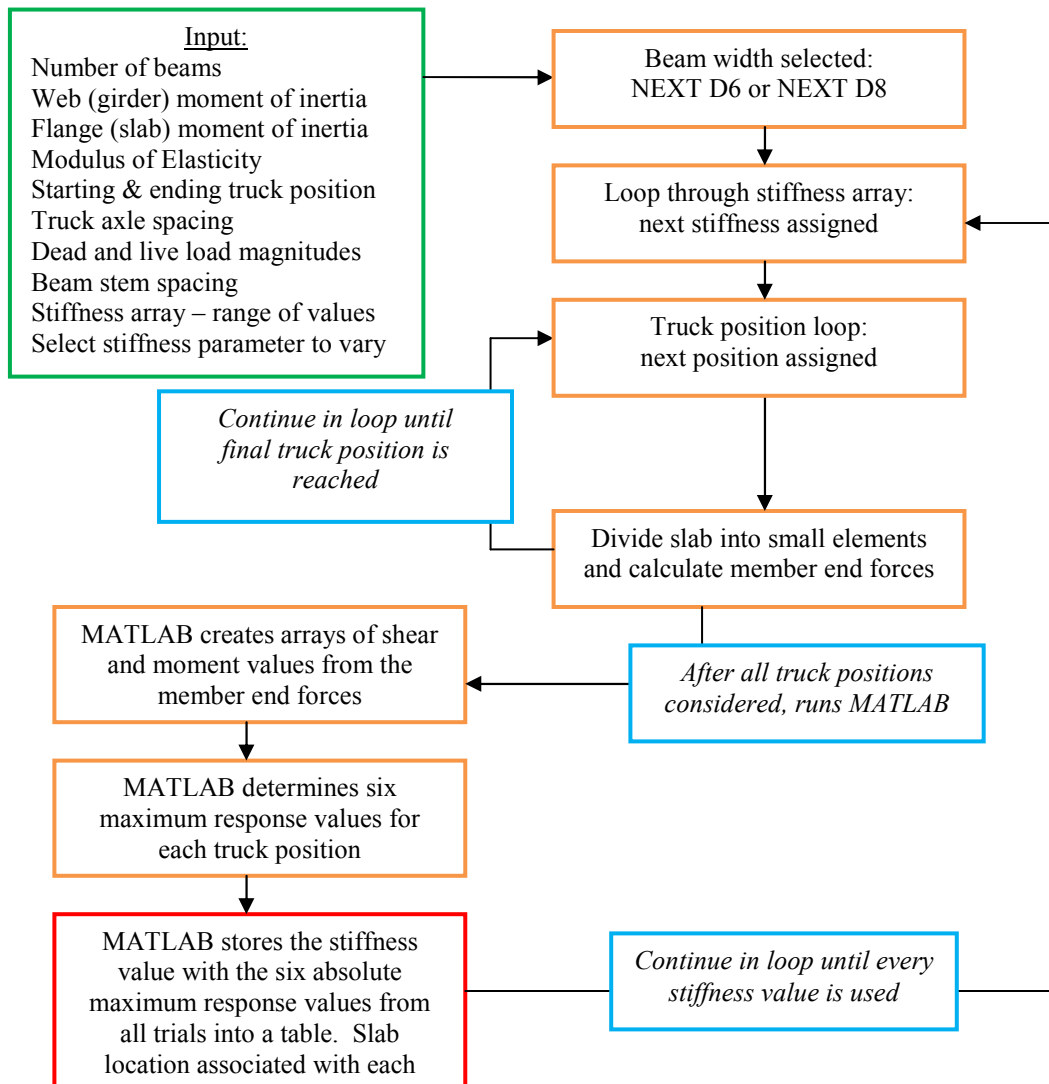


Figure 5.3: Software Flowchart

Results and Discussion

The complete set of plots illustrating the relationships between each stiffness parameter and each response value of interest can be found in Appendix H. The plots containing the stiffness values and the corresponding maximum response values were analyzed and since the response was found to be essentially constant on all plots once the stiffness reached 1×10^8 the plots only include a stiffness range of 0 to 1×10^8 . One of the first impressions from the plots was that many of the relationships were not a continuous function. This was a result of maximum response values from different locations along the slab since some stiffness values produced the maximum effects at different locations. This effect is illustrated in Figure 5.4, which shows the relationship between support stiffness and maximum negative moment in the slab.

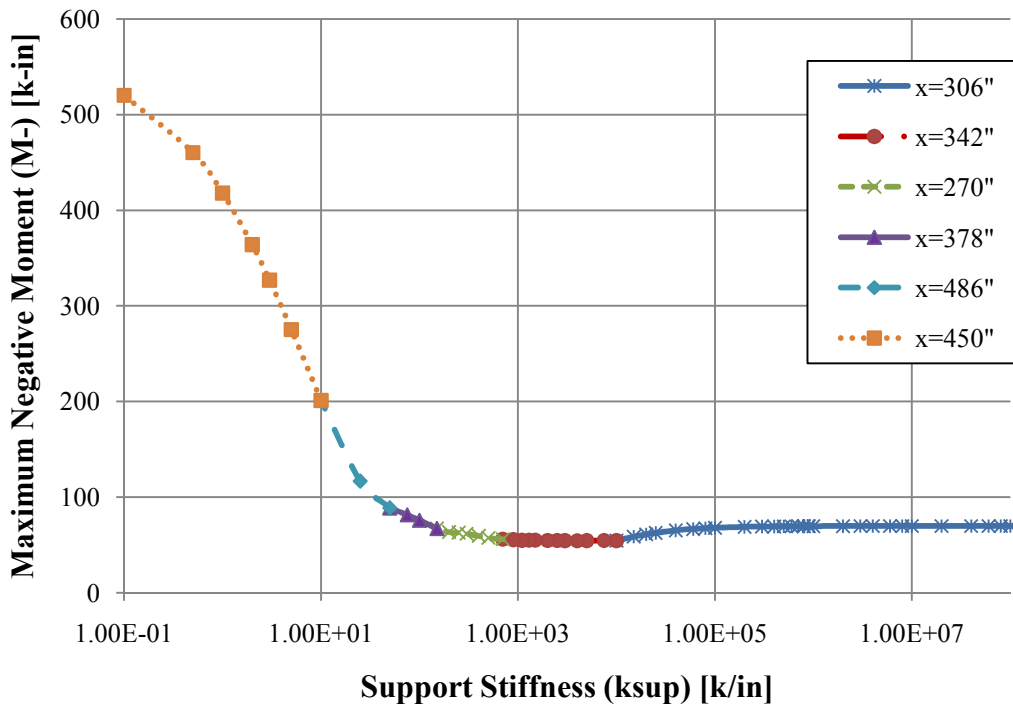


Figure 5.4: Influence of Support Stiffness on Maximum Negative Moment for NEXT D6

Response values from six different slab locations were compiled to create the plot. Overall, the trend illustrates that increased support stiffness causes lower negative moment responses in the slab, however the varying degrees of curvature are due to the responses obtained from six different locations. The locations contributing to the plot are shown in Figure 5.5.

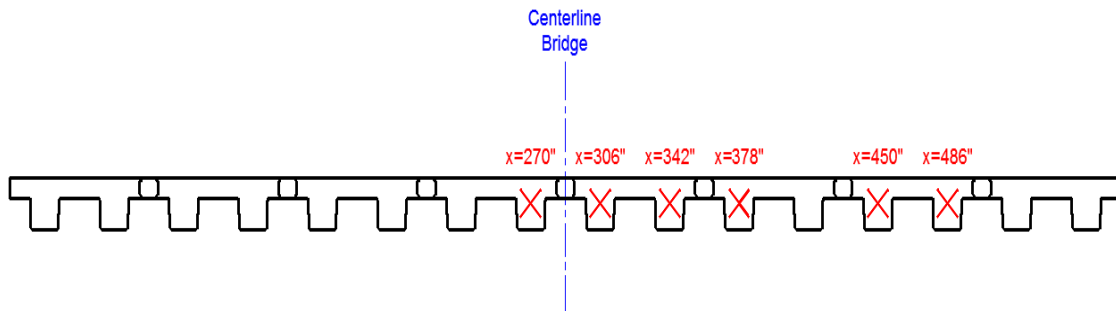


Figure 5.5: Maximum Negative Moment Response Locations for NEXT D6

Once the plot curvature was justified, the relationships were analyzed to determine the impact of support stiffness on the design positive and negative moment and shear in the slab. The 40-ft. span stiffness of 28.77 k/in calculated using classical beam theory was identified on the plots (labeled as 40' span) and the corresponding forces in the slab were recorded. In addition to this value the stiffness was calculated for other common span lengths using Equation 5.3 and these were all marked on the plots as well to illustrate the influence of span length on the web stiffness and thus the design forces in the slab. All of these response values were compared to the slab design forces calculated by MATLAB for the NEXT D8 system due to infinitely rigid supports, which are given below in Table 5.1 and can be found in Figure 5.6 as well.

Table 5.1: Slab Forces due to Infinitely Rigid Supports

Response Quantity	Value
Support Stiffness (k/in)	1×10^{15}
Positive M_{\max} (k-in)	172.39
Negative M_{\max} (k-in)	155.84
V_{\max} (k)	28.59

The stiffness values associated with 20-ft., 40-ft., 80-ft., and 150-ft. spans and the corresponding maximum responses for the NEXT D8 system are shown in Table 5.2.

Table 5.2: Influence of Span Length on NEXT D8 Responses

Span Length (ft)	Response Quantity			
	20	40	80	150
Stiffness (k/in)	260.44	28.77	3.48	0.52
+ M_{\max} (k-in)	289	501	844	1414
- M_{\max} (k-in)	86	148	331	705
V_{\max} (k)	24.5	21.2	18.9	18.2

The response values shown in Table 5.2 were compared to the values in Table 5.1 and the ratios are given below in Table 5.3.

Table 5.3: Ratio of NEXT D8 Response Value to Infinitely Rigid Responses

Span Length (ft)	Response Quantity			
	20	40	80	150
+ M_{\max} (k-in)	1.68	2.90	4.89	8.20
- M_{\max} (k-in)	0.55	0.95	2.12	4.52
V_{\max} (k)	0.86	0.74	0.66	0.63

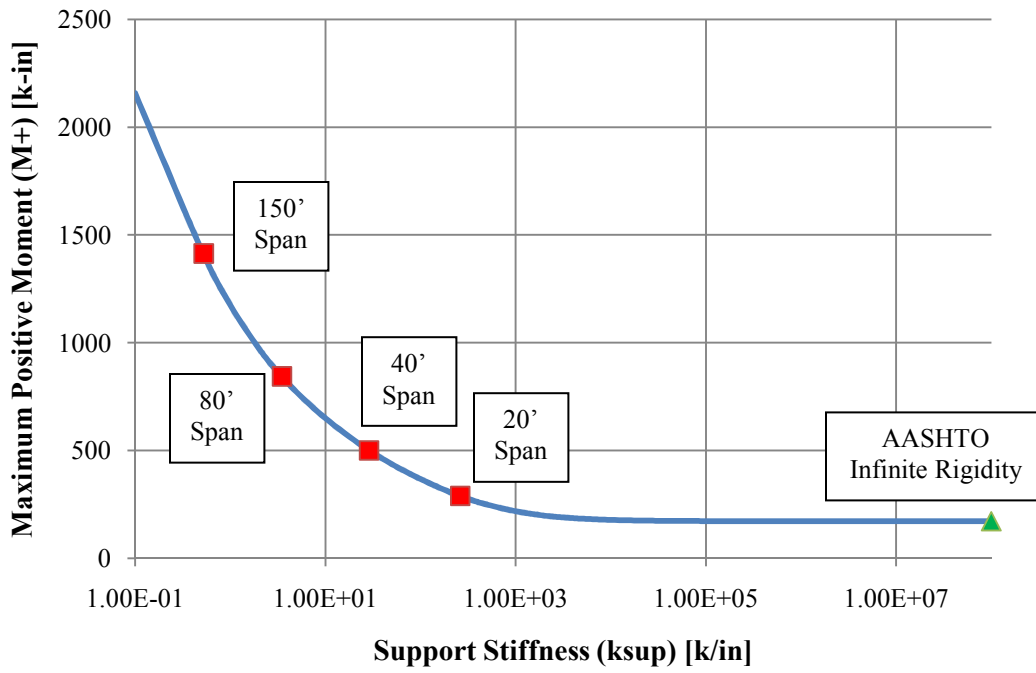
This process was repeated for the NEXT D6, and the ratios are given below in Table 5.4.

Table 5.4: Ratio of NEXT D6 Response Values to Infinitely Rigid Responses

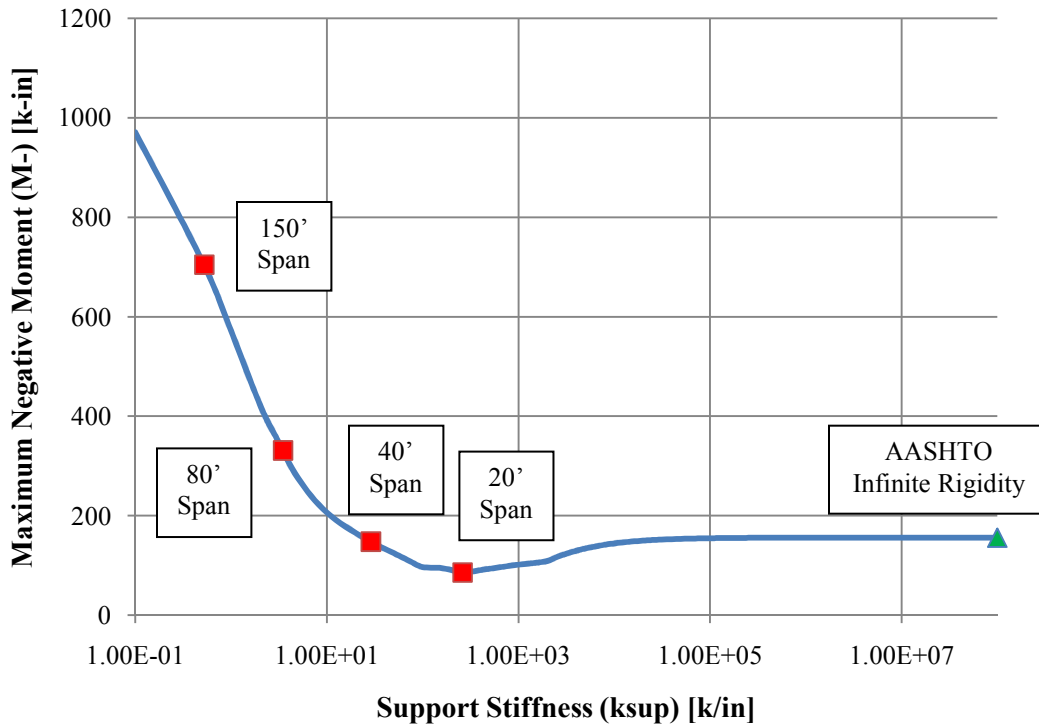
Span Length (ft)	Response Quantity			
	20	40	80	150
+M _{max} (k-in)	1.99	3.70	6.38	11.12
-M _{max} (k-in)	0.90	1.54	4.47	6.56
V _{max} (k)	0.77	0.67	0.62	0.60

All of the ratios for the positive moment response and some for the negative moment response indicate that the responses due to the calculated support stiffness are larger than the responses due to infinitely rigid supports. Therefore, the AASHTO assumption appears to not be a conservative method for determining the design moments for the NEXT D slab. In addition, as the span length increases, the design moments also increase, thus the ratio between the values and the response due to infinitely rigid supports increases (see Tables 5.2 and 5.3). Therefore, for longer NEXT D spans, the assumption of infinitely rigid supports produces even less conservative design moments.

The complete design force responses for positive and negative moment and shear for the NEXT D8 slab due to varying support stiffness are shown in Figure 5.6.

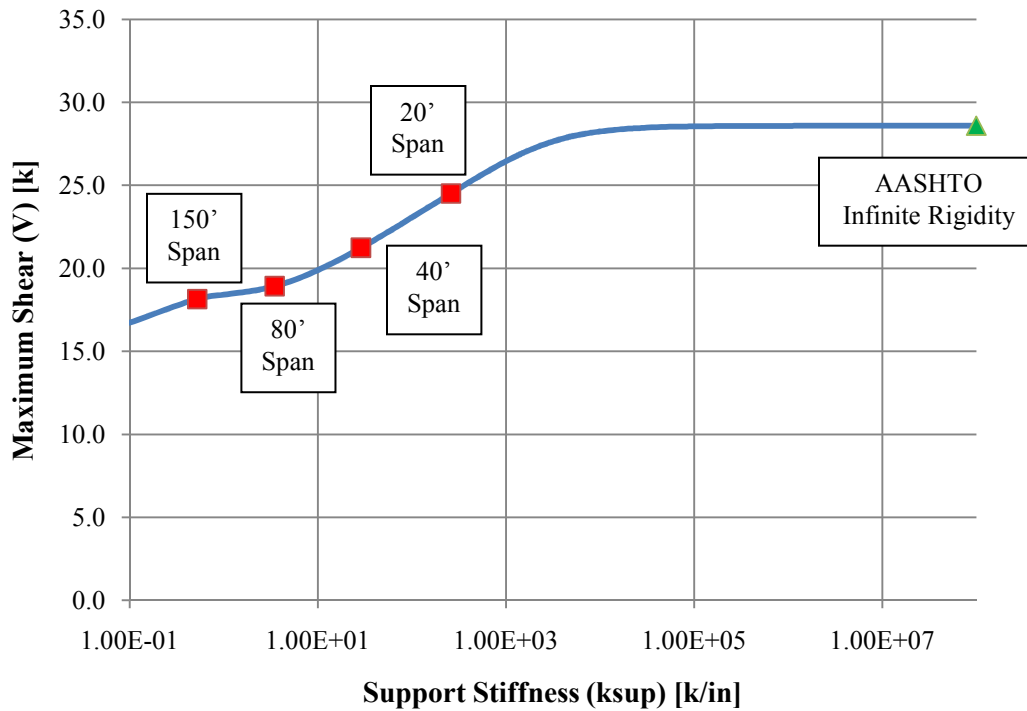


(a) Maximum Positive Moment



(b) Maximum Negative Moment

Figure 5.6: NEXT D8 – Influence of Support Stiffness on Slab Design Forces



(c) Maximum Shear

Figure 5.6 (Continued)

These relationships illustrate that increased support stiffness actually produces smaller positive and negative moments in the slab; therefore the AASHTO assumption of infinite rigidity would certainly produce design forces that are not conservative for moment. For shear, however, the AASHTO assumption would be conservative, although not by a large factor for short spans, as evidenced in Tables 5.3 and 5.4.

All of these values were calculated assuming infinite rigidity in the shear key, since in the sensitivity study as one parameter was varied all other parameters were held constant at 1×10^{15} k/in. Therefore, the ratios shown in Tables 5.3 and 5.4 and the relationships in Figure 5.6 assume the shear key is infinitely rigid. Since the behavior of the shear key is unknown, more simulations were run for a range of shear key

translational and rotational stiffness values. In each of these simulations, the response values based on the beam stiffness, 28.77 k/in, were compared to the response values based on infinite support stiffness. Sixty shear key stiffness values, varying from 0.01 k/in to 1×10^{15} k/in (translational) and 0.01 k-in/rad to 1×10^{15} k-in/rad (rotational), were considered and the average response ratios for the NEXT D6 and NEXT D8 are shown in Table 5.5 below.

Table 5.5: Ratio of NEXT D Response Values to Infinitely Rigid Responses

Response	NEXT D6	NEXT D8	Average
$+M_{\max}$ (k-in)	2.68	2.34	2.51
$-M_{\max}$ (k-in)	1.18	0.87	1.02
V_{\max} (k)	0.77	0.77	0.77

The positive moment and negative moment ratios indicate that the AASHTO assumption of infinitely rigid supports for a vast range of shear key stiffnesses is conservative for shear but non-conservative for both positive and negative moments.

The discrepancy in the AASHTO equivalent strip method and the values obtained through the sensitivity study was thought to be a result of the stocky geometry of the proposed NEXT D section. The AASHTO assumption of infinite support stiffness should be conservative in cases where the relative stiffness of the deck to the girders is very small, thus the girders can be assumed to be infinitely rigid. In a case of AASHTO girders spaced eight feet on center, for example, this assumption should be reasonable, since the slab would be much more flexible and the girders would be much stiffer since they have a higher moment of inertia. An AASHTO Type II girder, which has a maximum span of 70 ft., for example, has a moment of inertia of $50,980 \text{ in}^4$, which

compared to $14,439 \text{ in}^4$ for the NEXT D web is quite large (AASHTO 2007). In this case, however, the eight-inch slab only spans 36 in. between webs and the webs have a low moment of inertia compared to traditional AASHTO girders, thus the assumption does not accurately describe the situation and should not be used for the proposed NEXT D geometry.

The validity of this explanation was examined further by running a simulation with 96-in. web spacing throughout the bridge. Therefore, the model was essentially an eight-inch slab with a 96-in. girder spacing where the girder geometry was taken as the proposed NEXT D webs. For a 40-ft. span, the ratio of positive moment found in the simulations compared to the value determined using the AASHTO assumptions was 1.52, which is much less than the positive moment ratios shown in Table 5.5 for NEXT D8 and NEXT D6 sections. In addition, both the negative moment and shear based on classical beam theory stiffness calculations were less than the values produced by assuming infinite stiffness. Therefore, the AASHTO assumption was conservative for negative moment and shear and was much closer for positive moment. The positive moment should also be found to be conservative when using the AASHTO assumption for this scenario if a more traditional girder with a larger moment of inertia was used instead of the NEXT D web since the AASHTO girders are much stiffer.

In the slab design covered in Chapter 4, the AASHTO equivalent strip method was used to calculate the design forces which assumed infinite support stiffness (AASHTO 2007). Therefore, based on the results from the sensitivity study, the design should be re-examined and corrected to account for the discrepancy in the AASHTO

design forces and the design forces calculated through the sensitivity study based on the support stiffness determined through classical beam theory. The AASHTO design positive moment and negative moment were scaled up by factors of 2.51 and 1.02, respectively (see Table 5.5). The design shear was held constant since the AASHTO assumption was found to be conservative shear. The revised reinforcement for the slab is shown in Tables 5.6 and 5.7.

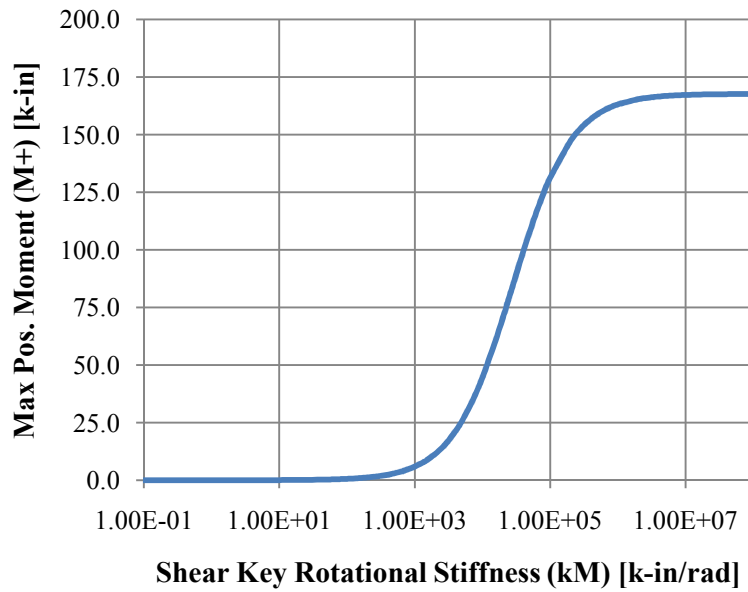
Table 5.6: Revised Slab Transverse Reinforcement

Location	NEXT D (Revised)	NEXT D (AASHTO method)
Bottom of Slab	#6 bars at 6" (0.88 in ² per ft)	#5 bars at 11" (0.34 in ² per ft)
Top of Slab	#5 bars at 12" (0.31 in ² per ft)	#5 bars at 12" (0.31 in ² per ft)
Top of Slab (overhang)	#5 bars at 6" (0.62 in ² per ft)	#5 bars at 6" (0.62 in ² per ft)

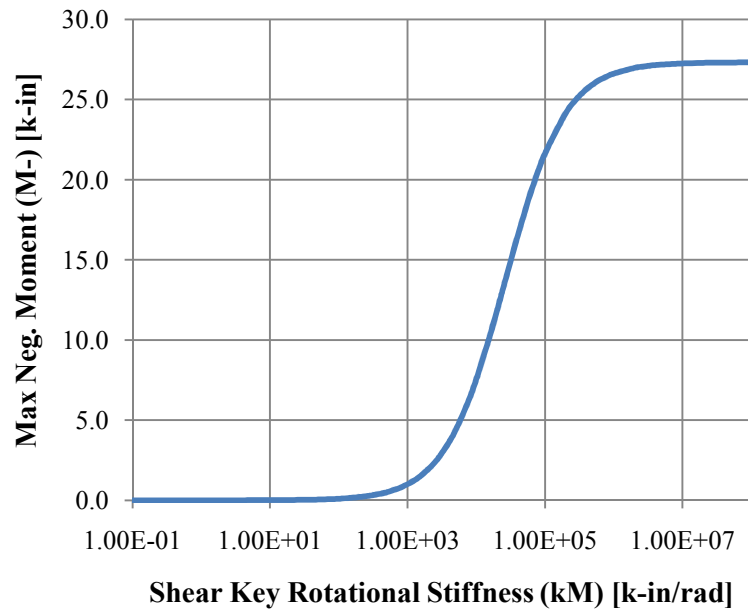
Table 5.7: Revised Slab Longitudinal Reinforcement

Location	NEXT D (Revised)	NEXT D (AASHTO method)
Bottom of Slab	#5 bars at 6" (0.62 in ² per ft)	#4 bars at 10" (0.24 in ² per ft)
Top of Slab	#4 bars at 18" (0.13 in ² per ft)	#4 bars at 18" (0.13 in ² per ft)

The behavior of the shear key in the NEXT D system was also investigated in this sensitivity study by varying the translational and rotational stiffness of the keyway and studying the resulting force effects in the key. The plots for the NEXT D8 section shear key responses are shown below in Figure 5.7.

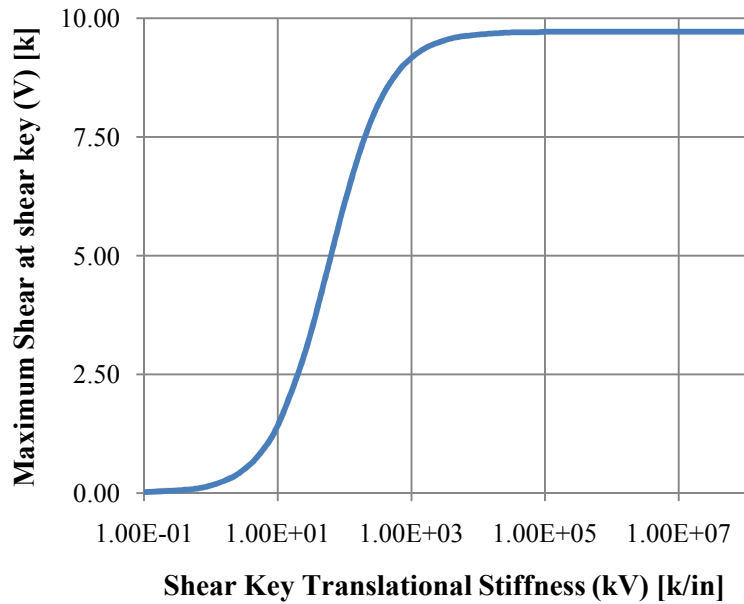


(a) Maximum Positive Moment in Shear Key



(b) Maximum Negative Moment in Shear Key

Figure 5.7: Influence of Shear Key Stiffness on Design Forces in Shear Key



(c) Maximum Shear in Shear Key

Figure 5.7 (Continued)

As expected, increased rotational stiffness in the shear key increased the maximum positive and negative values in the keyway and increased translational stiffness in the shear key increased the maximum shear in the key. At this stage, it is conservative to assume the force effects in the keyway are based on infinitely rigid keyways; however the appropriate design forces in the key will be verified through future numerical and experimental studies.

The AASHTO design positive moment for the shear key was scaled up by a factor of 2.51 based on the results of the sensitivity study. In order to account for this increased design moment, the headed bar reinforcement determined for the shear key in Chapter 4 was increased to #7 bars at six inches on-center. In addition, the depth of the headed

reinforcing bars was shifted down one-half inch to a total depth of five inches below the top of the shear key. This was done to increase the positive moment capacity of the shear key.

The change in the location of the headed reinforcing bars in the shear keys was checked based on the results of the sensitivity study. The ratio of the positive and negative moments produced in the shear keys throughout all of the simulations was calculated in order to determine the most advantageous depth for the headed reinforcing bars in the keyway to provide adequate positive and negative moment capacity. The original NEXT D details provide the headed reinforcing bars at a depth of four and one-half inches in the eight inch deep key. The results from the sensitivity study found that the maximum positive moment was on-average, approximately six times the maximum negative moment in the shear key for the NEXT D8 (see Figure 5.7) and on-average, approximately 2.3 times the maximum negative moment in the shear key for the NEXT D6. Therefore, shifting the position of the headed reinforcing bars down one-half inch to five inches below the top of the key in order to provide a larger eccentricity and thus higher capacity for the larger positive moments in the section is valid. Further numerical and experimental studies should be performed to determine more accurate design forces in the slab based on the anticipated shear key stiffness in order to provide an adequate amount of reinforcing in the shear key for both positive and negative moment.

The typical section reinforcement, revised based on the results of the sensitivity study, is shown in Figure 5.8.

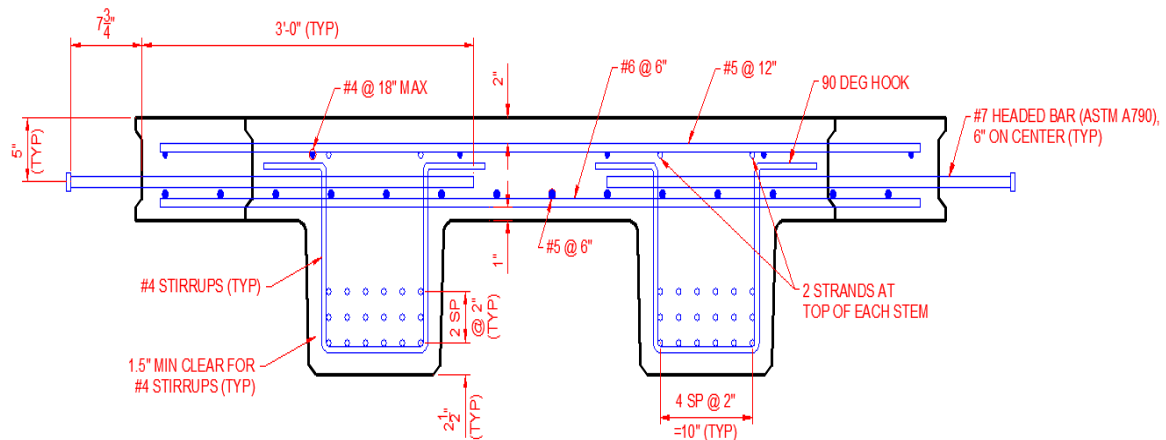


Figure 5.8: Revised NEXT D Beam Typical Reinforcement

Conclusions and Closure

The following was concluded from the deck sensitivity study:

- The AASHTO equivalent strip method generates NEXT D slab design forces that are conservative for shear and non-conservative for positive and negative moments.
- Increased span length decreases the calculated stiffness of the supporting components, thus causing design forces further deviated from those generated using the equivalent strip method.
- Positive moment determined by calculating the support stiffness using classical beam theory can be up to three times the value calculated by AASHTO for a 40-ft. span NEXT D. The slab design forces should not be based on AASHTO's equivalent strip method for the NEXT D section for this reason.
- Increased stiffness in the shear key produces higher design forces in the shear key; further research should study the behavior of the shear key and determine reasonable stiffness parameters for the connection.
- When designed as a moment connection, the depth of the headed reinforcing bars in the shear key must be optimized in order to provide adequate positive and negative moment capacity. In this case, #7 bars at six inches on-center located 5 in. from the top of the shear key were found to be adequate.

CHAPTER SIX

CONCLUSIONS

This research covered the completed studies and current practices used to provide solutions for short span, rapid construction bridge projects. A flat slab alternative section was sought that the SCDOT could use on short span, rapid construction projects for all routes, regardless of annual daily traffic restrictions, to minimize longitudinal and transverse cracking. After carefully considering all known alternatives identified through literature reviews and designer, contractor, and fabricator interviews nationwide, the SCDOT steering committee selected the NEXT D beam as the alternative system for flat slab bridges in South Carolina.

The NEXT D section designed by the developers in the Northeast for longer spans was scaled down to produce an economical section for the target span of 22 to 40 ft. for this research. The revised section geometry included a total depth of 20 inches, a web spacing of 36 in., and a total beam width adjustable from 64 in. to 88 in. wide with an eight-inch wide shear key between sections to create a six-foot (NEXT D6) and an eight-foot (NEXT D8) beam spacing, respectively. The CONSPAN (Bentley Systems 2010) preliminary prestressed design of the system required 28 ½"-diameter, 270-ksi low-relaxation prestressing strands per section for the NEXT D6 and 40 of these strands per section for the NEXT D8 (see Figure 5.8), which was comparable to the total amount of prestressing used in the current SCDOT flat slab bridges.

The transverse and longitudinal reinforcement for the slab was determined using the AASHTO equivalent strip method assuming the supporting components were

infinitely rigid (AASHTO 2007). The AASHTO equivalent strip method was investigated for appropriateness when applied to the NEXT D details and was found to be conservative for determining the design shear, but non-conservative for determining the design moments in the slab. The equivalent strip method's assumption of infinitely rigid supports generated lower moments in the slab than supports with stiffness calculated through classical beam theory (Table 5.5). The NEXT D slab reinforcement was refined to account for the necessary increase in design moments (Tables 5.6 and 5.7 and Figure 5.8). The AASHTO equivalent strip method should only be used in cases where the supports can be considered infinitely stiff in comparison to the slab; otherwise using the approach will result in non-conservative design forces. In addition, it was found that increased span length causes design forces further deviated from those generated using the equivalent strip method. For these reasons, the slab design forces for the NEXT D section should not be determined using the equivalent strip method, but instead should be based on the web stiffness calculated based on classical beam theory, using Equations 5.1-5.3 to determine the web stiffness for the system.

It was found that increased translational and rotational stiffness in the shear key increases the maximum shear and moment, respectively, in the keyway. Also, the shear key was assumed to act as a moment connection and was designed to resist both positive and negative moment in the keyway. Therefore, the depth of the headed reinforcing bars in the key must be optimized in provide to provide adequate positive and negative moment capacity. It was determined that #7 bars at six inches on-center located five inches from the top of the shear key is sufficient to resist positive and negative moment.

Recommendations for Future Work

There are many improvements which could be made to the sensitivity study in this thesis. In the study, as one stiffness was varied each additional stiffness was held constant at an assumed infinitely rigid value of 1×10^{15} k/in (translational stiffness) or 1×10^{15} k-in/rad (rotational stiffness). Further study on the impact of a vast range of stiffness values, including setting the control parameters to values other than infinite rigidity, would be useful in further exploring the behavior of the NEXT D slab. Also, following in-depth shear key study, more targeted ranges of stiffness values could be implemented into the sensitivity study software to determine more accurate slab design forces. The effects of multiple HL-93 truck loads could be considered as well.

In addition to the design issues covered in this thesis, the NEXT developers identified many others issues that will require some study before the NEXT D section can be used confidently on high volume bridges. One of those involves the appropriate live load distribution factor for the section. Currently, the NEXT developers have decided to use the AASHTO Section I equations to determine the live load distribution factor, however shear key behavior verified through finite element modeling or experimental testing will validate this assumption. A separate study at Clemson University is currently using finite element analysis to further understand the behavior of the shear key and the impact of this behavior on the performance of the bridge. Other design issues requiring attention include: barrier detail, approach slab, crown, vertical curves, and skew. These concepts must be considered and more research performed before the NEXT D section can be implemented confidently as the flat slab alternative in South Carolina.

REFERENCES

- AASHTO. (2007). *AASHTO LRFD Bridge Design Specification*, 4th Ed., American Association of State Highway and Transportation Officials, Washington, D.C.
- ACI. (2009). *ACI 550.1R-09: Guide to Emulating Cast-in-Place Detailing for Seismic Design of Precast Concrete Structures*, American Concrete Institute, Farmington Hills, MI.
- American Crane Corporation. (2001). "American HC 80 Hydraulic Crawler Crane."
- Barker, R., and Puckett, J. (2007). *Design of Highway Bridges: An LRFD Approach*, 2nd ed. Ed., John Wiley & Sons, Inc., Hoboken, NJ.
- Baur, K., Culmo, M., Dillman, B., Lorah, D., Medlock, R. (2010). "The NEXT beam: A robust double-tee prestressed beam." Federal Highway Administration.
- Bentley Systems, I. (2010). "LEAP CONSPAN V8i." (09.00.02.01).
- Bergeron, K. A. (2008). "Leap, Not Creep: Delivery and Deployment of Vanguard Technologies Promise to Lead the Way to Faster, Safer, Better Highway Construction." *Public Roads*, 71(4), NA.
- Big R Bridge. "Vehicular bridges for accelerated construction | big R bridge." <<http://www.bigrmfg.com/products/modular/>> (5/20/2010, 2010).
- Cangiano, S., Meda, A., Plizzari, G. A. (2009). "Rapid Hardening Concrete for the Construction of a Small Span Bridge." *Construction & Building Materials*, 23(3), 1329-37.
- Cho, A. (2007). "Utah Embraces Accelerated Construction Method.(Ralph L. Wadsworth Construction Company Inc. Given Contract for Bridge Construction)." *ENR*, 259(16), 15(1).
- Christian, G., and Deery, D. (2010). "Interview with George Christian at the New York DOT."
- Chung, P., and Deery, D. (2010). "Interview with Paul Chung at the California DOT."
- Computers and Structures, Inc. "SAP2000 Advanced." (14.1.0), Berkeley, CA.
- Culmo, M. P., Seraderian, R. L., Nielson, B. G., Schiff, S., Pang, W. (2010). "NEXT Beam Developer Phone Interview."

- Culmo, M. P. (2009). *Connection Details for Prefabricated Bridge Elements and Systems*, Federal Highway Administration, Washington, D.C.
- Culmo, M. P., and Seraderian, R. L. "Development of the northeast extreme tee (NEXT) beam for accelerated bridge construction." *Proc., 2008 Concrete Bridge Conference*.
- Domagalski, T., and Roberts, S. (2010). "Interview with Thomas Domagalski at the Illinois DOT."
- El-Remaily, A., Tadros, M. K., Yamane, T., Krause, G. (1996). "Transverse Design of Adjacent Precast Prestressed Concrete Box Girder Bridges." *PCI J.*, 41(4), 96-113.
- ERICO. "LENTON INTERLOK." <<http://www.erico.com/products/InterLock.asp>> (5/5/2010, 2010).
- FHWA. (2006). *Prefabricated Steel Bridge Systems: Final Report*.
- FHWA, and Michael Baker Jr., I. (2003). *LRFD Design Example for Steel Girder Superstructure Bridge*.
- Fisher, G., and Deery, D. (2010). "Interview with Gary Fisher of Flexicore (Texas Fabricator)."
- Frake, T., Beck, S., Roberts, S. (2010). "Interview with Terry Frake and Steve Beck at the Michigan DOT."
- Fu, C. C., and Jeong, S. (2009). *Behavior and Analysis of an Instrumented Slab Bridge*, Masters Ed., University of Maryland, College Park, Maryland.
- Geddis, T., and Deery, D. (2010). "Conversation with Ted Geddis."
- Grace, N. F., and Jensen, E. (2008). *Use of Unbonded CFCC for Transverse Post-Tensioning of Side-by-Side Box-Beam Bridges*, Michigan Department of Transportation, Lansing, Michigan.
- Hartle, R. A. (2009). "I-70 Overpass Beam Failure - Lakeview Drive Bridge - Ohio Bridge Conference."
- Heston, B., and Deery, D. (2010). "Interview with Bill Heston of Balfour Beatty (North Carolina Contractor)."
- Holt, J., and Deery, D. (2010). "Interview with John Holt at the Texas DOT."

- Jansson, P. O. (2008). *Evaluation of Grout-Filled Mechanical Splices for Precast Concrete Construction*, Michigan Department of Transportation, Lansing, MI.
- Jenkins, T., and Deery, D. (2010). "Interview with Troy Jenkins of Northeast Precast Products (New York Fabricator)."
- Kamel, M. R., and Derrick, D. (1997). "Precast Alternative for Bridge Slabs." *Concr. Int.*, 19(8), 41-42.
- Kapur, J., and Deery, D. (2010). "Interview with Jugesh Kapur at the Washington DOT."
- Keller, T., and Roberts, S. (2010). "Interview with Tim Keller at the Ohio DOT."
- Konda, T. F., Klaiber, F. W., Wipf, T. J. (2003). "Beam-in-slab low-volume-road bridge system." *Proc., Eight International Conference on Low-Volume Roads 2003, June 22, 2003 - June 25*, National Research Council, 405-412.
- Lall, J., Alampalli, S., DiCocco, E. F. (1998). "Performance of Full-Depth Shear Keys in Adjacent Prestressed Box Beam Bridges." *PCI J.*, 43(2), 72-79.
- Li, L., Ma, Z., Oesterle, R. G. (2010). "Improved Longitudinal Joint Details in Decked Bulb Tees for Accelerated Bridge Construction: Fatigue Evaluation." *J. Bridge Eng.*
- Liles, P., and Deery, D. (2010). "Interview with Paul Liles at the Georgia DOT."
- Losee, M., and Deery, D. (2010). "Interview with Mark Losee of Jefferson Concrete (New York Fabricator)."
- Miller, R. A., Hlavacs, G. M., Long, T., Greuel, A. (1999). "Full-Scale Testing of Shear Keys for Adjacent Box Girder Bridges." *PCI J.*, 44(6), 80-90.
- MoDOT. (2010). "MoDOT Engineering Policy Guide."
- Naito, C. J. (2010). "Inspection Methods & Techniques to Determine Non Visible Corrosion of Prestressing Strands in Concrete Bridge Components."
- Nawy, E. G. (2009). *Prestressed Concrete: A Fundamental Approach*, 5th ed. Ed., Prentice-Hall, Inc., Upper Saddle River, NJ.
- Nielson, B. G., Schiff, S., Pang, W., Deery, D., Flores Durón, A., Roberts, S. (2010). "South Carolina Department of Transportation Research Project No. 682: Precast Alternative for Flat Slab Interim Report."

Nims, D. K. (2010). "UTUTC-IU-12: Magnetic Sensor for Nondestructive Evaluation of Deteriorated Prestressing Strand."

NYSDOT. (2010). "NYSDOT Design Manual - Chapter 3 - Deck Systems."

NYSDOT. (2010). "New York State Department of Transportation Bridge Manual."

NYSDOT. (2010). "Bridge detail sheets." <<https://www.nysdot.gov/main/business-center/engineering/cadd-info/drawings/bridge-detail-sheets-usc>> (10/25/2010).

Oesterle, R. G., and Elremaily, A. F. (2009). *Design and Construction Guidelines for Long-Span Decked Precast, Prestressed Concrete Girder Bridges*, National Cooperative Highway Research Program.

Ohio DOT. "Standard bridge drawings."
<<http://www.dot.state.oh.us/Divisions/HighwayOps/Structures/standard/Bridges/Pages/StandardBridgeDrawings.aspx>> (5/5/2010, 2010).

Parimuha, J., and Deery, D. (2010). "Interview with JR Parimuha of Florence Concrete Products (North Carolina Fabricator)."

Patel, S., and Roberts, S. (2010). "Interview with Suresh Patel at the Missouri DOT."

PCI Northeast. "PCI northeast: Northeast extreme tee (NEXT) beam."
<http://pcine.org/index.cfm/resources/bridge/Northeast_Extreme_Tee_Beam> (06/2010, 2010).

Perfetti, G., and Roberts, S. (2010). "Interview with Greg Perfetti at the North Carolina DOT."

Prussack, C., and Deery, D. (2010). "Interview with Chuck Prussack of Central-Premix (Washington Fabricator)."

Ralls, M. L., and Tang, B. M. (2003). "Laying the Groundwork." *Public Roads*, 67(3), 8-11.

Roberts, S. E. (2010). "Influence of Shear Key Performance on the Fatigue Life of Adjacent Beam Bridges."

Roscoe Bridge. "County vehicular bridges — roscoe bridge — vehicle, trail, and pedestrian bridges." <<http://www.roscoebridge.com/?a=12&b=county-vehicular-bridges>> (5/5/2010, 2010).

- Russell, H. G. (2009). *Adjacent Precast Concrete Box Beam Bridges: Connection Details*, Transportation Research Board, Washington, D.C.
- Saadeghvaziri, M. A., Yin, L., Spillers, W. R. (2006). *Development of High Performance Continuity Connection using CFRP Composites*, Federal Highway Administration, Washington, D.C.
- SCDOT. (2010). "Doing business with SCDOT - bridge design drawings and details." <<http://www.scdot.org/doing/bridge/Bridgesdmenu1.asp?DIV=704&spec=y>> (5/27, 2010).
- SCDOT. (2009). "Suck Creek Bridge Plans - Cherokee County." *Received from South Carolina Department of Transportation November 2009.*
- SCDOT. (2006). "South Carolina Department of Transportation Bridge Design Manual - LRFD."
- Scott. (2006). "Adaption to presentation to AASHTO T-18 committee to inform other states of the SR 1014 over I-70 beam collapse."
- SurveyMonkey. (2010). "SurveyMonkey." <<http://www.surveymonkey.com>> .
- Tang, B., and Deery, D. (2010). "Interview with Benjamin Tang at the Oregon DOT."
- Tesch, S., and Deery, D. (2010). "Interview with Sandy Tesch of ConStar Construction (Texas Contractor)."
- The MathWorks, I. (2009). "MATLAB." (2009b), Natick, MA.
- TxDOT. (2010). "Bridge standards (English)." <<http://www.txdot.gov/insdot/orgchart/cmd/cserve/standard/bridge-e.htm>> (5/5, 2010).
- TXDOT. (2009). "Texas Department of Transportation Bridge Design Manual - LRFD."
- University of California Berkeley. (2009). "OpenSees." (2.1.0), Berkeley, CA.
- Volgyi, J., and Roberts, S. (2010). "Interview with Julius Volgyi at the Virginia DOT."
- WSDOT. "Bridge standard drawings - table of contents." <<http://www.wsdot.wa.gov/eesc/bridge/drawings/>> (5/11/2010, 2010).

APPENDICES

Appendix A

Clemson University Web DOT Survey

Survey of Adjacent Beam Bridge Design and Construction Practices

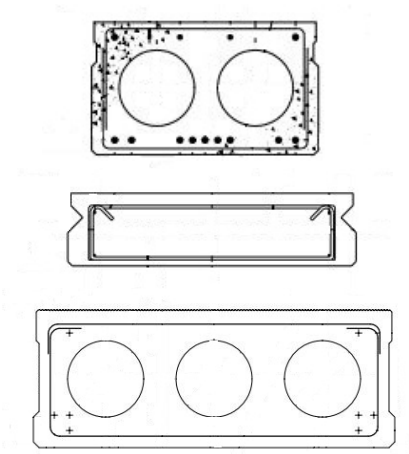
We are researching improved methods of accelerated bridge construction for short span bridges for the South Carolina Department of Transportation. Our goal in utilizing this survey is to gather construction and performance information about precast adjacent beam bridges. We aim to minimize cracking along the longitudinal joints of the bridge and create continuity details over interior bents. The survey will inquire about the design and erection of your adjacent beam members and the experienced performance of these bridges. By gathering this information from other DOTs, we hope to produce a standard with improved shear key and continuity performance that may be used on higher ADT roads.

In return for helping us gather information on these systems, we will send you a summary report of our findings from the survey.

What State are you representing? _____

A. Low Profile Adjacent Beam (LPAB) Bridges: Voided Slab/Hollow Core/Deck Beams/Solid Slab (sections and pictures shown below)

LPAB Sections:



Hollow Core:



*** If you do not use low profile adjacent beam (LPAB) bridges, please skip to part B.*

- General:

- How long have you been using LPAB bridges?
 - Past 2 years
 - Past 5 years
 - Past 10 years
 - Past 20 years
 - Past 50 years
- About how many LPAB bridges have you built in the past 10 years?
 - 5 or less
 - 6 to 10
 - 11 to 20
 - 21 to 50
 - More than 50
- What is the maximum span of your LPAB bridges?
 - 20 feet or less
 - 21 to 25 feet
 - 26 to 30 feet
 - 31 to 40 feet
 - More than 40 feet
- Are the LPAB bridge details available on your website the most current plans?
 - Yes No
 - Website: _____
- Do you limit the use of LPAB bridges to a particular AADT?
 - Yes
 - What is the maximum AADT for use?
 - Less than or equal to 1500
 - Less than or equal to 3000
 - Less than or equal to 5000
 - Less than or equal to 10,000
 - More than 10,000
 - No
- Do you permit using LPAB bridges on the National Highway System?
 - Yes No
- Have you had any recent major changes to the standards for this bridge type?
 - Yes No
 - What were the major design/construction changes?

 - Has there been noticeable improvement in performance after the changes were implemented?
 - Yes No Too early to tell

- Construction:
 - What is the average time needed to erect one span of a LPAB bridge?
 - Less than 1 week
 - 1 to 2 weeks
 - 2 to 3 weeks
 - 3 to 4 weeks
 - More than 4 weeks
 - What workforce constructs these bridges?
 - In house
 - Contractor
 - Both

- Post-Tensioning:
 - When do you apply the post-tensioning force to the bridge?
 - After grouting the shear keys
 - Before grouting the shear keys
 - Contractor's Preference
 - What post-tensioning material do you use?
 - Strands Rods Contractor's Preference
 - Do you have a target contact stress for post-tensioning?
 - Yes: _____ kips/ft² No

- Grouting/Shear Key:
 - What depth are the shear keys?
 - Partial Depth Full Depth
 - Where are the shear keys located?
 - Near the top face of the member
 - At the center of gravity of the member
 - What type of grout is used in the longitudinal shear keys?
 - Non-shrink Epoxy Cast-in-place concrete
 - Other: _____
 - Do you require a concrete overlay on the LPAB bridge members?
 - Yes
 - Is the overlay reinforced?
 - Yes No
 - No
 - Is an asphalt overlay required?
 - Yes No
 - Do you provide waterproofing?
 - Yes No
 - Have you tried placing mild reinforcing steel transversely through the shear key?
 - Yes No

- Do you use any other method of shear transfer (other than mild reinforcing or shear key)?
 - Yes: _____
 - No
- About what percentage of these bridges experience longitudinal reflective cracking along the shear keys?
 - 0 to 20%
 - 21 to 40%
 - 41 to 60%
 - 61 to 80%
 - 81 to 100%
- Do these cracks occur more in bridges with an AADT over 3000?
 - Yes □ No
- On the scale below, identify how concerned you are about these cracks distressing the bridge.
 (Not concerned) 1 2 3 4 5 6 7 8 9 10 (Very concerned)
- Longitudinal Continuity:
 - Do you ever make your multi-span LPAB bridges longitudinally continuous?
 - Yes □ No
 - Do you account for positive restraint moments when designing continuity diaphragms?
 - Yes □ No
 - If yes, what is the average girder age when continuity is established?
 - 7 days or less
 - 8 to 24 days
 - 25 to 90 days
 - Greater than 90 days
 - Not Considered
- Alternative:
 - Do you have an alternative system for this bridge type that is considered rapid construction?
 - Yes: _____
 - No
 - Are there any other alternative systems that you are interested in?
 - Yes: _____
 - No

B. High Profile Adjacent Beam (HPAB) Bridges: Box Beams

HPAB Section:



**** If you do not use high profile adjacent beam (HPAB) bridges, please skip to part C.**

- General:

- How long have you been using HPAB bridges?
 - Past 2 years
 - Past 5 years
 - Past 10 years
 - Past 20 years
 - Past 50 years
- About how many HPAB bridges have you built in the past 10 years?
 - 5 or less
 - 6 to 10
 - 11 to 20
 - 21 to 50
 - More than 50
- What is the maximum span of your HPAB bridges?
 - 20 feet or less
 - 21 to 25 feet
 - 26 to 30 feet
 - 31 to 40 feet
 - More than 40 feet
- Are the HPAB bridge details available on your website the most current plans?
 - Yes No
 - Website: _____
- Do you limit the use of HPAB bridges to a particular AADT?
 - Yes
 - What is the maximum AADT for use?
 - Less than or equal to 1500
 - Less than or equal to 3000
 - Less than or equal to 5000
 - Less than or equal to 10,000
 - More than 10,000
 - No

- Do you permit using HPAB bridges on the National Highway System?
 - Yes No
- Have you had any recent major changes to the standards for this bridge type?
 - Yes No
 - What were the major design/construction changes?

 - Has there been noticeable improvement in performance after the changes were implemented?
 - Yes No Too early to tell
- Construction:
 - What is the average time needed to erect one span of a HPAB bridge?
 - Less than 1 week
 - 1 to 2 weeks
 - 2 to 3 weeks
 - 3 to 4 weeks
 - More than 4 weeks
 - What workforce constructs these bridges?
 - In house
 - Contractor
 - Both
- Post-Tensioning:
 - When do you apply the post-tensioning force to the bridge?
 - After grouting the shear keys
 - Before grouting the shear keys
 - Contractor's Preference
 - What post-tensioning material do you use?
 - Strands Rods Contractor's Preference
 - Do you have a target contact stress for post-tensioning?
 - Yes: _____ kips/ft² No
- Grouting/Shear Key:
 - What depth are the shear keys?
 - Partial Depth Full Depth
 - Where are the shear keys located?
 - Near the top face of the member
 - At the center of gravity of the member
 - What type of grout is used in the longitudinal shear keys?
 - Non-shrink Epoxy Cast-in-place concrete
 - Other: _____

- Do you require a concrete overlay on the HPAB bridge members?
 - Yes
 - Is the overlay reinforced?
 - Yes No
 - No
 - Is an asphalt overlay required?
 - Yes No
 - Do you provide waterproofing?
 - Yes No
- Have you tried placing mild reinforcing steel transversely through the shear key?
 - Yes No
- Do you use any other method of shear transfer (other than mild reinforcing or shear key)?
 - Yes: _____
 - No
- About what percentage of these bridges experience longitudinal reflective cracking along the shear keys?
 - 0 to 20%
 - 21 to 40%
 - 41 to 60%
 - 61 to 80%
 - 81 to 100%
- Do these cracks occur more in bridges with an AADT over 3000?
 - Yes No
- On the scale below, identify how concerned you are about these cracks distressing the bridge.
 (Not concerned) 1 2 3 4 5 6 7 8 9 10 (Very concerned)

- Longitudinal Continuity:

- Do you ever make your multi-span HPAB bridges longitudinally continuous?
 - Yes No
 - Do you account for positive restraint moments when designing continuity diaphragms?
 - Yes No
 - If yes, what is the average girder age when continuity is established?
 - 7 days or less
 - 8 to 24 days
 - 25 to 90 days
 - Greater than 90 days
 - Not Considered

- Alternative:
 - Do you have an alternative system for this bridge type that is considered rapid construction?
 - Yes: _____
 - No
 - Are there any other alternative systems that you are interested in?
 - Yes: _____
 - No

C. Alternatives

*** If you skipped parts A & B (you do not use low or high profile adjacent beam bridges) please complete this section. Otherwise, please skip to Part D.*

- Do you have an alternative system for these bridge types that is considered rapid construction?
 - Yes: _____
 - No
- Are there any other alternative systems that you are interested in?
 - Yes: _____
 - No

D. Follow Up

- Information:
 - Name: _____
 - State: _____
 - Position: _____
 - Phone: _____
 - E-mail: _____
- Is it OK to call you for a follow-up conversation?
 - Yes No
- Would you like to have the results of this survey sent to you?
 - Yes No

Appendix B

DOT, Contractor and Fabricator Phone Interview Summaries

Phone Interview with Thomas Domagalski

Illinois Department of Transportation

By Sara Roberts, Clemson University

Date: Wednesday March 24th, 2010

- Made keyway wider and deeper so they could use a pencil vibrator to ensure distribution of the grout.
- Thickened the bottom slab of the member to add a half inch of cover for the strands.
- Didn't think they needed a post tensioning force, saw that many other states did not have one.
- Their concrete overlay would take about 4 to 7 days to cure.
- 5" overlay with #5 rebar mat at 12" centers in both directions
- Says Nebraska is experimenting with a very large shear key and post tensioning the top flange of the member.
- They believe the precast box beam bridges are fast enough construction for them and are not interested in self-propelled modular transports (SPMT).

Phone Interview with Julius Volgyi

Virginia Department of Transportation

By Sara Roberts, Clemson University

Date: Wednesday March 24th, 2010

- Does not like the box beams because of cracking and salt water building up in the voids.
- Believes Hollow Core performs better.
- Only 2 or 3 projects use concrete overlay.
- Curing an overlay would take up to 28 days.
- Target post tensioning stress is a handed down number, not sure where it came from.
- Longitudinal cracking has been a severe hindrance when choosing this type of bridge for construction.
- Full depth shear key has been in use for about 10 years, cannot tell yet if it is an improvement.
- Have never felt the need to make their hollow core bridges continuous.
- Does not know of any alternative systems he would like to try.

Phone Interview with Suresh Patel

Missouri Department of Transportation

By Sara Roberts, Clemson University

Date: Thursday March 25th, 2010

- Only use low profile adjacent beams when time is a very important factor. Otherwise the maintenance issue with salt water is not cost effective.
- The 1 – 2 week construction time is just for setting beams and grouting, not for concrete overlay curing.
- Do not post-tension. Only tighten rods enough to close the gap.
- Usually use 5 ½” concrete overlay, but may use asphalt on low AADT roads
- Continuity diaphragm: bend strands, place transverse rebar and make closure pour.
- Thinks making a continuity diaphragm may extend the project 1 or 2 weeks.
- Is not aware of cracking at continuity diaphragm but they use a lot of shear reinforcement at the beam ends when using a continuity diaphragm for bonding purposes.
- Does not know of any alternative systems.

Phone Interview with Tim Keller

Ohio Department of Transportation

By Sara Roberts, Clemson University

Date: Tuesday March 30th, 2010

- Usually use a three sided culvert or cast in place slab for 20 – 30 foot spans, instead of slab beams.
- Waterproofing membrane under asphalt has not been an effective water barrier.
- The leaking shear keys and deicing salts are a maintenance nightmare, so they don't use them at all on NHS and high AADT. They don't perform well there.
- Not currently specifying a post tensioning force, just tightening rods.
- Have started post tensioning a handful of bridges and are in the process of determining the best economical stress to specify. (currently thinking 90-100 psi is best).
- Curing of concrete overlay would take about 2 weeks more.
- They have a standard continuity diaphragm detail they use with all their box beam bridges. It was developed by Dr. Miller at the University of Cincinnati.
- Says diaphragm does not extend the time of construction very much when using a concrete overlay, but they don't like the diaphragm and the overlay being poured at the same time.

- Has experienced a lot of cracking at their continuity diaphragms. The design the bridge's live load capacity as simple span. Therefore, if the diaphragm cracks, the bridge will still have good capacity.
- Thinks the post tensioning change will really help the box beam's performance.
- Upset with having these type of bridges that have to be replaced every 25 years.
- Not interested in rapid bridge construction because of climate, more worried about blocking off high volume roads.

Phone Interview with Terry Frake and Steve Beck

Michigan Department of Transportation

By Sara Roberts, Clemson University

Date: Tuesday April 6th, 2010

- Maintenance forces are very against them because of old details that performed very badly.
- "Lots" build in the last 10 years.
- Usually use them when they have an under-clearance issue.
- No restriction on box beam bridge placement.
- Always use concrete overlay, average thickness of 6".
- Did not have post-tensioning force until lately after a research project
 - o Has not been adopted so he doesn't have the numbers
- Grout before post-tensioning
- Looking at increasing shear key depth because of research project.
- Worried about changing details because they will need more competent contractors.
- Only about 25% of box beams show longitudinal cracking
 - o They feel the advantages of the box beams outweigh the cracking problems.
- Make some bridges continuous for live load
 - o Boxes are simple and the deck overlay creates live load continuity
- Looked at alternative I beam sections that can mimic the box beams, but they are a little averse to steel because of the painting cost.
- Unfamiliar with the grout filled mechanical splices in practice.
- Starting many new research projects to look at improving old biased design ways.

Phone Interview with John Holt

Texas Department of Transportation
By Daniel Deery, Clemson University
Date: Monday March 29th, 2010

- Concrete and asphalt overlays are used on adjacent box beams.
- Erection time listed as less than a week does not include overlay curing.
- Target post-tension force has evolved over years and now it's one tendon every 5-10 feet at 31 kips initial tension, and it seems to be working.
- Robust concrete shear key used to transfer shear.
- Only use post-tensioning with asphalt but 99% of time a 5" concrete deck is used instead.
- Longitudinal cracks have not been an issue since they went to a 5" deck.
- Found long ago with I beams continuity was not saving them anything, so they do not use it much.
- Decked Slab Beam system: same depth and they span farther and use fewer beam lines to haul out onto a jobsite. They install quicker, but are used primarily on low-volume roads. Fairly new, only been out for 4 years.
- No cracking observed for decked slab beams, but they haven't been out long.
- Conventional 8" concrete deck and spacing beams out 8-10 feet is an alternative – finding it can span same amount as other low-profile adjacent beam bridges but it is a lower cost.
- Overlay Clarification
 - o 5" concrete overlay used on all of adjacent beam systems except decked slab beams
 - o Decked slab beams topped with course surface treatment and sometimes followed up with hot mix asphalt overlay
- Continuity Detail
 - o Place 5" deck continuously across bents for all of adjacent beam systems
 - o Beams are simply supported for all loads
 - o No cast-in-place concrete diaphragms or closures around beam ends
 - o Deck cracks at bents, but manageable and acceptable width
 - o Expansion joints placed at ends of 2 to 4 span units
 - o Have had good success with this method on both I-beams and adjacent beams for decades

Phone Interview with Paul Chung

California Department of Transportation

By Daniel Deery, Clemson University

Date: Tuesday March 30th, 2010

- Majority of bridges are cast-in-place box girders, do not use precast as much so do not construct many adjacent beam bridges.
- Most precast they do for rapid construction is I girders or bulb-T girders.
- Concrete decks on adjacent box beams included in specified 3-4 week erection time.
- They do have target post-tensioning forces but they are specific to project.
- They have not seen much of a longitudinal cracking problem – haven't heard anything from maintenance crews about them.
- Continuity is used: splice girders at bent cap that is cast-in-place and then use post-tensioning through that section.
- Spliced Girder Systems were listed as alternative: is still considered rapid construction but may add a week on for the span erection time.
- The construction time increase due to continuity is insignificant, girders are aged off-site.
- Conjugate beam theory used to estimate positive restraint moments or a finite element analysis can be used to account for creep and shrinkage and to obtain the positive moment and redistribute the moment.

Phone Interview with Benjamin Tang

Oregon Department of Transportation

By Daniel Deery, Clemson University

Date: Tuesday March 30th, 2010

- Deck beams generally do not use asphalt overlay but they do use concrete.
- The concrete overlay causes the erection time of one span to increase from less than one week. Increase depends on situation; some just require a 7-day cure, for example.
- Did not know the target post-tensioning force, but knows one exists in standards.
- Does not believe there is much longitudinal cracking at all, believes there may have been some reflective cracking in earlier designs. They are pleased with their details.
- Can erect some box beam bridges over a weekend (rapid-construction alternative), at least for low-volume bridges.
- Not sure if bridges are made continuous – knows for prestressed beam bridges a continuity diaphragm is used but it's designed like simple-span even though some negative steel may be on top of bent.

Phone Interview with Jugesh Kapur

Washington Department of Transportation

By Daniel Deery, Clemson University

Date: Friday April 2th, 2010

- Do not construct high-profile adjacent beam bridges since other structure types in their inventory are just as or more efficient.
- 5” concrete topping is used to control the longitudinal reflective cracking, it helps to bind everything together and avoid those types of cracks.
- There have been cracking issues, not with the box type, but if using a voided slab or a T-beam without any topping or overlay there is cracking.
- Noticed cracking worse when the AADT is higher.
- Can erect one span including curing of concrete overlay in less than one week – but may not put traffic on it yet.
- Use cast-in-place concrete diaphragm to make bridges continuous and they extend rebar and strands at the intermediate piers and provide longitudinal reinforcement in the topping over the pier at the negative moment location.
- Continuity diaphragm does add some time to the construction.
- To estimate positive restraint moments they take the plastic hinging moment in the column and split it evenly to the two sides (strands extended from superstructure designed to take half on each side).
- Deck bulb-T system is “faster” construction and they use 5” topping for these as well.
- They have refined shear key detail so normal concrete can be used in it, and it has a rod through it which is to help control cracking.
- Shear key detail
 - o The sawtooth detail helps with shear friction transfer especially for live loads at intermediate piers
 - o Pour key concrete with 5” topping because it creates better interlock and load transfer between adjacent beams
- Deck bulb-T system
 - o Erection time: depends on the size of the span, equipment available and experience of the contractor. Typically a beam can be lifted off the ground and placed into position within 30-60 minutes.
 - o Slightly higher span capability than adjacent slabs and adjacent voided slabs

Phone Interview with George Christian

New York Department of Transportation

By Daniel Deery, Clemson University

Date: Tuesday April 6th, 2010

- Not building adjacent beam bridges as much as they used to due to cracking issues and corroding of older bridges – have begun using high performance concrete (HPC) and corrosion inhibitors to attempt to make beams more durable – regional maintenance has soured on them a good bit due to corroding issues.
- Used up to 90-100 foot spans (assuming this is for the adjacent box beams)
- 6” overlay required on these bridges and it is a composite deck – this is to help with shear transfer and durability.
- Use full depth shear keys, used to have partial depth but changed detail over 20 years ago to reduce longitudinal cracking in deck.
- See less of cracking now that they’ve changed shear key and increased their post-tensioning stress (they post-tension after grouting shear key).
- To improve shear transfer began to use rebar in deck instead of mesh.
- Continuous for live load but not fully continuous for dead and live – do this as a matter of practice for multi-spans. Still design positive moment region as simple span to be conservative.
- Continuity does not prolong construction time – not an issue since the deck still needs to be poured and with continuity do not have to install a joint system.
- Deck Bulb-Ts have been used before – not too common.
- Upside down steel composite beams that come in panels and you place them side-by-side “inverset system” – use these a lot.
- “Double T” type of system proposed by PCI Northeast is a new system and they are about to do a job in NYC using it.
- More information on “Inverset” system – rapid, cracks minimized, not temperature sensitive, best quality concrete at the wearing surface (NYDOT design manual pg. 3.53-3.59).

Phone Interview with Greg Perfetti

North Carolina Department of Transportation

By Sara Roberts, Clemson University

Date: Tuesday April 6th, 2010

- Built about 800 – 1000 cored slabs in the last 10 years.
- Has just created preset strand diagrams for different spans at 5’ intervals.
- Use them on NHS and higher ADT with a minimum 3½” concrete overlay with #3 @ 6”.

- Erect the units within a few hours, concrete overlay needs at least 7 days (w/ strength) to cure.
- Use PT, 6/10 strand with around 40,000 lbs of force.
- Grout after post tensioning
- Concrete overlay is very new and only a few cored slab bridges use it.
- Only use the cored slab on higher AADT to avoid clearance/hydraulic issues.
- Got cracks in concrete overlays because the first ones did not have reinforcement, new ones do.
- They have used some cored slabs with no overlay (about 6 of these)
 - o added 2" to the precast unit and then grind the top down
 - o They are new and have done fairly well
 - o Needed to tighten grout specs because some would pull out during grinding
 - o Increased grout strength to 5000 psi (non-shrink, non-metallic)
- Has toyed with using DYWIDAG bars instead of post tensioning.
- In box beams, they use two transverse strands at each location.
 - o Also, they don't put post-tensioning at very end of beam, start at about 8' from end of box beam
- Rare to see longitudinal cracking in bridges with asphalt overlay.
- Do see transverse crack at expansion joints
 - o To prevent this they fix the dowel holes and don't allow expansion anywhere
- Has made a bridge continuous on a design build contract
 - o used U bars coming out of the dowel holes to "staple" the spans together, then put a concrete overlay over that
 - o Do not account for positive moment restraints
- No alternatives. Think it's the most cost effective and they're happy with their performance

Phone Interview with Sandy Tesch

Texas Contractor – ConStar Construction

By Daniel Deery, Clemson University

Date: Wednesday May 19th, 2010

- Constructed some of the first decked slab beams in Texas
- Voids saved about 20,000 pounds on the jobs they performed
- Decked slab beams are advantageous due to speed – takes just one day to install one of the members, one week to complete the bridge
- Worked on a skew bridge and had a problem lining up the beam since the bearing pads were not designed properly
- Expense in these is the large crane, for one day, mainly mobilization it cost \$15,000

- Certified welder is required in order to weld the plates in the shear key – the welding takes 1-2 days to complete
- Largest challenge with deck slab beams is working with a large crane in an area that may limit where the crane can operate

Phone Interview with Bill Heston

North Carolina Contractor – Balfour Beatty

By Daniel Deery, Clemson University

Date: Friday May 28th, 2010

- Cored slab bridges well suited where span lengths can be short and top-down construction is preferred/required
- General work sequence: Excavate first end bent, Drive end bent pile, Form & place end bent concrete & cure, Backfill end bent, Place rip-rap slope protection at end bent, Drive pile at first intermediate bent, Construct intermediate cap & cure concrete, Set first span bearings and cored slabs & install temp handrail, Install transverse PT strands/anchors and stress, Grout cored slab keys, PT strand anchor blockouts, Repeat until structure complete, Place concrete barrier, Place concrete wearing surface, Place approach slabs, Install expansion joints
- The hardest details to work with are the shear keys and concrete wearing surface
- Key width is too narrow, causing significant quantities of grout to be wasted and too much time to ensure they are properly filled. The keys could be twice as wide, use about the same amount of grout, and make filling faster and quality more consistent
- Shear keys are required to be grouted after post-tensioning – grouting before is not an option.
- There is a specified target transverse post-tensioning force and strand elongation and it is not difficult to achieve
- Non-corrosive (PVC) pipes are embedded at the correct location in each cored slab by the fabricator, so transverse-post tensioning alignment has not been an issue
- Typically out of the loop when it comes to performance of any bridge type once construction is completed.

Phone Interview with Chuck Prussack

Washington Fabricator – Central Pre-Mix

By Daniel Deery, Clemson University

Date: Thursday April 29th, 2010

- Built adjacent beam bridges for 50 years, with generally excellent service life

- Most of this type of bridge done in Washington are not for the DOT
- Many of the adjacent member bridges do not have any type of overlay
- Full-length grouted keyway is used
- Weld ties at 5' on-center is typical, even on slab bridges, as opposed to transverse high-strength rods
- Sandblast the keyways at the plant. The keyway configuration is based on an earlier NCHRP study by Mattock and Stanton.
- Weld ties or rods should be about 5' on-center max to provide a tensile tie across the keyway with the grout providing the shear capacity.
- Girder age is not an issue.
- Two current NCHRP projects that examine more robust joints if your state needs to go that direction, Cathy French and Ralph Oesterle are their respective PI's.

Phone Interview with Troy Jenkins

New York Fabricator – Northeast Precast Products

By Daniel Deery, Clemson University

Date: Tuesday May 18th, 2010

- Typically only see adjacent box beams when the vertical clearance cannot be met with a spread beam
- Some states do not permit a joint between the beams. This causes issues in the field because beams are not always perfectly straight and spaces end up between the beams
- No preference on size or type of shear key
- No problem with the 90-day girder age rule since they submit for payment after they hit 28-day strength therefore project schedule must permit this time
- Anytime closed loop stirrups are used the cost goes up
- The only issue, since they use tub forms, is if the shear key gets too thick (3/4") for too low in the beam, they cannot get the beams out of the forms.
- Informed of performance since he sits on a few committees such as PCEF and PCI
- No longer use cardboard hollow beams, the center void is now formed with Styrofoam
- PCI certified plants are required to closely follow Quality Procedure Manuals that didn't exist 15 years ago and are subject to random audits both in-house and independent.
- Joints in bridges need to be shifted off the bridges to keep the chlorides from destroying the ends of the girders

Phone Interview with Mark Losee

New York Fabricator – Jefferson Concrete

By Daniel Deery, Clemson University

Date: Thursday May 20th, 2010

- Originally a small keyway, then they switched to full-depth shear key in early 1990s. There is more labor for full keyway for fabrication but not a problem
- Water blast shear keys to 12,000 psi – 13,000 psi, and also treat the key with a silane sealer which protects it from chloride infiltration
- The state is holding them to 60 days from time of the last pour until the deck pour
- The state is having problems with the older adjacent box beams (his comments suggested corrosion issues)
- In some bridges, the cardboard forms have collapsed and clogged drain holes and once they are unclogged some bridges have been known to drain for days
- New York has taken steps to increase the longevity of their adjacent box beams. New York has looked into double T – they are very wide and large and the new design isn't smooth underneath so not as hydraulically sound as adjacent beam bridges
- There is not a problem with the beams but more of a problem with maintenance; maintenance must be performed in order for these adjacent beam bridges to last.
- Most counties love adjacent box beams: they are friendly to put in and you don't have to worry about the deck or much open space

Phone Interview with Gary Fisher

Texas Fabricator – Flexicore

By Daniel Deery, Clemson University

Date: Thursday May 20th, 2010

- Do not really prefer one to the other (adjacent beams or decked slabs).
- As for advantages, fewer deck slab beams than box beams, but the deck slabs are heavier and cost more, so they require bigger cranes for contractors and are more freight
- They use a solid piece of Styrofoam for voids in members
- Not aware of any fatigue issues yet in decked slab beams (due to dependence on welds), but the bridges have not been used long enough
- Might see a fatigue issue in some of the older double-Ts, which are actually a similar connection, but didn't start using that connection in those until about 10 years ago
- There is no real difficulty in manufacturing the large keyway. There might be some issues with reinforcing if reinforcing is not bent correctly.

- Fabricator's concrete is different than what they are putting on the topping so there may be some expansion differences in the fabricator's concrete and the deck.
- It is an issue to hang on to stuff longer than required and have it take up space – therefore accounting for girder age to ensure positive restraint moments can be ignored in design of continuous bridges is not ideal

Phone Interview with J.R. Parimuha

North Carolina Fabricator – Florence Concrete Products

By Daniel Deery, Clemson University

Date: Thursday May 27th, 2010

- Produce adjacent box beams the most – contractors prefer as well since they are easy to construct and there is an immediate working platform
- North Carolina uses 75% asphalt overlay and the rest concrete, South Carolina uses primarily only asphalt overlay
- Sections without overlay in North Carolina are a good system, but grout used to patch hold-down locations can chip out without an overlay, so must be careful
- In South Carolina they used to use strands for construction and tie rods for maintenance, but now all of South Carolina uses tie rods, which is still post-tensioning but rods instead of strands
- North Carolina always uses the strands, they use a 6/10 cable jacked at 44,000 lbs and for their bigger boxes they use a double 6/10 and two separate ducts next to each other, which gives them a little more post-tensioning.
- Double duct is not difficult for fabricator to put in – South Carolina could use this
- No reason to abandon use of voided sections due to problems in the Northeast
- Made bridge longitudinally continuous in North Carolina before for a federal job – box beams that had continuous steel sticking out of the ends but it was a single span it was more or less to make it continuous with the approaches, it was not multi-span.

Appendix C

Fabricator Workshop Survey

South Carolina Department of Transportation
Research Project No. 682:
Precast Alternative for Flat Slab

Fabricator Workshop Survey



Clemson University
Department of Civil Engineering

Bryant G. Nielson, PhD, PE
Assistant Professor

Scott D. Schiff, PhD
Professor

WeiChiang Pang, PhD
Assistant Professor

Daniel Deery
Graduate Research Assistant

Armando Flores
Graduate Research Assistant

Sara Roberts
Graduate Research Assistant

Purpose:

This survey was created in order to gauge the interest of producers on the various sections that are under consideration by the SCDOT for their short span bridges. Please take some time and fill in all questions completely and accurately so that we can get a clear picture of the fabricator perspective on these sections. These answers are confidential and will not be seen by anyone outside of this research group. Thank you for your time.

1. Please provide your contact information below:

Name:

Company:

City:

State:

Email Address:

Phone Number:

2. What are the three most relevant factors regarding cost when fabricating precast members for bridges?

3. Does adding a void in the members increase the cost of members significantly due to limited space for tendons and other steel?

- Yes
 - Please give an estimate of the cost difference:
- No

4. Does the requirement of voids in members present a significant problem during fabrication?

- Yes
 - Please list some problems:
- No

5. Would requiring headed reinforcing bars for shear transfer between adjacent members present a problem during fabrication?

- Yes
 - Please explain:
- No

6. When surface roughening is required, which method do you prefer?

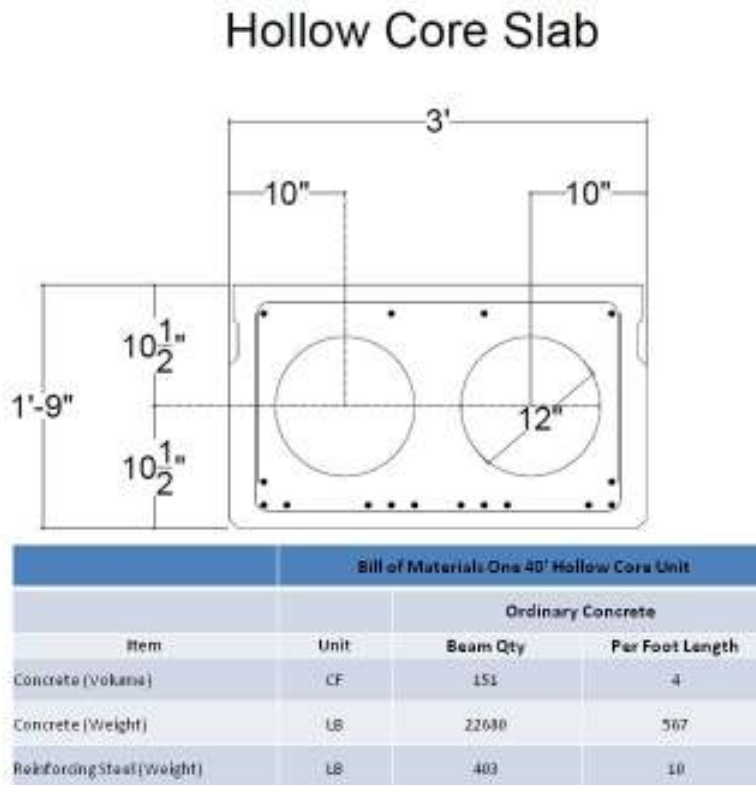
- Sandblasting
- Waterblasting
- No Preference
- Other (please specify)

7. The design is not yet available for some of the new sections and thus the exact number of strands is not known at the time of this survey. Therefore, please provide a cost estimate per foot for ½" diameter straight (not debonded) strands.

In the following segment you are provided with several sections that are being considered for South Carolina's short span bridges. Please provide a cost estimate per linear foot for each section. Please be aware that this information will only be shared outside the research group in a general format without specifics to a particular respondent.

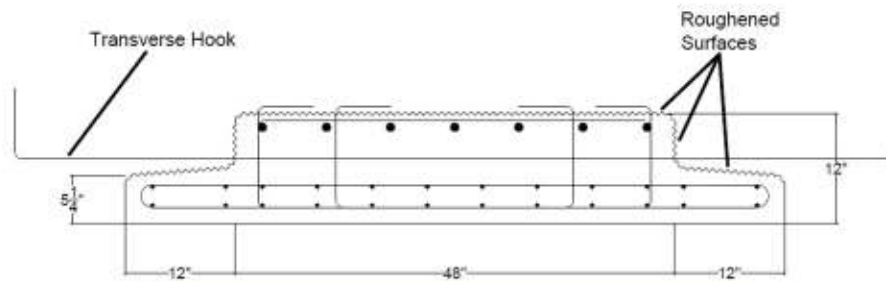
** Note: For the cost of the sections, do not include the cost involving the making of the formwork.

1. Please give a cost per linear foot estimate for the SC Hollow Core Slab (current detail) shown below.



2. Please give a cost per linear foot estimate for the Minnesota DOT Inverted T system shown below. The roughened surfaces shown are likely to be mechanically roughened for this section, not blasted like the shear keys in other details.

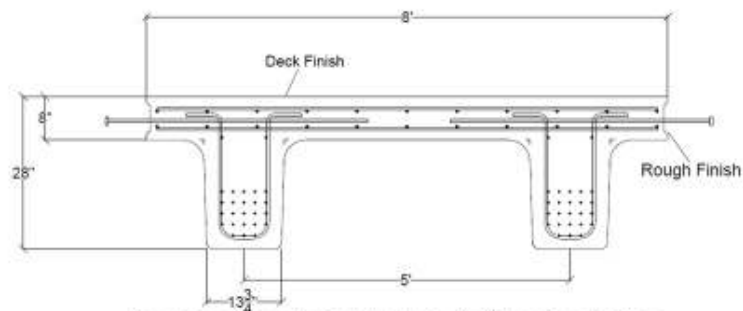
Inverse Tee System (MNDOT)



Bill of Material One 40' Inverted Tee Unit			
Ordinary Concrete			
Item	Unit	Beam Qty	Per Foot Length
Concrete (Volume)	CF	200	5
Concrete (Weight)	LB	2953	749
Reinforcing Steel (Weight)	LB	1804	45

3. Please give a cost per linear foot estimate for NEXT D Beam shown below.

NEXT D Beam

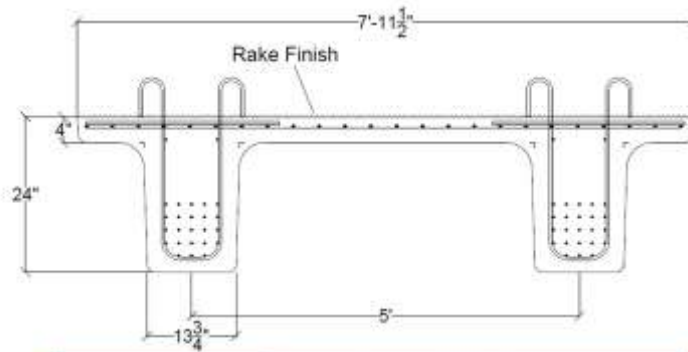


- Roughening of shear key should be done with Water or Sand Blasting

Bill of Material One 40' Next D Unit			
Ordinary Concrete			
Item	Unit	Beam Qty	Per Foot Length
Concrete (Volume)	CF	374	9
Concrete (Weight)	LB	5639	1401
Reinforcing Steel (Weight)	LB	2132	53

4. Please give a cost per linear foot estimate for NEXT F Beam shown below.

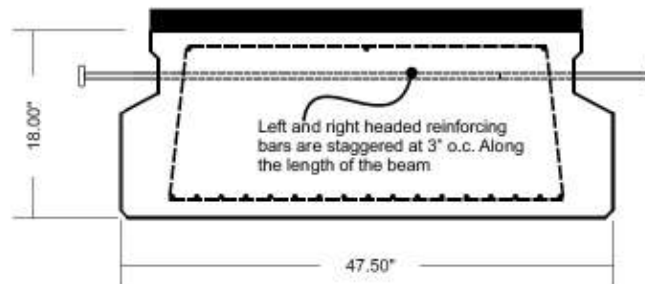
NEXT F Beam



Bill of Material One 40' Next F Unit			
Ordinary Concrete			
Item	Unit	Beam Qty	Per Foot Length
Concrete (Volume)	CF	267	7
Concrete (Weight)	LB	4052	1001
Reinforcing Steel (Weight)	LB	776	776

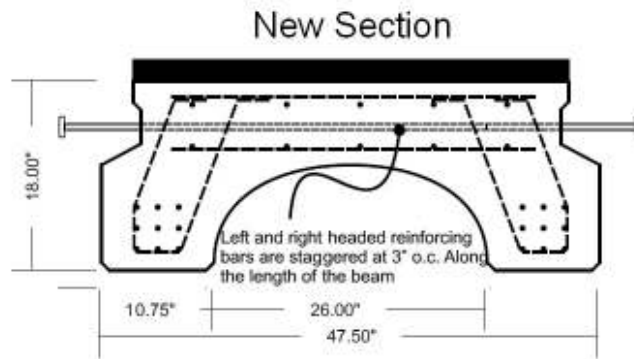
5. Please give a cost per linear foot estimate for the new Clemson 4' wide section shown below.

New Section



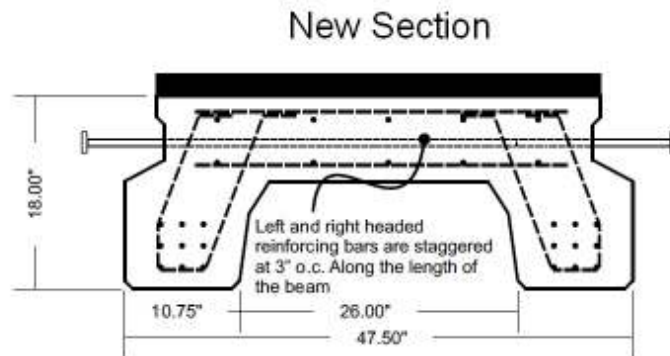
Bill of Materials One 40' 4CU18 Unit			
Ordinary Concrete			
Item	Unit	Beam Qty	Per Foot Length
Concrete (Volume)	CF	247	6
Concrete (Weight)	LB	3702	929
Reinforcing Steel (Weight)	LB	348	348

6. Please give a cost per linear foot estimate for the new Clemson 4' wide section shown below.



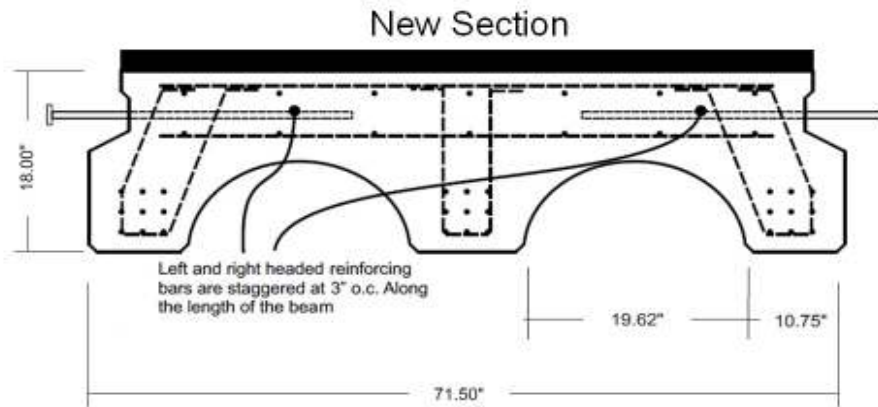
Bill of Materials One 40' 4CU18 Unit			
Ordinary Concrete			
Item	Unit	Beam Qty	Per Foot Length
Concrete (Volume)	CF	189	5
Concrete (Weight)	LB	23375	709
Reinforcing Steel (Weight)	LB	1284	1284

7. Please give a cost per linear foot estimate for the new Clemson 4' wide section shown below.



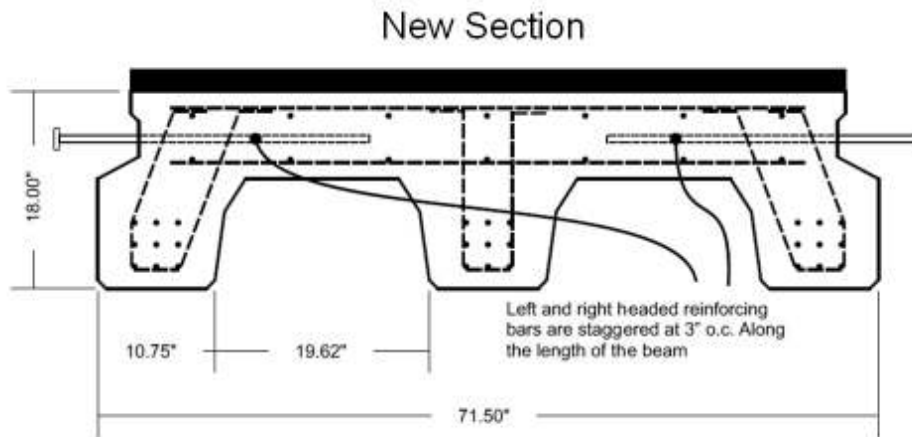
Bill of Materials One 40' 4CU18 Unit			
Ordinary Concrete			
Item	Unit	Beam Qty	Per Foot Length
Concrete (Volume)	CF	180	4
Concrete (Weight)	LB	22955	674
Reinforcing Steel (Weight)	LB	1284	1284

8. Please give a cost per linear foot estimate for the new Clemson 6' wide section shown below.



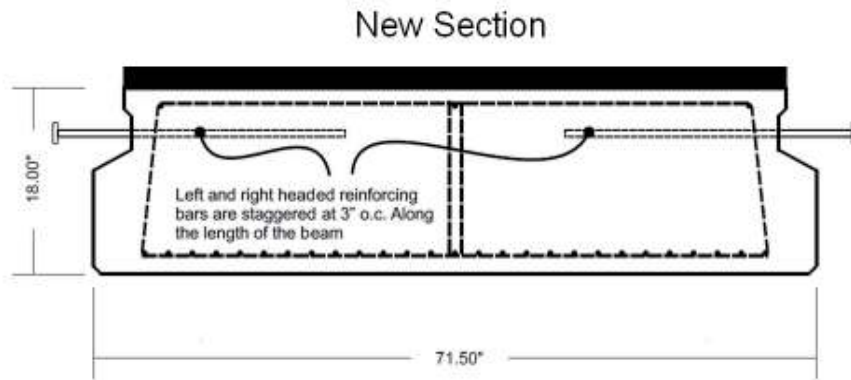
Bill of Materials One 40' SCU21 Unit			
Ordinary Concrete			
Item	Unit	Beam Qty	Per Foot Length
Concrete (Volume)	CF	293	7
Concrete (Weight)	LB	44000	1100
Reinforcing Steel (Weight)	LB	1481	1481

9. Please give a cost per linear foot estimate for the new Clemson 6' wide section shown below.



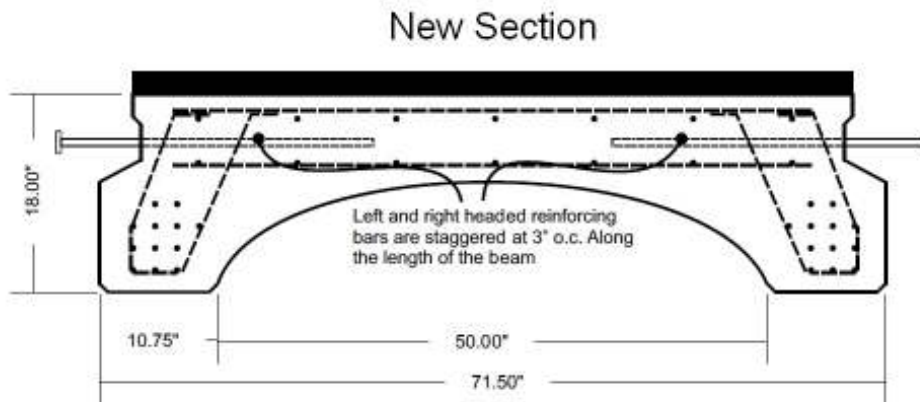
Bill of Materials One 40' SCU21 Unit			
Ordinary Concrete			
Item	Unit	Beam Qty	Per Foot Length
Concrete (Volume)	CF	281	7
Concrete (Weight)	LB	4208	1053
Reinforcing Steel (Weight)	LB	1481	1481

10. Please give a cost per linear foot estimate for the new Clemson 6' wide section shown below.



Bill of Materials One 40' 6CU21 Unit			
Ordinary Concrete			
Item	Unit	Beam Qty	Per Foot Length
Concrete (Volume)	CF	380	10
Concrete (Weight)	LB	57042	1428
Reinforcing Steel (Weight)	LB	1110	1110

11. Please give a cost per linear foot estimate for the new Clemson 6' wide section shown below.



Bill of Materials One 40' 6CU21 Unit			
Ordinary Concrete			
Item	Unit	Beam Qty	Per Foot Length
Concrete (Volume)	CF	269	7
Concrete (Weight)	LB	40375	1009
Reinforcing Steel (Weight)	LB	1072	1072

Appendix D

Contractor and Fabricator Breakout Session Summaries

*** *Questions are set in the context of a 48' wide x 40' single span bridge*

General Contractor Question:

What is the maximum section weight which is reasonable to set without taking extraordinary measures?

Max Section Weight is 30,000 pounds but ideal is 22,000 pounds.

Inverted-Tee:

1. Fabrication difficulty compared to cored-slab (1 being easier, 5 being more difficult):

Five (5)

2. What is the relative time, compared to hollow-core bridges, to construct one span (i.e. set beams, place reinforcement and any concrete/grout)?

More than 50% longer to construct compared to hollow core

3. What is the relative erection cost, compared to hollow-core bridges, to construct one span (i.e. set beams, place reinforcement and any concrete/grout, crane capacity)?

More than 50% more expensive compared to hollow core due to crane size

4. What details of the proposed section are friendly?
 - Bottom flange can be used as a form (Contractor)
 - Knowing the key is filled (Contractor)
 - Concrete is better than grout for keys (Contractor)
 - Expansion coefficient of section and key material are the same (Contractor)
 - No Post-Tensioning (Contractor)
 - Strands are low (Fabricator)
 - Low center of gravity (Fabricator)
5. What details of the proposed section are unfriendly?
 - Transverse Hook – section must be slid under adjacent sections (Contractor)

- Transverse hook may be a safety issue (Contractor)
 - 90 degree hook must be capped for OSHA (Contractor)
 - Multiple Span Set-up (Contractor)
 - Projected steel complicates fabrication (Fabricator)
 - Not top-down construction friendly (Fabricator)
 - Bottom horizontal stirrup too tight (Fabricator)
 - Raked finish difficult on sides (Fabricator)
 - Removal of side forms may be difficult (Fabricator)
 - Hard to screed (Fabricator)
 - Clearance to the bottom flange (Fabricator)
6. What modifications would you propose to ease construction difficulties and cost or fabrication and transport difficulties and cost?
- Eliminate Transverse Hook with Headed Bar, also try a drop-in cage (Contractor)
 - Non-composite design to support crane for top-down construction (Contractor)
 - Roughening of the surface should be done with water-blasting (Fabricator)
 - Draft sides (Fabricator)
 - Cast sides smooth and get bond with rebar (Fabricator)
7. Would light-weight concrete make a big difference in construction time and/or cost or have an impact on fabrication cost?
- No advantage to light-weight concrete (Contractor)
 - It would reduce shipping cost if more than one element can be shipped on truck (Fabricator)
 - Higher material cost (Fabricator)
 - Possibly on shorter widths or shorter sections (Fabricator)

NEXT-D Beam:

1. Fabrication difficulty compared to cored-slab (1 being easier, 5 being more difficult):

Four (4)

2. What is the relative time, compared to hollow-core bridges, to construct one span (i.e. set beams, place reinforcement and any concrete/grout)?

Relatively same time to construct as hollow core

3. What is the relative erection cost, compared to hollow-core bridges, to construct one span (i.e. set beams, place reinforcement and any concrete/grout, crane capacity)?

Between 5% and 25% more expensive compared to hollow core (would require 100 ton crane)

4. What details of the proposed section are friendly?
- Deck in Place (Contractor)
 - Key Details (Contractor)
 - Difficulty similar to hollow core (Fabricator)
 - Forms would allow F or D (Fabricator)
 - Quite versatile, would allow producer to invest in forms (Fabricator)
 - For cross-slope, sloping of the cap would be allowed (It is not preferred in hollow core) (Fabricator)
 - Side forms can be reused for multiple depths and/or widths (Fabricator)
 - No voids (DOT)
 - Clean (Fabricator)
5. What details of the proposed section are unfriendly?
- Weight (Contractor)
 - Studs must be off-set at plant correctly (Contractor)
 - Possible broken corners (Contractor)
 - Grinding – camber between sections (Contractor)
 - Vertical and sag vertical curves would be difficult (Contractor)
 - Projected steel is too frequent (Fabricator)
 - New forms (Fabricator)
6. What modifications would you propose to ease construction difficulties and cost or fabrication and transport difficulties and cost?
- Use stay-in-place-forms for key detail (Contractor)
 - Add 2” cover for grinding (2” min cover must be maintained after grinding) (Contractor)
 - Reduce depth to 18” (Fabricator)
 - Use of sleeves or another alternative to studs (Fabricator)
 - Section should not be 12’ because of the need of a permit to transport it (Fabricator)
 - Removable heads (Fabricator)
 - 4” development length for welded wire fabric, D31 wire = #5 bars and would likely be cheaper (Fabricator)
 - Make joint at center wider to slope crown at center (Fabricator)
 - Possibly using threaded couplers to eliminate bolts (DOT)

7. Would light-weight concrete make a big difference in construction time and/or cost or have an impact on fabrication cost?
 - Could drop crane size one class (Contractor)
 - Could make erection easier (Contractor)
 - It would reduce shipping cost if more than one element can be shipped on truck (Fabricator)
 - Higher material cost (Fabricator)

Clemson Adaptation:

1. Fabrication difficulty compared to cored-slab (1 being easier, 5 being more difficult):

Four (4)

2. What is the relative time, compared to hollow-core bridges, to construct one span (i.e. set beams, place reinforcement and any concrete/grout)?

Relatively same time to construct as hollow core (Contractor)

3. What is the relative erection cost, compared to hollow-core bridges, to construct one span (i.e. set beams, place reinforcement and any concrete/grout, crane capacity)?

Between 5% and 25% more expensive to construct as hollow core (Contractor)

4. What details of the proposed section are friendly?

- Good width (Contractor)
- Fewer joints to grout (Contractor)
- No use of formwork (Contractor)
- Minimal rebar use (Contractor)
- Crane movement for multiple spans (Contractor)
- Not quite as flexible with width adjustment (Fabricator)
- No voids (Fabricator)

5. What details of the proposed section are unfriendly?

- Headed rebar projections (Contractor)
- Headed rebar offset (Contractor)
- Projected steel for side forms (More difficult than NEXT D because of shape) (Fabricator)
- Stirrup placement would be difficult (Fabricator)

- Not as friendly, width difficult to adjust (Fabricator)
 - Stirrups (Fabricator)
6. What modifications would you propose to ease construction difficulties and cost or fabrication and transport difficulties and cost?
- Reduce rebar projection length in order to avoid setting conflict (Contractor)
 - Grinding vs. grooving (Contractor)
 - Dowel details projection from cap. Drill and epoxy (Contractor)
 - Stud splice (Fabricator)
 - Welded wire fabric (Fabricator)
 - Change stirrups to rectangular shape
7. Would light-weight concrete make a big difference in construction time and/or cost or have an impact on fabrication cost?
- It would reduce erection cost (Contractor)
 - It would reduce crane size (Contractor)
 - The use of lightweight might be an issue for the shear key (Contractor)
8. Please rank in order the section most fabricator and contractor friendly.

Section	Rank		Comments (if any)
	Contractors	Fabricators	
Option A	3 rd	4 th	• Elliptical hollows may be a safety issue (Fabricator)
Option B	2 nd	2 nd	
Option C	4 th	1 st	• Simplest (DOT)
Option D	1 st	3 rd	• Elliptical hollows may be a safety issue (Fabricator)

9. Please identify the shear key detail which is most fabricator and contractor friendly.

Section	Rank		Comments (if any)
	Contractors	Fabricators	
Option A	1 st	1 st	• Vertically offset studs (Fabricator)
Option B	2 nd	2 nd	• Makes bed directional (Fabricator) • Makes vertical offset good for fabrication in either section (Fabricator)

Continuity – Headed Option:

1. What aspects of the proposed details are friendly?
 - Pier B intermediate (Contractor)
 - Abutment detail supports multiple construction sequence (Contractor)
 - Pier A intermediate, length of product would affect tolerances (Fabricator)

2. What aspects of the proposed details are unfriendly?
 - Potential rebar conflict (Contractor)
 - Keyway incap (Contractor)
 - Preferred (Contractor)
 - Sensitive tolerance on length (1 1/2" joint) (Fabricator)
 - Not quite flexible, especially for side form (Fabricator)
 - Longitudinal direction location specific, no turn around (Fabricator)

3. What modifications would you propose to ease construction difficulties and cost or fabrication and transport difficulties and cost?
 - Allow straight drop (Contractor)
 - Allow symmetry placement (Contractor)
 - Hook overlap for bar placement (Contractor)
 - Design allowed for crane to move across for multiple spans (Contractor)
 - 6' panel preferred (Contractor)
 - Product symmetry would be easier (Fabricator)

4. Is the fact that the profile at each end of a section is not identical overlay problematic?
 - No (Contractor)
 - Yes, it is too easy to place sections backwards (Contractor)
 - Yes (Fabricator)

Continuity – Hooked Option:

1. What aspects of the proposed details are friendly?
 - Preferred over headed option (Contractor)
 - Runs concrete amount up and makes pouring joints out of truck reasonable (Contractor)
 - Helps being able to have time between pouring approach slab and setting members (Contractor)
 - Ends are the same (Fabricator)
 - Seems pretty clean (Fabricator)

2. What aspects of the proposed details are unfriendly?
 - Projecting rebar (Fabricator)
 - Requires holes in formwork
 - Hooks would reduce shear resistance (Fabricator)
 - Headers made need to be slotted (Fabricator)


3. What modifications would you propose to ease construction difficulties and cost or fabrication and transport difficulties and cost?
 - Increase space to put rods in (Contractor)
 - Change hook to L-shape (Fabricator)
 - Hooked bars placed at top (Fabricator)
 - Preferably bend bars after fabricated (Fabricator)
 - Rebar projecting from cap into hole cast in slab (Fabricator)

Appendix E

Slab Design Sheets

Clemson University		Made DPD	Date 8/28/2010	Sheet No.
		Checked	Date	Of
For NEXT D		BackchK'd	Date	

<p><u>NEXT D8-20 Design Slab</u></p> <p><u>Design Specifications</u> 2007 AASHTO LRFD Bridge Design Specifications, Fourth Edition with 2008 interims.</p> <p><u>Design Method</u> Deck - Load & Resistance Factor Design</p> <p><u>Design Stresses</u> Concrete - $f_c = 6.5$ ksi Reinforcing Steel - $f_y = 60$ ksi</p> <p><u>Design Loading</u> Live Load - HL93 Construction Load - Crane & Timber Distribution Mats Design Speed = 60 mph</p> <p><u>Bridge Geometry</u> Three span prestressed concrete NEXT beam (no skew) Bridge length: 120' CL Brg. To CL Brg. Abutments. Span lengths: 40'-0" - 40'-0" - 40'-0" 6 prestressed NEXT D8-20 girders spaced at 8'-0" centers. 2 Barrier Parapets at 1'-7" each</p> <p><u>Design Lanes</u></p> <table style="margin-left: 20px;"> <tr><td>Roadway Width =</td><td>44.833</td><td>ft</td></tr> <tr><td>2 Barrier Parapets at 1'-7" each =</td><td>3.167</td><td>ft</td></tr> <tr><td>Sidewalk =</td><td>0.000</td><td>ft</td></tr> <tr><td>Out to Out Width =</td><td>48.000</td><td>ft</td></tr> </table> <p>Maximum number of 12 ft lanes = 3</p> <p><u>Dynamic Load Allowance - LRFD</u> (AASHTO 3.6.2) IM = 33% - Applied to vehicular live load on deck</p> <p><u>Load Modifiers - LRFD</u> (AASHTO 13.2)</p> <table style="margin-left: 20px;"> <tr><td>Ductility:</td><td>$\eta_D = 1.0$</td></tr> <tr><td>Redundancy:</td><td>$\eta_R = 1.0$</td></tr> <tr><td>Operation Importance:</td><td>$\eta_I = 1.0$</td></tr> </table> <p style="margin-left: 40px;">$\eta_i = 1.0 \times 1.0 \times 1.0 = 1.0$ $\eta_j = 1.00$</p>	Roadway Width =	44.833	ft	2 Barrier Parapets at 1'-7" each =	3.167	ft	Sidewalk =	0.000	ft	Out to Out Width =	48.000	ft	Ductility:	$\eta_D = 1.0$	Redundancy:	$\eta_R = 1.0$	Operation Importance:	$\eta_I = 1.0$	<p><u>Dead Load</u> Reference dead loads and input loads.</p> <p><u>Live Load</u> Reference load case A and input loads.</p> <p><u>Construction Load</u> Reference load case B and input loads.</p>
Roadway Width =	44.833	ft																	
2 Barrier Parapets at 1'-7" each =	3.167	ft																	
Sidewalk =	0.000	ft																	
Out to Out Width =	48.000	ft																	
Ductility:	$\eta_D = 1.0$																		
Redundancy:	$\eta_R = 1.0$																		
Operation Importance:	$\eta_I = 1.0$																		

Clemson University		Made DPD	Date 9/1/2010	Sheet No.
		Checked	Date	Of
For NEXT D		Backchk'd	Date	

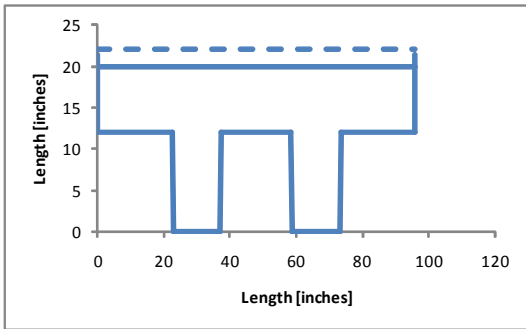
**Next D8-20 Design
Dead Loads**

Slab / Barrier / NEXT D Geometry

- Slab Width, W_s = **48.00** ft
- Deck Slab Thickness, T_s = **8.00** in
- Beam Width, B_f = **96.00** in
- Future Grinding Surface, T_g = **2.00** in
- Overhang Length, O = **22.50** in
- Barrier Area = **425.00** in²
- Slab Edge to Front Face of Barrier = **19.00** in
- Barrier Height = **34.00** in

NEXT D Geometry

- Beam Length, L = **40.00** ft
- Web Spacing, L_w = **3.00** ft
- Beam Depth, D = **1.67** ft
- Beam Spacing, S = **8.00** ft



Weight of Components

- Concrete Weight = **150.00** pcf
- Concrete Grinding Surface = **0.025** ksf
- Top Flange/Deck = **0.100** ksf
- Barrier = **0.443** kip/ft

Exterior Girder Reaction Dead Loads

- Deck (DC) = **0.450** kip/ft
- Concrete Grinding Surface (DC) = **0.113** kip/ft
- Barrier Parapet (DC) = **0.730** kip/ft

Maximum Positive Moment Dead Loads

- Deck (DC) = **0.124** k-ft/ft
- Concrete Grinding Surface (DC) = **0.031** k-ft/ft
- Barrier Parapet (DC) = **0.026** k-ft/ft

Maximum Negative Moment Dead Loads

- Deck (DC) = **-0.313** k-ft/ft
- Concrete Grinding Surface (DC) = **-0.078** k-ft/ft
- Barrier Parapet (DC) = **-0.824** k-ft/ft

Maximum Positive Moment at Shear Key Dead Loads

- Deck (DC) = **0.123** k-ft/ft
- Concrete Grinding Surface (DC) = **0.031** k-ft/ft
- Barrier Parapet (DC) = **0.020** k-ft/ft

Clemson University 	Made DPD	Date 9/7/2010	Sheet No.
	Checked	Date	Of
For NEXT D	Backchk'd	Date	

**Next D8-20 Design - Slab Case A
Completed Structure Load Case**

Total Factored Force Effects, Q (AASHTO 3.4.1-1)

$$Q = \sum ni\gamma_i Q_i$$

Load Modifier (ni) = **1.00**

Qi = Force Effects from Loads
 γi = Load Factors from AASHTO Tables 3.4.1-1 and 3.4.1-2

Load Factors: Strength I (AASHTO Table 3.4.1-1)

- Min. Components and Attachments (DC) = **0.90**
- Max. Components and Attachments (DC) = **1.25**
- Min. Wearing Surface and Utilities (DW) = **0.65**
- Max. Wearing Surface and Utilities (DW) = **1.50**
- Live Load (LL) = **1.75**
- Dynamic Load Allowance (IM) = **1.33**
(AASHTO 3.6.2)

$$\sum ni\gamma_i Q_i = 1.00(\gamma_p DC + \gamma_p DW + (1.75)(LL + IM))$$

Live Load Multiple Presence Factors (AASHTO 3.6.1.1.2)

# Loaded Lanes:	Mult. Presence Factor, m:
1	1.20
2	1.00
3	0.85
4	0.65

Equivalent Strip Widths (AASHTO Table 4.6.2.1.3-1)

+M: 26.0 + 6.6S
-M: 48.0 + 3.0S
Overhang: 45.0 + 10.0X

- Spacing of Supporting Components (S) = **3.00** ft
- Dist. from Barrier cg to Support Point (X) = **0.86** ft
- Width of Primary +M Strip = **3.82** ft
- Width of Primary -M Strip = **4.75** ft
- Width of Primary Overhang Strip = **4.47** ft

Exterior Girder Reaction

$$LL = \frac{mQ}{strip\ width}$$

- Max Live Load Reaction (QR) = **13.66** kip
- Max Live Load Reaction per ft (QR) = **4.29** kip/ft
- Reaction (R) = **7.01** kip/ft
- Factored Reaction (R) = **17.92** kip/ft

Maximum Positive Moment


$$LL = \frac{mQ}{strip\ width}$$

- Max Live Load Positive Moment (Q+M) = **12.78** kip-ft
- Max Live Load Positive Moment per ft (Q+M) = **4.02** k-ft/ft
- Positive Moment (+M) = **5.53** k-ft/ft
- Factored Positive Moment (+M) = **9.58** k-ft/ft

Maximum Negative Moment

$$LL = \frac{mQ}{strip\ width}$$

- Max Live Load Negative Moment (Q-M) = **-6.92** kip-ft
- Max Live Load Negative Moment per ft (Q-M) = **-1.75** k-ft/ft
- Negative Moment (-M) = **-3.54** k-ft/ft
- Factored Negative Moment (-M) = **-5.59** k-ft/ft

Clemson University 	Made DPD	Date 9/12/2010	Sheet No.
	Checked	Date	Of
For NEXT D	Backch'kd	Date	

**Next D8-20 Design - Slab Case B
Construction Load Case**

Total Factored Force Effects, Q (AASHTO 3.4.1-1)

$$Q = \sum n_i \gamma_i Q_i$$

Load Modifier (n_i) = **1.00**

Q_i = Force Effects from Loads
 γ_i = Load Factors from AASHTO Tables 3.4.1-1 and 3.4.1-2

Load Factors: Construction Loads (AASHTO 3.4.2.1)

Components and Attachments (DC) = **1.25**

Construction Loads (CL) = **1.50**

$$\sum n_i \gamma_i Q_i = 1.00(1.25DC + 1.5CL)$$

Construction Loads

Assuming Terex-American HC-80 Hydraulic Crawler Crane

Carbody, 47HI Boom Inner, Side Frames = **88.00** k

47H 40' Boom Center = **2.05** k

47HI Boom Outer = **2.23** k

Jib = **0.00** k

Counterweights = **58.00** k

Crate: Misc Parts, Block and Ball = **3.50** k

NEXT Beam Weight = **51.85** k

Total = **205.62** k

Wood Distribution Loads

Assuming Oak mats used to distribute loads

Unit Weight of Wood = **0.06** kcf
(AASHTO Table 3.5.1-1)

Length of Crane Mat = **20.00** ft

Width of Crane Mat = **4.00** ft

Thickness of Crane Mat = **1.00** ft

Number of Crane Mats = **6**

Weight of Wood = **28.80** k

Total Load on 1 Beam Section (1' strip)

Total Load = **234.42** k

Area of Load = **480** sf

Unfactored Construction Load = **0.488** kip/ft

Unfactored Deck Weight = **0.125** kip/ft

Total Factored Load (C_u) = **0.889** kip/ft
(Not including barrier parapet)

Maximum Support Reaction

Barrier Parapet (DC) = **0.730** kip/ft

Distance from edge beam to support (d) = **2.50** ft

Factored Reaction (R) = **4.47** kip/ft

Maximum Positive Moment

Barrier Parapet (DC) = **0.026** k-ft/ft

Distance from edge beam to support (d) = **2.50** ft


Factored Positive Moment (+M) = **-1.75** k-ft/ft

Maximum Negative Moment

Barrier Parapet (DC) = **-0.824** k-ft/ft

Distance from edge beam to support (d) = **2.50** ft

Factored Negative Moment (-M) = **-3.81** k-ft/ft

Clemson University		Made DPD	Date 9/7/2010	Sheet No.
		Checked	Date	Of
For NEXT D		Backchk'd	Date	

<p>NEXT D8-20 Design Slab</p> <p>Slab / Barrier / NEXT D Geometry</p> <p>Slab Width, W_s = 48.00 ft</p> <p>Deck Slab Thickness, T_s = 8.00 in</p> <p>Beam Width, B_f = 96.00 in</p> <p>Future Grinding Surface, T_g = 2.00 in</p> <p>Overhang Length, O = 22.50 in</p> <p>Barrier Area = 425.00 in²</p> <p>Slab Edge to Front Face of Barrier = 19.00 in</p> <p>Barrier Height = 34.00 in</p> <p>NEXT D Geometry</p> <p>Beam Length, L = 40.00 ft</p> <p>Web Spacing, L_w = 3.00 ft</p> <p>Beam Depth, D = 1.67 ft</p> <p>Beam Spacing, S = 8.00 ft</p> <div data-bbox="263 1165 797 1497" data-label="Figure"> </div>	<p>Bottom Reinforcing Design</p> <p>Total Moment to Resist, $+M_u$ = 9.58 k-ft/ft (factored)</p> <p>Longitudinal Bar Size = 0.500 in (#4 bars)</p> <p>Area of Longitudinal Steel = 0.20 in²</p> <p>Area of Longitudinal Steel = 0.24 in² per ft</p> <p>Reinforcing Size = 0.625 in (#5 main bars)</p> <p>Minimum Cover = 1.000 in (AASHTO Table 5.12.3-1)</p> <p>d = 6.688 in (average)</p> <p>Reinforcing Bar Spacing = 11 in</p> <p>Area of Reinforcing Bar = 0.31 in²</p> <p>Total Area of Steel, A_s = 0.34 in² per ft</p> <p>Reference Bottom Slab Reinforcement Sheet</p> <p>M_r = 9.94 k-ft/ft O.K.</p> <p>USE: #5 Bars at 11 in (Bottom of Slab), A_s = 0.34 in² per ft USE: #4 Longitudinal Bars (Bottom of Slab) @ 10 in</p> <p>Top Reinforcing Design</p> <p>Total Moment to Resist, $-M_u$ = 5.59 k-ft/ft (factored)</p> <p>Longitudinal Bar Size = 0.500 in (#4 bars)</p> <p>Area of Longitudinal Steel = 0.13 in² per ft</p> <p>Reinforcing Size = 0.625 in (#5 main bars)</p> <p>Minimum Cover = 2.000 in (AASHTO Table 5.12.3-1)</p> <p>d = 5.688 in (average)</p> <p>Reinforcing Bar Spacing = 12 in</p> <p>Area of Reinforcing Bar = 0.31 in²</p> <p>Total Area of Steel, A_s = 0.31 in² per ft</p> <p>Reference Top Slab Reinforcement Sheet</p> <p>M_r = 7.74 k-ft/ft O.K.</p> <p>USE: #5 Bars at 12 in (Top of Slab), A_s = 0.31 in² per ft USE: #4 Longitudinal Bars (Top of Slab) @ 18 in</p>
--	---

NEXT D8-20 Design

Slab-Bottom Reinforcement (page 1 of 2)

Flexural Check:

AASHTO 5.7.3.2

$$\begin{aligned}M_u &= 9.58 \text{ k-ft} \\A_s &= 0.338 \text{ in}^2 \\d &= 6.69 \text{ in} \\b &= 12 \text{ in} \\f_c &= 6.50 \text{ ksi} \\f_y &= 60 \text{ ksi} \\E_c &= 4888 \text{ ksi} \\n &= 5.93\end{aligned}$$

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = 0.31 \text{ in}$$

$$M_r = \phi A_s F_y \left(d - \frac{a}{2} \right)$$

$$M_r = 9.94 \text{ k-ft}$$

$$\frac{4}{3} M_u = 12.77 \text{ k-ft}$$

O.K.

$$9.94 \text{ k-ft} > 9.58 \text{ k-ft}$$

Cracking Moment:

AASHTO 5.7.3.6.2, 5.7.3.3.2

$$D = 8.00 \text{ in}$$

$$f_r = 0.24 \sqrt{f_c'}$$

$$\text{(normal wt) } f_r = 0.612 \text{ ksi}$$

(AASHTO 5.4.2.6)

$$I_g = 512 \text{ in}^4$$

$$y_t = 4.00 \text{ in}$$

$$M_{cr} = \frac{f_r I_g}{y_t}$$

$$M_{cr} = 6.53 \text{ k-ft}$$

$$1.2 M_{cr} = 7.83 \text{ k-ft}$$

1.2 M_{cr} Controls over 4/3 M_u

Min. Reinforcement O.K.

$$9.94 \text{ k-ft} > 7.83 \text{ k-ft}$$

Crack Control Check:

AASHTO 5.7.3.4

$$M_{SL} = 5.53 \text{ k-ft}$$

$$d_c = 1.31 \text{ in}$$

$$h = 8.00 \text{ in}$$

$$S_{max} = 11.00 \text{ in}$$

$$Y_e = 1.00$$

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

$$\beta_s = 1.280$$

$$A_s = 0.338 \text{ in}^2$$

$$f_{sa} = \frac{M_{sl}}{d - a/2} / A_s$$

$$f_{sa, \text{actual}} = 30.00 \text{ ksi}$$

$$s \leq \frac{700 y_e}{\beta_s f_s} - 2 d_c$$

$$S_{req'd} = 15.60 \text{ in}$$

O.K.

$$11.00 \text{ in} < 15.60 \text{ in}$$

NEXT D8-20 Design

Slab-Bottom Reinforcement (page 2 of 2)

Maximum Reinf Spacing

AASHTO 5.10.3.2

$$\begin{aligned} D &= 8.00 \text{ in} \\ s \text{ (limit)} &= 12.00 \text{ in} \\ s &= 11.00 \text{ in} \end{aligned}$$

O.K.

$$11.00 \text{ in} < 12.00 \text{ in}$$

Temperature/Shrinkage

AASHTO 5.10.8

$$As \geq \frac{1.30bh}{2(b+h)fy}$$

$$Asl \text{ (min)} = 0.052 \text{ in}^2 \text{ per ft}$$

$$0.11 \leq As \leq 0.60$$

$$Asl \text{ (req)} = 0.110 \text{ in}^2 \text{ per ft}$$

$$s \text{ (limit)} = 18.00 \text{ in}$$

$$Asl \text{ (prov)} = 0.133 \text{ in}^2 \text{ per ft}$$

O.K.

$$0.133 \text{ sq in} > 0.110 \text{ sq in}$$

Distribution Reinforcement

AASHTO 9.7.3.2

$$S = 1.8 \text{ ft}$$

For primary reinforcement perpendicular to traffic:

Percentage of primary reinforcement req'd:

$$\frac{220}{\sqrt{S}} \leq 67\%$$

$$\% \text{ Primary} = 67.00$$

$$Asl \text{ (req)} = 0.227 \text{ in}^2 \text{ per ft}$$

$$s \text{ (req)} = 10.59 \text{ in}$$

$$s \text{ (prov)} = 10.00 \text{ in}$$

$$Asl \text{ (prov)} = 0.240 \text{ in}^2 \text{ per ft}$$

O.K.

$$0.240 \text{ sq in} > 0.227 \text{ sq in}$$

Fatigue Check:

AASHTO 9.5.3: Fatigue need not be investigated for concrete decks in multigirder applications

NEXT D8-20 Design

Slab-Top Reinforcement (page 1 of 2)

Flexural Check:

AASHTO 5.7.3.2

$$\begin{aligned}M_u &= 5.59 \text{ k-ft} \\A_s &= 0.310 \text{ in}^2 \\d &= 5.69 \text{ in} \\b &= 12 \text{ in} \\f_c &= 6.50 \text{ ksi} \\f_y &= 60 \text{ ksi} \\E_c &= 4888 \text{ ksi} \\n &= 5.93\end{aligned}$$

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = 0.28 \text{ in}$$

$$M_r = \phi A_s F_y \left(d - \frac{a}{2}\right)$$

$$M_r = 7.74 \text{ k-ft}$$

$$\frac{4}{3} M_u = 7.45 \text{ k-ft}$$

O.K.

$$7.74 \text{ k-ft} > 7.45 \text{ k-ft}$$

Cracking Moment:

AASHTO 5.7.3.6.2, 5.7.3.3.2

$$D = 8.00 \text{ in}$$

$$f_r = 0.24 \sqrt{f_c'}$$

$$\text{(normal wt) } f_r = 0.612 \text{ ksi}$$

(AASHTO 5.4.2.6)

$$I_g = 512 \text{ in}^4$$

$$y_t = 4.00 \text{ in}$$

$$M_{cr} = \frac{f_r I_g}{y_t}$$

$$M_{cr} = 6.53 \text{ k-ft}$$

$$1.2 M_{cr} = 7.83 \text{ k-ft}$$

4/3 Mu Controls over 1.2 Mcr

Min. Reinforcement O.K.

$$7.74 \text{ k-ft} > 7.45 \text{ k-ft}$$

Crack Control Check:

AASHTO 5.7.3.4

$$M_{SL} = 3.54 \text{ k-ft}$$

$$d_c = 1.31 \text{ in}$$

$$h = 8.00 \text{ in}$$

$$S_{max} = 12.00 \text{ in}$$

$$Y_e = 1.00$$

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

$$\beta_s = 1.280$$

$$A_s = 0.310 \text{ in}^2$$

$$f_{sa} = \frac{M_{sl}}{d - a/2} / A_s$$

$$f_{sa, \text{actual}} = 24.70 \text{ ksi}$$

$$s \leq \frac{700 y_e}{\beta_s f_s} - 2 d_c$$

$$S_{req'd} = 19.51 \text{ in}$$

O.K.

$$12.00 \text{ in} < 19.51 \text{ in}$$

NEXT D8-20 Design

Slab-Top Reinforcement (page 2 of 2)

Maximum Reinf Spacing

AASHTO 5.10.3.2

$$\begin{aligned} D &= 8.00 \text{ in} \\ s \text{ (limit)} &= 12.00 \text{ in} \\ s &= 12.00 \text{ in} \end{aligned}$$

O.K.

$$12.00 \text{ in} = 12.00 \text{ in}$$

Temperature/Shrinkage

AASHTO 5.10.8

$$As \geq \frac{1.30bh}{2(b+h)f_y}$$

$$Asl \text{ (min)} = 0.052 \text{ in}^2 \text{ per ft}$$

$$0.11 \leq As \leq 0.60$$

$$Asl \text{ (req)} = 0.110 \text{ in}^2 \text{ per ft}$$

$$s \text{ (limit)} = 18.00 \text{ in}$$

$$Asl \text{ (prov)} = 0.133 \text{ in}^2 \text{ per ft}$$

O.K.

$$0.133 \text{ sq in} > 0.110 \text{ sq in}$$

Fatigue Check:

AASHTO 9.5.3: Fatigue need not be investigated for concrete decks in multigirder applications

Clemson University 	Made DPD	Date 9/17/2010	Sheet No.
	Checked	Date	Of
For NEXT D	Backchk'd	Date	

Next D8-20 Design Deck Overhang

Assume barrier is TL-4 - Test Level Four - generally acceptable for majority of applications on high speed highways, freeways, expressways, and Interstate highways with mixture of trucks and heavy vehicles (AASHTO 13.7.2)

Assume the traffic railings are proven satisfactory through crash testing for desired test level (AASHTO 13.7.3.1)

Height of barrier must be at least 32" for TL-4 (AASHTO 13.7.3.2)

Total Factored Force Effects, Q (AASHTO 3.4.1-1, A13.4)

$$Q = \sum niyiQi$$

Load Modifier (ni) = **1.00**

Qi = Force Effects from Loads
 yi = Load Factors from AASHTO Tables 3.4.1-1 and 3.4.1-2

Overhang Load Cases (AASHTO A13.4, 13.6.1, 13.6.2)

- Design Case 1: Transverse/Longitudinal Forces
 Specified in AASHTO A13.2
 Extreme Event Load Combination II Limit State
- Design Case 2: Vertical Forces
 Specified in AASHTO A13.2
 Extreme Event Load Combination II Limit State
- Design Case 3: Loads that occupy overhang
 Specified in Article 3.6.1
 Strength I Load Combination Limit State

Assume Design Case 2 does not control since this case never controls over Case 1 for a concrete parapet (FHWA Design Example)

Assume Design Case 3 does not control since this case only controls if the length of the cantilever is very long (Barker 563-564)

Design philosophy is to ensure deck overhang region has a larger resistance than the actual resistance of the concrete parapet, therefore, the parapet, which can be replaced easily, would fail before the deck overhang (AASHTO C A13.3.1)

Case 1 Load Factors: Extreme Event II (AASHTO Table 3.4.1-1)

Max. Components and Attachments (DC) = **1.25**

Vehicle Collision Load Factor (CT) = **1.00**

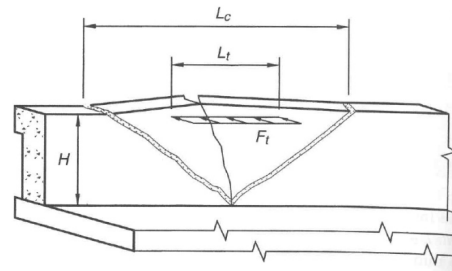
$$\sum niyiQi = 1.00(1.25DC + 1.00CT)$$

Concrete Barrier Strength (AASHTO A13.3.1)

Developed using a yield line approach - must be used to determine the magnitude of loads that must be transferred to deck overhang.

$$Rw = \left(\frac{2}{2Lc - Lt} \right) \left(8Mb + 8Mw + \frac{McLc^2}{H} \right)$$

$$Lc = \frac{Lt}{2} + \sqrt{\left(\frac{Lt}{2} \right)^2 + \frac{8H(Mb + Mw)}{Mc}}$$



(Barker 566)

Extra Beam Resistance, Mb = **0.00** k-ft
 (Assumed)

Height of Wall, H = **2.83** ft

Distrib. Length of Force, Lt = **3.50** ft
 (AASHTO Table A13.2-1)

Flexural Resistance of Wall about Vertical Axis, Mw

Assume wall has uniform thickness with actual wall area

Barrier Area = **425.00** in²

h_{avg} = **12.5** in

d_{avg} = **9.5** in
 (assumed 3" cover - bars to face)

As = **0.44** in²
 (assumed 4 - No. 3 bars)

fc = **6.50** ksi


fy = **60** ksi

$$a = \frac{Asfy}{0.85fc'b}$$

a = **0.141** in

$$Mw = \phi AsFy \left(d - \frac{a}{2} \right)$$

Mw = **20.75** k-ft

Clemson University 	Made DPD	Date 9/20/2010	Sheet No.
	Checked	Date	Of
For NEXT D	Backch'kd	Date	

**Next D8-20 Design
Deck Overhang**

Flexural Resistance of Wall about Longitudinal Axis, Mc

Yield lines crossing vertical reinforcement produce only tension in the sloping face of the wall, so only negative bending strength needs to be calculated.

Split Barrier into 2 segments: seg 1 = top 19", seg 2 = bottom 15"

Vertical Bar Diameter 1 = **0.50** in

Vertical Bar Area 1 (A1) = **0.20** in² per ft
(assumed No. 4 bars at 12" oc)

$h_{avg1} = 10.00$ in

$d_{avg1} = 7.25$ in
(assumed 2.75" cover - vert bars to sloped face)

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

$a = 0.181$ in

$$M_{c1} = \phi A_s F_y \left(d - \frac{a}{2} \right)$$

$M_{c1} = 7.16$ k-ft/ft

Vertical Bar Diameter 2 = **0.50** in

Vertical Bar Area 2 (A2) = **0.20** in² per ft
(assumed No. 4 bars at 12" oc)

$h_{avg2} = 15.67$ in

$d_{avg2} = 12.92$ in
(assumed 2.75" cover - vert bars to sloped face)

$$M_{c2} = \phi A_s F_y \left(d - \frac{a}{2} \right)$$

$M_{c2} = 12.83$ k-ft/ft

Weighted Average of segment 1 and 2 Resistances:

$M_c = 9.66$ k-ft/ft

Critical Length of Yield Line Failure Pattern, Lc

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8H(M_b + M_w)}{M_c}}$$

$L_c = 8.94$ ft

Nominal Resistance to Transverse Load, R_w

$$R_w = \left(\frac{2}{2L_c - L_t} \right) \left(8M_b + 8M_w + \frac{M_c L_c^2}{H} \right)$$

$R_w = 60.98$ k

$F_t = 54.00$ k
(AASHTO Table A 13.2-1)

O.K.

$60.98 \text{ k} > 54.00 \text{ k}$

Barrier can resist transverse vehicular collision force

Shear Transfer Between Barrier and Deck (AASHTO 5.8.4)

Nominal resistance R_w must be transferred across cold joint by shear friction.

The tensile force per unit of length in the overhang, T:

$$T = \frac{R_w}{L_c + 2H}$$

(AASHTO A 13.4.2-1)

$T = 4.17$ kip/ft

The nominal shear resistance of interface plane, V_n:

$$V_n = c A_{cv} + \mu (A_{vf} F_y + P_c)$$

(AASHTO 5.8.4.13)

$A_{cv} = 216.00$ in²

$A_{vf} = 0.20$ in² per ft

Assuming concrete placed against clean, laitance free, not intentionally roughened concrete surface (AASHTO 5.8.4.3):

$c = 0.08$ ksi
(AASHTO 5.8.4.3)

$\mu = 0.60$
(AASHTO 5.8.4.3)

$K_1 = 0.20$
(AASHTO 5.8.4.3)

$K_2 = 0.80$ ksi
(AASHTO 5.8.4.3)

$f_c = 6.50$ ksi

$f_y = 60$ ksi

$P_c = 0.443$ kip/ft

Clemson University 	Made DPD	Date 9/21/2010	Sheet No.
	Checked	Date	Of
For NEXT D	Backch'kd	Date	

**Next D8-20 Design
Deck Overhang**

$$V_n = 23.67 \text{ kip/ft}$$

$$V_n \leq K_1 f'_c A_{cv}$$

and

$$V_n \leq K_2 A_n$$

$$V_n \text{ limit} = 172.8 \text{ kip/ft}$$

$$\phi V_n = 21.30 \text{ kip/ft}$$

O.K.

$$21.30 \text{ kip/ft} > 4.17 \text{ kip/ft}$$

Interface plane can resist shear caused by collision

Minimum Area of Interface Shear Reinforcement:

$$A_{vf} \geq \frac{0.05b_v}{f_y}$$

(AASHTO 5.8.4.4-1)

$$A_{vf} \geq \frac{\left(\frac{1.33V_u}{\phi} - cA_{cv}\right) - P_c}{\mu f_y}$$

(AASHTO 5.8.4.4)

$$A_{vf} \text{ (min)} = 0.18 \text{ in}^2 \text{ per ft}$$

O.K.

$$0.20 \text{ sq in/ft} > 0.18 \text{ sq in/ft}$$

USE: #4 Hairpin Dowels at 12 in (barrier to deck), $A_s = 0.20 \text{ in}^2$

Development Length for Vertical Dowel Bar:

$$l_{dh} = \frac{38.0d_b}{\sqrt{f'_c}}$$

(AASHTO 5.112.4.1-1)

$$l_{dh} = 7.45 \text{ in}$$

$$l_{dh} \text{ (reduction)} = 0.70 \text{ (side cover)}$$

(AASHTO 5.112.4.2)

$$l_{dh} = 6.00 \text{ in}$$

(AASHTO 5.112.4.1)

$$l_{dh} \text{ (prov)} = 6.00 \text{ in}$$

O.K.

USE: #4 Hairpin Dowels developed 6 in into deck

Top Reinforcement in Deck Overhang

Top Reinforcement must resist negative bending moment over the exterior beam due to the collision and dead load of overhang.

$$\text{Collision Moment, } M_{CT} = 11.83 \text{ k-ft/ft}$$

$$\text{Total Factored Moment, } M_u = 13.34 \text{ k-ft/ft}$$

$$\text{Reinforcing Size} = 0.625 \text{ in}$$

(#5 headed bars)

$$d = 5.688 \text{ in}$$

(average)

$$\text{Reinforcing Bar Spacing} = 6 \text{ in}$$

$$\text{Area of Reinforcing Bar} = 0.31 \text{ in}^2$$

$$\text{Total Area of Steel, } A_s = 0.62 \text{ in}^2 \text{ per ft}$$

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

$$f_c = 6.50 \text{ ksi}$$

$$f_y = 60 \text{ ksi}$$

$$a = \frac{A_s f_y}{0.85 f'_c b}$$

$$a = 0.56 \text{ in}$$

$$M_r = \phi A_s F_y \left(d - \frac{a}{2}\right)$$

$$M_r = 15.09 \text{ k-ft/ft}$$

Must reduce moment strength due to axial tension force, T

$$T = 4.17 \text{ kip/ft}$$

$$M_u \leq \phi M_n \left(1.0 - \frac{P_u}{\phi P_n}\right)$$

$$P_u = 4.17 \text{ kip/ft}$$

$$\text{Total Long. Reinforcement, } A_{st} = 0.96 \text{ in}^2 \text{ per ft}$$

$$\phi P_n = 57.49 \text{ kip/ft}$$

$$M_r \text{ (including axial effect)} = 13.99 \text{ k-ft/ft}$$

O.K.

USE: #5 Bars at 6 in (Top of Overhang), $A_s = 0.62 \text{ in}^2$

Clemson University 	Made DPD	Date 9/22/2010	Sheet No.
	Checked	Date	Of
For NEXT D	Backchk'd	Date	

**Next D8-20 Design
Deck Overhang**

Development for Top Reinf in Overhang (AASHTO 5.11.2.4.1)

Top reinforcement must resist M_{CT} directly below barrier.
Therefore, use standard 180 degree hooks for top reinforcement.

$$l_{dh} = \frac{38.0d_b}{\sqrt{f'c}}$$

(AASHTO 5.11.2.4.1-1)

$$l_{dh} = 9.32 \text{ in}$$

$$l_{dh} \text{ (reduction)} = 0.70 \text{ (side cover)}$$

(AASHTO 5.11.2.4.2)

$$l_{dh} = 6.52 \text{ in}$$

(AASHTO 5.11.2.4.1)

$$l_{dh} \text{ (prov)} = 11.50 \text{ in}$$

O.K.

USE: #5 Bars hooked 180 deg and developed 11.50 in

Length of Additional Overhang Reinf (AASHTO 5.11.2.1.1)

Must find point where moment caused by vehicle collision is equal to the capacity of the standard top reinforcement bars. At this point, plus the distance specified in AASHTO 5.11.1.2, the additional bars for the overhang can be cut off.

x = distance from centerline of 1st support to the point where extra bars are not needed.

Reference Slab Design Sheet

$$M_r = 7.74 \text{ k-ft/ft}$$

$$M_c = 8.60 \text{ k-ft/ft}$$

(adjusting from $\phi=0.9$ to $\phi=1.0$)

Assuming carryover factor of 0.5 and no further distribution
Neglecting moment contribution from dead loads (conservative)

$$M_{CT} \left(1 - \frac{x}{\frac{2}{3} \text{ span}} \right) = M_r$$

$$x = 0.91 \text{ ft}$$

$$\text{Additional Length Required} = 9.38 \text{ in}$$

(AASHTO 5.11.12)

$$\text{Total Length of Bar Required} = 20.29 \text{ in}$$

Check Development Length (AASHTO 5.11.1, 5.11.2):
Calculating development length from face of support
Compare this value to that determined based on moment capacity

$$l_d \geq 0.4d_b f_y$$

$$\text{Development Length min} = 15.00 \text{ in}$$

(AASHTO 5.11.12.1)

$$l_d = \frac{1.25A_b F_y}{\sqrt{f'c}}$$

$$\text{Development Length} = 18.24 \text{ in}$$

$$\text{Total Length of Bar Required} = 25.74 \text{ in}$$

(add in half of web width)

USE: #5 extra bars for 26 in past centerline of first web

Clemson University 	Made DPD	Date 9/12/2010	Sheet No.
	Checked	Date	Of
For NEXT D	Backchk'd	Date	

Next D8-20 Design
Shear Key - Headed Reinforcing Bar

Total Factored Force Effects, Q (AASHTO 3.4.1-1)

$$Q = \sum ni\gamma_i Q_i$$

Load Modifier (ni) = **1.00**

Qi = Force Effects from Loads
γi = Load Factors from AASHTO Tables 3.4.1-1 and 3.4.1-2

Load Factors: Strength I (AASHTO Table 3.4.1-1)

- Min. Components and Attachments (DC) = **0.90**
- Max. Components and Attachments (DC) = **1.25**
- Min. Wearing Surface and Utilities (DW) = **0.65**
- Max. Wearing Surface and Utilities (DW) = **1.50**
- Live Load (LL) = **1.75**
- Dynamic Load Allowance (IM) = **1.33**
(AASHTO 3.6.2)

$$\sum ni\gamma_i Q_i = 1.00(\gamma_p DC + \gamma_p DW + (1.75)(LL + IM))$$

Live Load Multiple Presence Factors (AASHTO 3.6.1.1.2)

# Loaded Lanes:	Mult. Presence Factor, m:
1	1.20
2	1.00
3	0.85
4	0.65

Equivalent Strip Widths (AASHTO Table 4.6.2.1.3-1)

+M: 26.0 + 6.6S
-M: 48.0 + 3.0S
Overhang: 45.0 + 10.0X

- Spacing of Supporting Components (S) = **3.00** ft
- Dist. from Barrier cg to Support Point (X) = **0.86** ft
- Width of Primary +M Strip = **3.82** ft

Maximum Positive Moment

$$LL = \frac{mQ}{strip\ width}$$

- Max Live Load Positive Moment (Q_{+M}) = **13.03** kip-ft
- Max Live Load Positive Moment per ft (Q_{+M}) = **4.10** k-ft/ft
- Positive Moment (+M) = **5.62** k-ft/ft
- Factored Positive Moment (+M) = **9.75** k-ft/ft

Headed Reinforcing Bar Design (AASHTO 5.7.3.2)

Total Moment to Resist, +M_u = **9.75** k-ft/ft (factored)

- Reinforcing Size = **0.625** in
(#5 headed bars)
- d = **4.000** in
(mid-depth of key)
- Reinforcing Bar Spacing = **6** in

Area of Reinforcing Bar = **0.31** in²
Total Area of Steel, A_s = **0.62** in² per ft

- b = **12** in
- f_c = **6.50** ksi
- f_y = **60** ksi


$$\alpha = \frac{A_s f_y}{0.85 f_c' b}$$

a = **0.56** in

$$M_r = \phi A_s F_y \left(d - \frac{a}{2} \right)$$

M_r = **10.38** k-ft
O.K.

USE: #5 Headed Bars at 6 in, A_s = 0.62 in² per ft

Clemson University		Made	DPD	Date	9/12/2010	Sheet No.
		Checked		Date		Of
For	NEXT D	Backch'kd		Date		

Next D8-20 Design
Shear Key - Headed Reinforcing Bar

Development Length (AASHTO 5.11.1, 5.11.2)

$$ld \geq 0.4d_b f_y$$

Development Length min = 15.00 in
(AASHTO 5.11.2.1)

Total Area of Steel, A_s (prov) = 0.62 in² per ft

Total Area of Steel, A_s (req) = 0.60 in² per ft

f_c = 6.50 ksi

f_y = 60 ksi

$$ld = \frac{1.25 A_b F_y}{\sqrt{f_c}}$$

Development Length = 18.24 in

$$ld \text{ reduction} = \frac{A_s(\text{req})}{A_s(\text{prov})}$$

Required Development Length = 17.7 in

USE: #5 Headed Bars with at least 18 in of embedment

Appendix F

OpenSees Scripts

1. Simulation.tcl Script:

```
source BridgeDeck.tcl

set N_beam    8
set I_beam    512
set I_girder  14439
set E         4400
set stem_center 36
set W_beam    96
set name      "Bridge"

set fileID [open beamwidth.out w]
puts $fileID $W_beam
close $fileID

set k_stiff(0,0) 1e15
set k_stiff(0,1) 1e12
set k_stiff(0,2) 1e9
set k_stiff(0,3) 5e8
set k_stiff(0,4) 1e8
set k_stiff(0,5) 8e7
set k_stiff(0,6) 6e7
set k_stiff(0,7) 4e7
set k_stiff(0,8) 2e7
set k_stiff(0,9) 1e7
set k_stiff(0,10) 8e6
set k_stiff(0,11) 6e6
set k_stiff(0,12) 4e6
set k_stiff(0,13) 3e6
set k_stiff(0,14) 2e6
set k_stiff(0,15) 1e6
set k_stiff(0,16) 9e5
set k_stiff(0,17) 8e5
set k_stiff(0,18) 7e5
set k_stiff(0,19) 6e5
set k_stiff(0,20) 5e5
set k_stiff(0,21) 4e5
set k_stiff(0,22) 3e5
set k_stiff(0,23) 2e5
```

```
set k_stiff(0,24) 1e5
set k_stiff(0,25) 80000
set k_stiff(0,26) 60000
set k_stiff(0,27) 40000
set k_stiff(0,28) 25000
set k_stiff(0,29) 20000
set k_stiff(0,30) 15000
set k_stiff(0,31) 10000
set k_stiff(0,32) 7500
set k_stiff(0,33) 5000
set k_stiff(0,34) 4000
set k_stiff(0,35) 3000
set k_stiff(0,36) 2500
set k_stiff(0,37) 2000
set k_stiff(0,38) 1500
set k_stiff(0,39) 1300
set k_stiff(0,40) 1100
set k_stiff(0,41) 900
set k_stiff(0,42) 700
set k_stiff(0,43) 500
set k_stiff(0,44) 400
set k_stiff(0,45) 300
set k_stiff(0,46) 250
set k_stiff(0,47) 200
set k_stiff(0,48) 150
set k_stiff(0,49) 100
set k_stiff(0,50) 75
set k_stiff(0,51) 50
set k_stiff(0,52) 25
set k_stiff(0,53) 10
set k_stiff(0,54) 5
set k_stiff(0,55) 3
set k_stiff(0,56) 2
set k_stiff(0,57) 1
set k_stiff(0,58) .5
set k_stiff(0,59) .1
set k_stiff(0,60) 27.57
set k_stiff(0,61) 28.77
```

```
set wDL [expr -0.1/12.] ; #kips per inch
set pDL -0.443; # kips
set pLL -16.0; # kips
set st 31; #in -- starting point to the left
set space 72; #in -- space between tires
```

```

set end [expr $N_beam*$W_beam - $space-$st]; # in. -- ending point to right
for {set j 0} {$j < 62} {incr j 1} {
    set k 0
    if {[file exists dummy.out] == 1} {
        file delete dummy.out
    }

    for {set i 0} {$i < [expr $N_beam*$W_beam-$space-2*$st+3]} {incr i 4} {

        set k [expr $k+1]

        BridgeDeck $N_beam $I_beam $E $stem_center $W_beam
        $k_stiff(0,$j) 1e15 1e15 [concat $name$k] $wDL $pDL $pLL
        $st+$i $space $end

        source [concat $name$k/$name$k.tcl]

        wipe
    }

    set fileID [open number.out w]
    puts $fileID $k
    close $fileID

    set fileID [open stiffness.out w]
    puts $fileID $k_stiff(0,$j)
    close $fileID

    exec {C:\Program Files\MATLAB\R2009b\bin\matlab.exe} /r plot_it

    set dum 0
    while {$dum < 1} {
        set dum [file exists dummy.out]
    }

    wipe
}

file delete dummy.out

```


2. BridgeDeck.tcl Script:

```
proc BridgeDeck {N_beam I_beam E stem_center W_beam k_sup kV kM name wDL
pDL pLL st space end} {

#####
#####
#% This program will permit a parametric study of the internal forces
#% present in a bridge deck. The bridge deck will be modeled as a
#% continuous span beam and will permit one to modify the elastic support
#% conditions. It will also be able to modify the interface between deck
#% elements (i.e. modify spring stiffnesses) to model an imperfect shear
#% key.
#%
#% Specifically, this is written to model a double-Tee beam so that the
#% shear key is only present at the middle of ever other span. The first
#% span must be a cantilever.
#%
#% UNITS ADOPTED THROUGHOUT ARE KIPS and INCHES!!!!!!!!!!!!!!
#%
#%
#% N_beam = Number of double-Tees
#% I_beam = Moment of inertia for HALF of the Double TEE width
#% E      = Modulus of elasticity for beam material
#% stem_center = Distance between center to center of stem
#% W_beam = Width of beam measured from center to center of shear key
#% k_sup  = Stiffness of beam supports (either use 48EI/L3 for the
#%          bridge girders or use a very large stiffness to approximate
#%          fixity
#% kV     = Stiffness of shear spring at shear key
#% kM     = Stiffness of rotational spring at shear key
#%
#% September 14, 2010
#% Created by: BGN
#% Where:    Clemson University
#####
#####

#set wDL [expr 0.1/12.] ; #kips per foot
#set pDL 0.160; # kips per foot
#
#set pLL -16.0; # kips
```

```
#set st 18; #in
#set space 72; #in
```

```
file mkdir $name
set fileID [open [concat $name/$name.tcl] w]
```

```
puts $fileID "#####"
puts $fileID "#
#
puts $fileID "# Generated Automatically for the sake of a parametric #"
puts $fileID "# sensitivity study. #"
puts $fileID "# Multi-Span continuous deck for the NEXT D beam. #"
puts $fileID "#
#
puts $fileID "# Number of Double-Tees: $N_beam #"
puts $fileID "# Double-Tee width: $W_beam in. #"
puts $fileID "# Stem Spacing: $stem_center in. #"
puts $fileID "# Shear Key Stiffness: #"
puts $fileID "# Shear: $kV k/in. #"
puts $fileID "# Moment: $kM (k-in)/rad #"
puts $fileID "# Slab Support Stiffness: $k_sup k/in. #"
puts $fileID "#
#
puts $fileID "# Units: in and kips #"
puts $fileID "# Bryant Nielson #"
puts $fileID "# Auto Created: [clock format [clock seconds] -format
%D__%H:%M:%S] (time) #"
puts $fileID "#
#
puts $fileID "# Slab Model $name #"
puts $fileID "#####"
puts $fileID "#
puts $fileID {set begin [clock clicks -milliseconds]}
```

```
puts $fileID "#\n# number of dimensions"
puts $fileID "model BasicBuilder -ndm 2 -ndf 3"
puts $fileID "#\n#"
puts $fileID
"#\n#-----"
=====
```

```
puts $fileID "# NODE GENERATION"
puts $fileID
"#-----"
=====\n#"
puts $fileID "#\n# NODES FOR DECK\n#"
```

```

set div1 10.0
set div2 20.0
set L1 [expr ($W_beam-$stem_center)/2.0/$div1]
set L2 [expr ($stem_center)/$div2]

```

```

=====
#
#           DECK NODE GENERATION
#
=====

```

```

set n 0
set x 0
set m 0

```

```

for {set i 0} {$i < $N_beam} {incr i 1} {
  puts $fileID "#\n#      ID      X      Y      "
  puts $fileID "#      DOUBLE-TEE NUMBER [expr $i+1]"
  for {set j 0} {$j < $div1} {incr j 1} {
    set n [expr $n+1]
    set coord($n,0) $n
    set coord($n,1) $x
    set coord($n,2) 0.0
    set x [expr $x+$L1]
    puts $fileID [format "%-8s %3d %9.1f %9.1f " node $coord($n,0)
$coord($n,1) $coord($n,2)]
  }
}

```

```

set m [expr $m +1]
set node_sup($m,0) [expr $n+1]

```

```

for {set j 0} {$j < $div2} {incr j 1} {
  set n [expr $n+1]
  set coord($n,0) $n
  set coord($n,1) $x
  set coord($n,2) 0.0
  set x [expr $x+$L2]
  puts $fileID [format "%-8s %3d %9.1f %9.1f " node $coord($n,0)
$coord($n,1) $coord($n,2)]
}
set m [expr $m +1]
set node_sup($m,0) [expr $n+1]

```

```

    for {set j 0} {$j < $div1+1} {incr j 1} {
        set n [expr $n+1]
        set coord($n,0) $n
        set coord($n,1) $x
        set coord($n,2) 0.0
        set x [expr $x+$L1]
        puts $fileID [format "%-8s %3d %9.1f %9.1f " node $coord($n,0)
$coord($n,1) $coord($n,2)]
    }

    set x [expr $x - $L1]
    set fix($i) $n
    set key_node($i,0) [expr $n]
    set key_node($i,1) [expr $n+1]
}

set node_rng $n
#
#

puts $fileID "#####"
puts $fileID "#\n# NODES FOR SUPPORTS\n#"
puts $fileID "#####"

set n 1000
set x 0
set m 0

for {set i 0} {$i < $N_beam} {incr i 1} {
    puts $fileID "#\n#      ID      X      Y      "
    puts $fileID "#      DOUBLE-TEE NUMBER [expr $i+1]"

    set x [expr $x+$L1*$div1]
    set n [expr $n+1]
    puts $fileID [format "%-8s %3d %9.1f %9.1f " node $n $x 0.0]

    set m [expr $m +1]
    set node_sup($m,1) [expr $n]

    set x [expr $x+$L2*$div2]
    set n [expr $n+1]
    puts $fileID [format "%-8s %3d %9.1f %9.1f " node $n $x 0.0]
}

```

```

    set m [expr $m +1]
    set node_sup($m,1) [expr $n]

    set x [expr $x+$L1*$div1]
}

#
#=====
=====
#
#           NODE CONSTRAINTS
#=====
=====
#
puts $fileID
"\n\n#=====
=====
puts $fileID "#           NODE CONSTRAINTS"
puts $fileID
"#=====
=====
#
set n 1000

for {set i 0} {$i < $N_beam} {incr i 1} {

    puts $fileID "#     DOUBLE-TEE NUMBER [expr $i+1]"
    puts $fileID "#     TAG X Y MZ"
    puts $fileID [format "%-8s %3d %3d %3d %3d" fix $fix($i) 1 0 0]
    set n [expr $n+1]
    puts $fileID [format "%-8s %3d %3d %3d %3d" fix $n 1 1 1]

    set n [expr $n+1]
    puts $fileID [format "%-8s %3d %3d %3d %3d" fix $n 1 1 1]
}

puts $fileID
"#\n#=====
=====
puts $fileID "#     GENERATE MATERIAL AND ELEMENTS FOR SPRING
SUPPORTS"

```

```
puts $fileID
"#=====
=====\\n#"

```

```
puts $fileID "# Define uniaxialMaterial\\n# This material defines the response of the "
puts $fileID "#"

```

```
puts $fileID "#          tag K  "
puts $fileID "uniaxialMaterial Elastic 200 $k_sup ; # Elastic Support Stiffness (k/in)"
puts $fileID "#\\n#=====Generate
elements=====\\n#"

```

```
set n 0
```

```
for {set i 0} {$i < $N_beam} {incr i 1} {
  puts $fileID "#\\n#\\n# Elastic Support - Beam No. [expr $i+1]
#          tag i-node j-node material Y "
  set n [expr $n +1]
  puts $fileID [format "%-8s %-10s %5d %6d %6d %6s %3d %5s %4d" element
zeroLength $n $node_sup($n,0) $node_sup($n,1) -mat 200 -dir 2]
  set n [expr $n +1]
  puts $fileID [format "%-8s %-10s %5d %6d %6d %6s %3d %5s %4d" element
zeroLength $n $node_sup($n,0) $node_sup($n,1) -mat 200 -dir 2]
}

```

```
puts $fileID
"#\\n#=====
=====#"
puts $fileID "# GENERATE MATERIAL AND ELEMENTS FOR SHEAR KEY"
puts $fileID
"#=====
=====\\n#"

```

```
puts $fileID "# Define uniaxialMaterial\\n# This material defines the response of the
shear key "
puts $fileID "#"
puts $fileID "#          tag K  "
puts $fileID "uniaxialMaterial Elastic 201 $kV ; # Shear stiffness (k/in)"

```

```

puts $fileID "uniaxialMaterial Elastic 202 $kM ;# Rotational stiffness (k-in)/rad"
puts $fileID "#\n#====Generate
elements====\n#"

set n 200

for {set i 0} {$i < [expr $N_beam-1]} {incr i 1} {
    set n [expr $n +1]
    puts $fileID "#\n#\n# Shear Key No. [expr $i+1]
# tag i-node j-node material Y Mz "
    puts $fileID [format "%-8s %-10s %5d %6d %6d %6s %3d %3d %5s %4d
%4d" element zeroLength $n $key_node($i,0) $key_node($i,1) -mat 201 202 -dir 2 6]
}

puts $fileID
"#\n#\n#====
===="
puts $fileID "# GENERATE BEAM ELEMENTS FOR DECK BEAM"
puts $fileID
"#====
====\n#\n#"
puts $fileID "# TAG "
puts $fileID "geomTransf Linear 1\n"

set A 1e10
set strip_width 12.0
set t 8.0
set Iz [expr $strip_width*$t*$t*$t/12.]

set n 0
set m 1000

for {set i 0} {$i < $N_beam} {incr i 1} {

    puts $fileID "\n#\n# DOUBLE-TEE NUMBER [expr $i+1]"
    puts $fileID "#\n# ID iNode jNode Area E Iz
TransTag "
    for {set j 0} {$j < $div1} {incr j 1} {
        set n [expr $n+1]
        set m [expr $m+1]
        puts $fileID [format "%-8s %-18s %4d %3d %3d %3.1e %9.1f %9.1f
%2d" element elasticBeamColumn $m $n [expr $n +1] $A $E $Iz 1]
    }
}

```

```

    }

    for {set j 0} {$j < $div2} {incr j 1} {
        set n [expr $n+1]
        set m [expr $m+1]
        puts $fileID [format "%-8s %-18s %4d %3d   %3d   %3.1e %9.1f %9.1f
%2d" element elasticBeamColumn $m $n [expr $n + 1] $A $E $Iz 1]
    }

    for {set j 0} {$j < $div1} {incr j 1} {
        set n [expr $n+1]
        set m [expr $m+1]
        puts $fileID [format "%-8s %-18s %4d %3d   %3d   %3.1e %9.1f %9.1f
%2d" element elasticBeamColumn $m $n [expr $n + 1] $A $E $Iz 1]
    }

    set n [expr $n+1]
}

puts $fileID
"#\n#\n#-----"
=====
puts $fileID "#          END OF MODEL GENERATION"
puts $fileID
"#-----"
=====
      \n#"

puts $fileID
"#\n#-----"
=====
puts $fileID "#          DEFINE RECORDERS"
puts $fileID
"#-----"
=====
      \n#"
#
puts $fileID " recorder Element -file [concat $name/beam.out] -eleRange 1001 [expr
$n-1+1000] localForce"
puts $fileID " recorder Element -file [concat $name/shear_key.out] -eleRange 201
[expr 200+($N_beam-1)] force"

```



```

puts $fileID " recorder Node -file [concat $name/beam_def.out] -nodeRange 1
$node_rng -dof 2 disp"

puts $fileID
"#\n#=====  

=====  

puts $fileID "#          DEFINE GRAVITY LOADS"  

puts $fileID  

"#=====  

=====\n#"

#
puts $fileID "# Define and assign loads due to a distributed dead load (wDL)"
puts $fileID "# Dead load of bridge deck is wDL = $wDL (k/in) or wDL = [expr
$wDL*12] (k/ft)\n\n"

set pDL1 [expr -$wDL*$L1]
set pDL2 [expr -$wDL*$L2]

puts $fileID "#\npattern Plain 1 \"Linear\" {"

set n 0

for {set i 0} {$i < $N_beam} {incr i 1} {
  puts $fileID "# DOUBLE-TEE NUMBER [expr $i+1]"
  puts $fileID "#\n#          ID X Y          "  

  for {set j 0} {$j < $div1} {incr j 1} {
    set n [expr $n+1]
    if {$j == 0} {
      puts $fileID [format " %-6s %4d %4.1f %5.3e %4.1f" load $n
0.0 [expr $pDL1/2.0 -$pDL] 0.0 ]
    } else {
      puts $fileID [format " %-6s %4d %4.1f %5.3e %4.1f" load $n
0.0 $pDL1 0.0 ]
    }
  }
}

for {set j 0} {$j < $div2} {incr j 1} {
  set n [expr $n+1]

```

```

        if {$j == 0} {
#         if {$j == 0 || $j == [expr $div2-1]} {}

                puts $fileID [format "%-6s %4d %4.1f %5.3e %4.1f" load $n
0.0 [expr $pDL1/2.0 + $pDL2/2.0] 0.0 ]
                } else {
                puts $fileID [format "%-6s %4d %4.1f %5.3e %4.1f" load $n
0.0 $pDL2 0.0 ]
                }
        }

        for {set j 0} {$j < $div1+1} {incr j 1} {
                set n [expr $n+1]
                if {$j == 0} {
                        puts $fileID [format "%-6s %4d %4.1f %5.3e %4.1f" load $n
0.0 [expr $pDL1/2.0 + $pDL2/2.0] 0.0 ]
                } elseif {$j == $div1} {
                        puts $fileID [format "%-6s %4d %4.1f %5.3e %4.1f" load $n
0.0 [expr $pDL1/2.0-$pDL] 0.0 ]
                } else {
                        puts $fileID [format "%-6s %4d %4.1f %5.3e %4.1f" load $n
0.0 $pDL1 0.0 ]
                }
        }
}
puts $fileID "}"

for {set i 1} {$i < $node_rng} {incr i 1} {
        if {$scoord($i,1) < $st && $scoord([expr $i+1],1) >= $st} {
                set load_st $scoord([expr $i],0)

        }

        if {$scoord($i,1) < [expr $st+$space] && $scoord([expr $i+1],1) >= [expr $st +
$space]} {
                set load_end $scoord([expr $i],0)
        }
}

puts $fileID "#\npattern Plain 2 \"Linear\" {"
        puts $fileID "#\n#          ID X      Y      "

```

```

        puts $fileID [format "   %-6s %4d %4.1f %5.3e %4.1f" load $load_st 0.0
[expr $pLL] 0.0 ]
        puts $fileID [format "   %-6s %4d %4.1f %5.3e %4.1f" load $load_end 0.0
[expr $pLL] 0.0 ]
puts $fileID "}"

puts $fileID
"#\n#====="
====="
puts $fileID "#          START OF ANALYSIS GENERATION FOR GRAVITY
ANALYSIS"
puts $fileID
"#====="
=====\n#"
puts $fileID "# Create the convergence test"
puts $fileID "test NormDispIncr 1.0e-8  50  "

puts $fileID "#\nalgorithm Newton\n#\nintegrator LoadControl 1. 1 1. 1."
puts $fileID "#\nnumberer  RCM\n#\nconstraints Plain\n#\nanalysis Static"
puts $fileID
"#\n#====="
====="
puts $fileID "#          PERFORM GRAVITY LOAD ANALYSIS"
puts $fileID
"#====="
=====\n#"
puts $fileID "analyze 1"

puts $fileID {puts "#####"}
puts $fileID {puts "Gravity Analysis Complete"}
puts $fileID {puts "#####"}

puts $fileID {set endt [clock clicks -milliseconds]}
puts $fileID {set totaltime [expr ($endt-$begin)]}
puts $fileID {set totaltimem [expr ($endt-$begin)/60000.0]}
puts $fileID " "
puts $fileID {puts "Time in hours: [expr $totaltimem/60.]"}
puts $fileID {puts "$totaltimem is the total time in minutes"}

close $fileID
}

```

Appendix G

MATLAB Script

```
clear
close all

num = load('number.out');
stiff = load('stiffness.out');
beamwidth = load('beamwidth.out');

div1=10.0;
div2=20.0;

N_beam = 8;
width = beamwidth;
stem = 36;
d1 = (width - stem)/div1/2;
d2 = stem/div2;

for ii = 1:num

a = load(strcat('Bridge',num2str(ii),'beam_def.out'));
b = load(strcat('Bridge',num2str(ii),'beam.out'));
c = load(strcat('Bridge',num2str(ii),'shear_key.out'));

s=0;
k = 0;
kk = 0;
xx = 0;

for i = 1:N_beam
    for j = 1:div1
        s = s + 1;
        x(s) = xx;
        xx = xx + d1;
    end
    k = k+1;
    xs(k) = xx;

    for j = 1:div2
        s = s+1;
        x(s) = xx;
        xx = xx + d2;
```

```

end

k = k+1;
xs(k) = xx;

for j = 1:div1+1
    s = s+1;
    x(s) = xx;
    xx = xx + d1;
end
xx = xx - d1;
kk = kk + 1;
xk(kk) = xx;
end

for i = 1:N_beam-1
    V_k(i) = -c((i-1)*6 + 5);
    M_k(i) = -c((i-1)*6 + 3);
end

s = s+1;
xx_1=0;
kk = 0;

for i = 1:N_beam
    for j = 1:div1
        kk = kk+1;
        s = s + 1;
        V(s) = b((kk-1)*6 + 2);
        M(s) = -b((kk-1)*6 + 3);
        xr(s) = xx_1;
        xx_1 = xx_1 + d1;
        s = s + 1;
        V(s) = -b((kk-1)*6 + 5);
        M(s) = b((kk-1)*6 + 6);
        xr(s) = xx_1;
    end

    for j = 1:div2
        kk = kk+1;
        s = s + 1;
        V(s) = b((kk-1)*6 + 2);
        M(s) = -b((kk-1)*6 + 3);
        xr(s) = xx_1;
    end
end

```

```

    xx_1 = xx_1 + d2;
    s = s + 1;
    V(s) = -b((kk-1)*6 + 5);
    M(s) = b((kk-1)*6 + 6);
    xr(s) = xx_1;
end

```

```

for j = 1:div1
    kk = kk+1;
    s = s + 1;
    V(s) = b((kk-1)*6 + 2);
    M(s) = -b((kk-1)*6 + 3);
    xr(s) = xx_1;
    xx_1 = xx_1 + d1;
    s = s + 1;
    V(s) = -b((kk-1)*6 + 5);
    M(s) = b((kk-1)*6 + 6);
    xr(s) = xx_1;
end

```

end

%Scans for 6 max responses and their positions and writes these responses to 12 separate arrays

```

Mmax(ii) = max(M);
index1 = find(M==Mmax(ii)); %Finding index of maximum moment
position1(ii) = xr(index1(1)); %Using this index to find position, x, and saving it to
position1 array

```

```

Mmin(ii) = min(M);
index2 = find(M==Mmin(ii)); %Finding index of maximum negative moment
position2(ii) = xr(index2(1)); %Using this index to find position, x, and saving it to
position2 array

```

```

Vmax(ii) = max(abs(min(V)),max(V));
index3 = find(abs(V)==Vmax(ii)); %Finding index of maximum shear
position3(ii) = xr(index3(1)); %Using this index to find position, x, and saving it to
position3 array

```

```

Mmax_sk(ii) = max(M_k);
index4 = find(M_k==Mmax_sk(ii)); %Finding index of maximum moment in shear key
position4(ii) = beamwidth*index4(1); %Using this index to find position, x, and saving it
to position4 array

```

```

Mmin_sk(ii) = min(M_k);
index5 = find(M_k==Mmin_sk(ii)); %Finding index of maximum negative moment in
shear key
position5(ii) = beamwidth*index5(1); %Using this index to find position, x, and saving it
to position5 array

```

```

Vmax_sk(ii) = max(abs(min(V_k)),max(V_k));
index6 = find(abs(V_k)==Vmax_sk(ii)); %Finding index of maximum shear in shear key
position6(ii) = beamwidth*index6(1); %Using this index to find position, x, and saving it
to position6 array

```

```

end

```

```

% Scans each set for maximum response

```

```

M_max = max(Mmax);
index11 = find(Mmax==M_max); % Finding index of final max moment
x_M_max = position1(index11(1)); % Searching for index in position1 array and
returning the x-position

```

```

M_min = min(Mmin);
index22 = find(Mmin==M_min); % Finding index of final max negative moment
x_M_min = position2(index22(1)); % Searching for index in position2 array and
returning the x-position

```

```

V_max = max(Vmax);
index33 = find(Vmax==V_max); % Finding index of final max shear
x_V_max = position3(index33(1)); % Searching for index in position3 array and
returning the x-position

```

```

M_max_sk = max(Mmax_sk);
index44 = find(Mmax_sk==M_max_sk); % Finding index of final max moment in shear
key
x_M_max_sk = position4(index44(1)); % Searching for index in position4 array and
returning the x-position

```

```

M_min_sk = min(Mmin_sk);
index55 = find(Mmin_sk==M_min_sk); % Finding index of final max negative moment
in shear key
x_M_min_sk = position5(index55(1)); % Searching for index in position5 array and
returning the x-position

```

```

V_max_sk = max(Vmax_sk);
index66 = find(Vmax_sk==V_max_sk); % Finding index of final max shear in shear key

```

```

x_V_max_sk = position6(index66(1)); % Searching for index in position6 array and
returning the x-position

%Writes maximum responses to one final file and associates the set of
%maximum responses with the stiffness value used to determine them
if exist('Output.out') == 0
    fid=fopen('Output.out','a+');
    fprintf(fid,'Stiffness  M max   x pos  M min   x pos  V max   x pos  Mmax sk  x
pos  Mmin sk  x pos  Vmax sk  x pos \n');
    fprintf(fid,'(k/in)   (k-in)  (in)   (k-in)  (in)   (kip)  (in)   (k-in)  (in)   (k-in)
(in)   (kip)  (in)   \n');

    fprintf(fid,'=====
=====
=====');
    fprintf(fid,'%4.3e ',stiff);
    fprintf(fid,'%6.2f ',M_max);
    fprintf(fid,'%3.0f ',x_M_max);
    fprintf(fid,'%6.2f ',M_min);
    fprintf(fid,'%3.0f ',x_M_min);
    fprintf(fid,'%6.2f ',V_max);
    fprintf(fid,'%3.0f ',x_V_max);
    fprintf(fid,'%6.2f ',M_max_sk);
    fprintf(fid,'%3.0f ',x_M_max_sk);
    fprintf(fid,'%6.2f ',M_min_sk);
    fprintf(fid,'%3.0f ',x_M_min_sk);
    fprintf(fid,'%6.2f ',V_max_sk);
    fprintf(fid,'%3.0f\n',x_V_max_sk);
else
    fid=fopen('Output.out','a+');
    fprintf(fid,'%4.3e ',stiff);
    fprintf(fid,'%6.2f ',M_max);
    fprintf(fid,'%3.0f ',x_M_max);
    fprintf(fid,'%6.2f ',M_min);
    fprintf(fid,'%3.0f ',x_M_min);
    fprintf(fid,'%6.2f ',V_max);
    fprintf(fid,'%3.0f ',x_V_max);
    fprintf(fid,'%6.2f ',M_max_sk);
    fprintf(fid,'%3.0f ',x_M_max_sk);
    fprintf(fid,'%6.2f ',M_min_sk);
    fprintf(fid,'%3.0f ',x_M_min_sk);
    fprintf(fid,'%6.2f ',V_max_sk);
    fprintf(fid,'%3.0f\n',x_V_max_sk);
end

```



```
fclose(fid);
```

```
% Creates dummy file to slow down simulation.tcl
```

```
fid=fopen('dummy.out','w+');
```

```
fprintf(fid,'dummy');
```

```
fclose(fid);
```

```
% Exits MATLAB to keep from multiple MATLAB programs opening in loops
```

```
exit
```

Appendix H

Sensitivity Study Plots

Figure H.1: Max Positive Moment vs. Support Stiffness (NEXT D8)

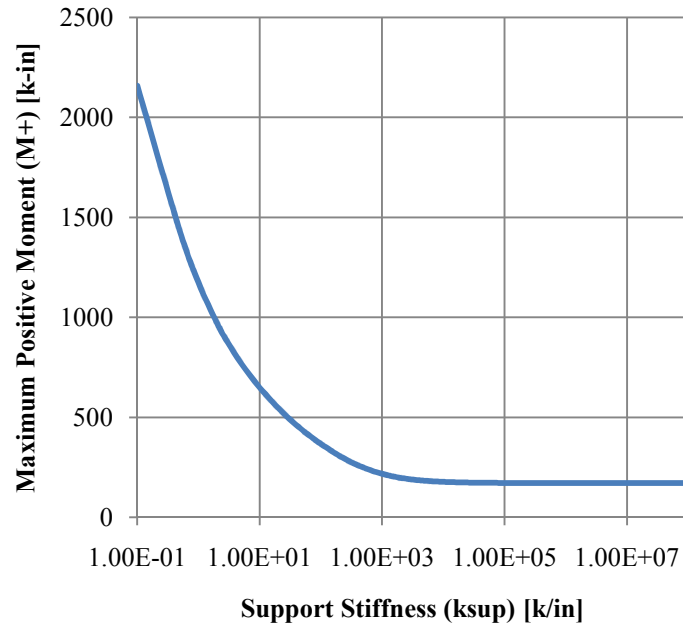


Figure H.2: Max Negative Moment vs. Support Stiffness (NEXT D8)

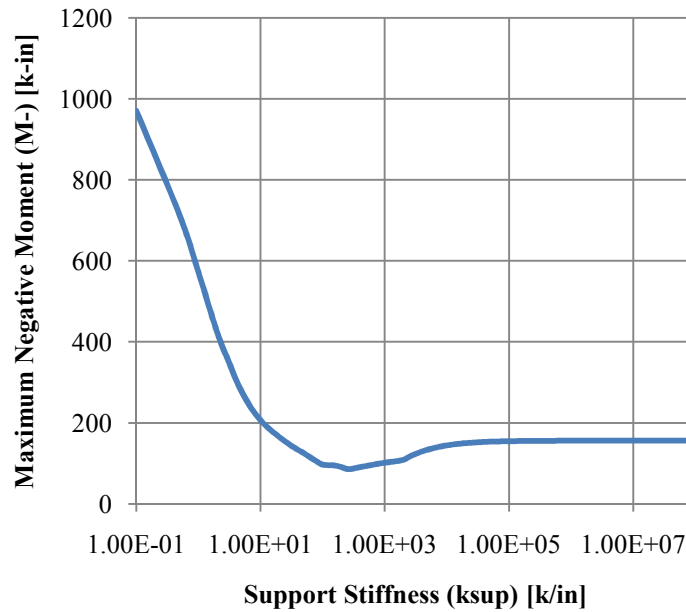


Figure H.3: Max Positive Moment (Shear Key) vs. Support Stiffness (NEXT D8)

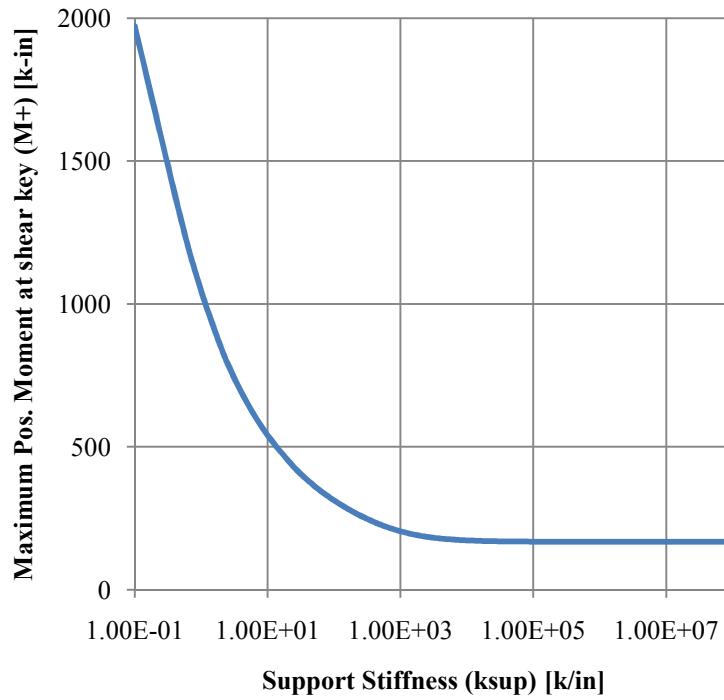


Figure H.4: Max Negative Moment (Shear Key) vs. Support Stiffness (NEXT D8)

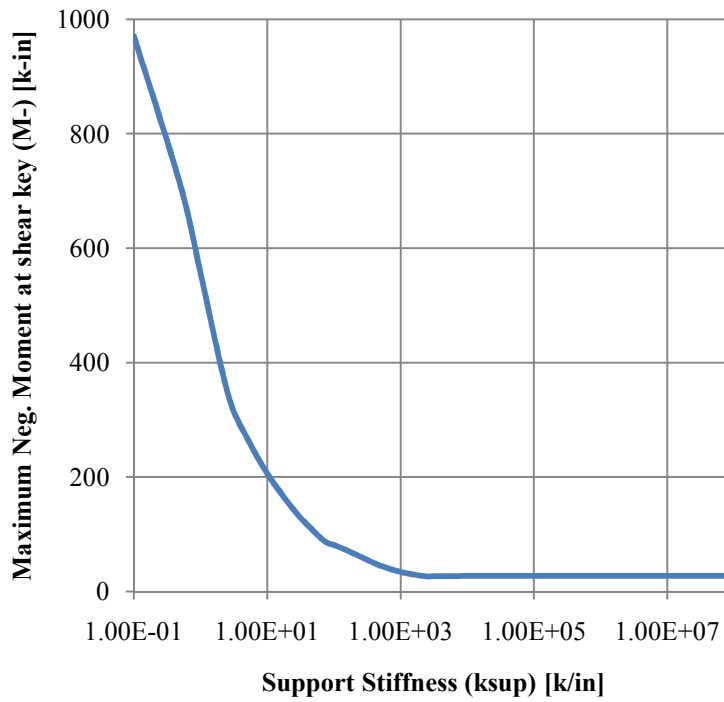


Figure H.5: Max Shear vs. Support Stiffness (NEXT D8)

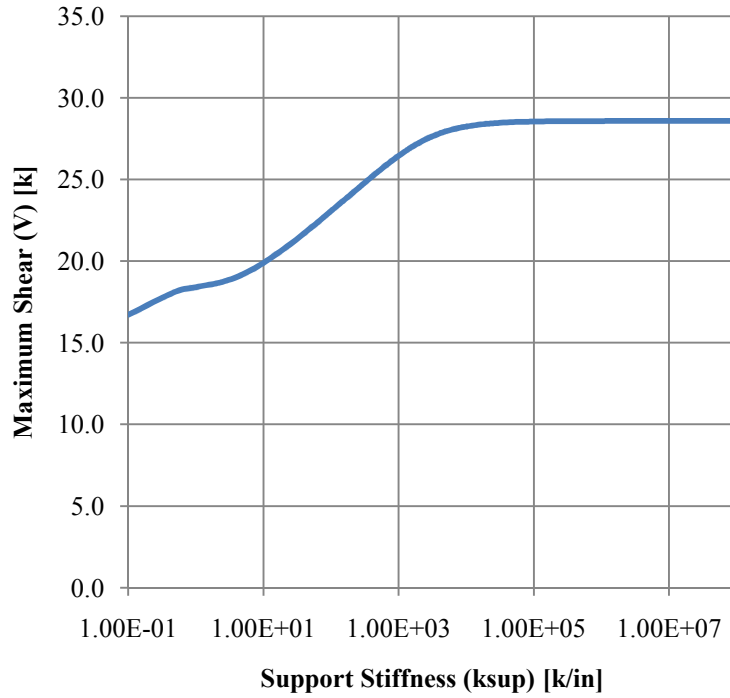


Figure H.6: Max Shear (Shear Key) vs. Support Stiffness (NEXT D8)

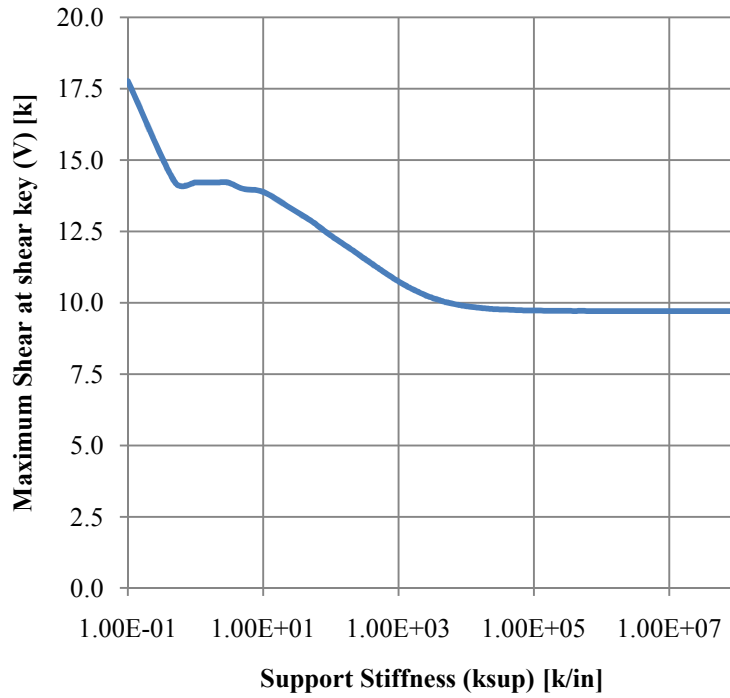


Figure H.7: Max Positive Moment vs. Key Rotational Stiffness (NEXT D8)

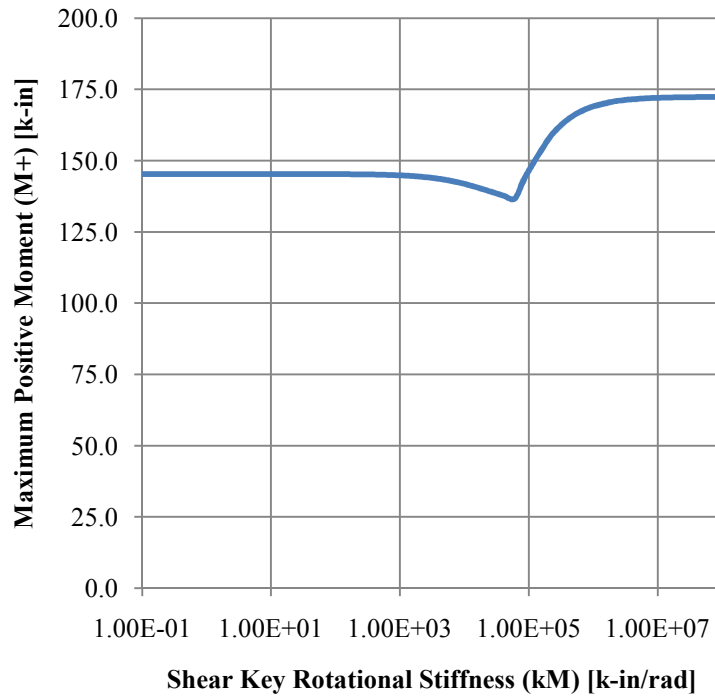


Figure H.8: Max Negative Moment vs. Key Rotational Stiffness (NEXT D8)

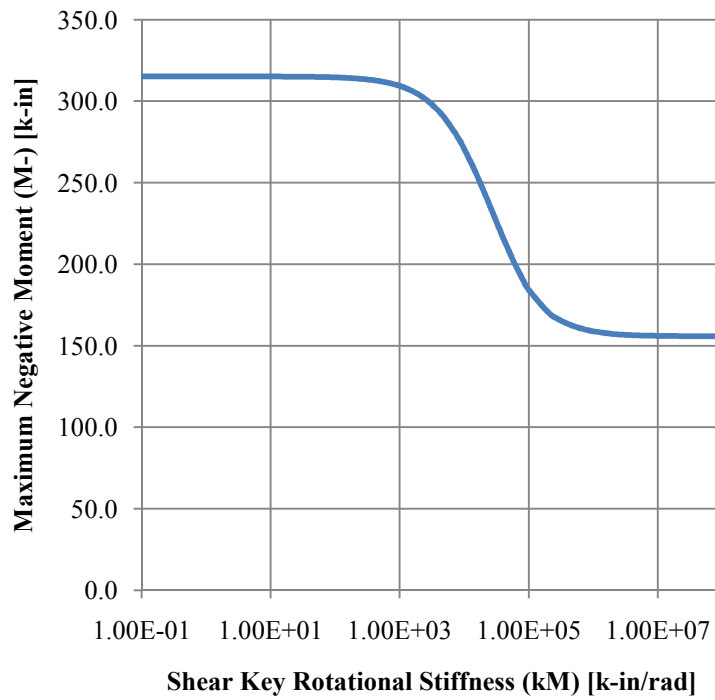


Figure H.9: Max Pos. Moment (Shear Key) vs. Key Rotational Stiffness (NEXT D8)

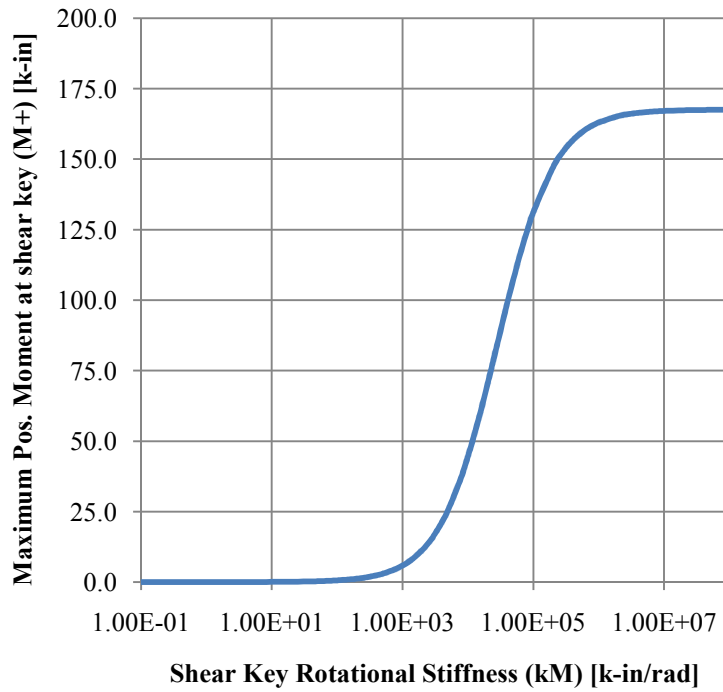


Figure H.10: Max Neg. Moment (Shear Key) vs. Key Rotational Stiffness (NEXT D8)

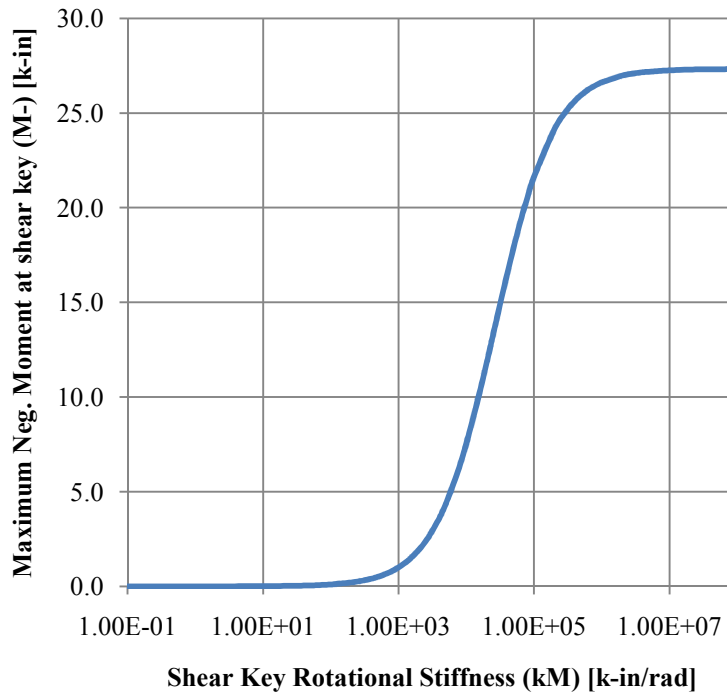


Figure H.11: Max Shear vs. Key Rotational Stiffness (NEXT D8)

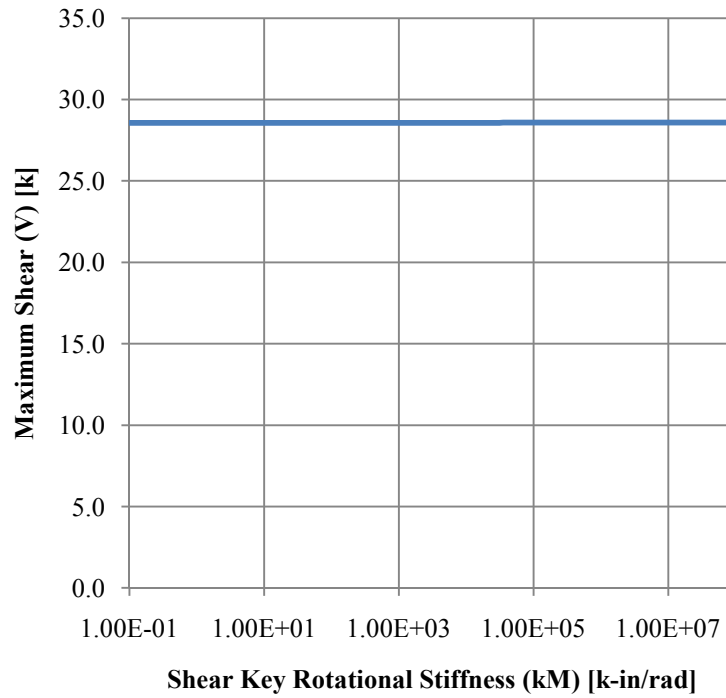


Figure H.12: Max Shear (Shear Key) vs. Key Rotational Stiffness (NEXT D8)

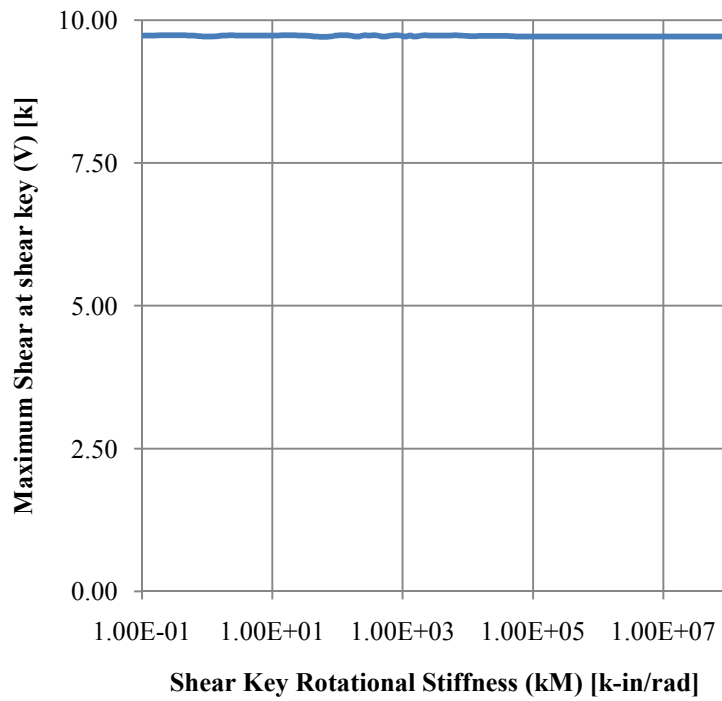


Figure H.13: Max Positive Moment vs. Key Translational Stiffness (NEXT D8)

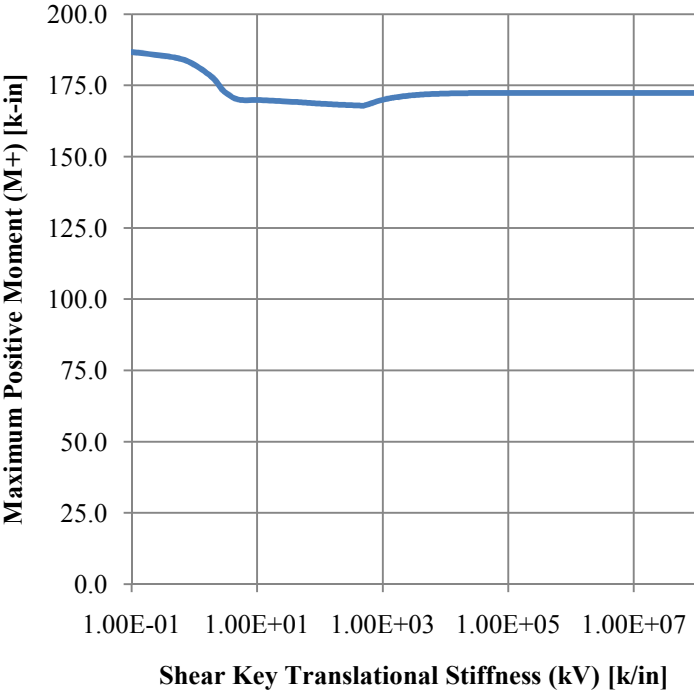


Figure H.14: Max Negative Moment vs. Key Translational Stiffness (NEXT D8)

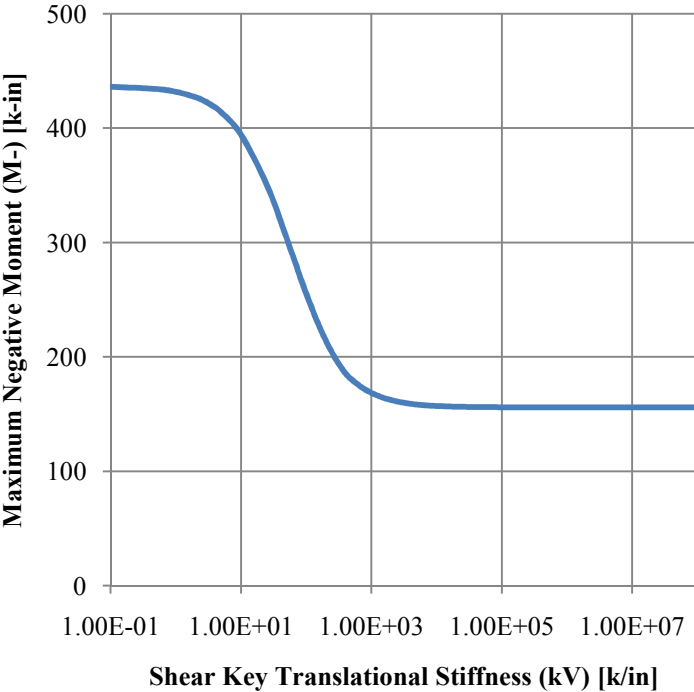


Figure H.15: Max Pos. Moment (Shear Key) vs. Key Translational Stiffness (NEXT D8)

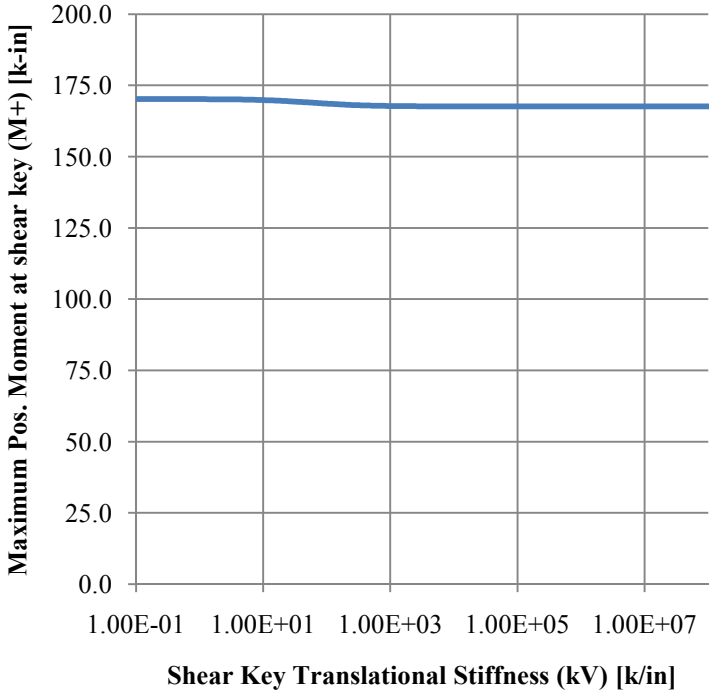


Figure H.16: Max Neg. Moment (Shear Key) vs. Key Translational Stiffness (NEXT D8)

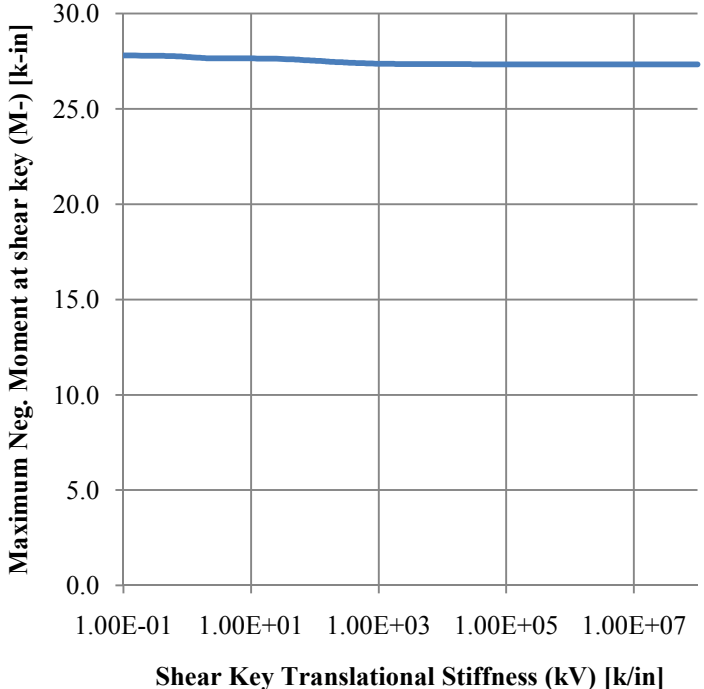


Figure H.17: Max Shear vs. Key Translational Stiffness (NEXT D8)

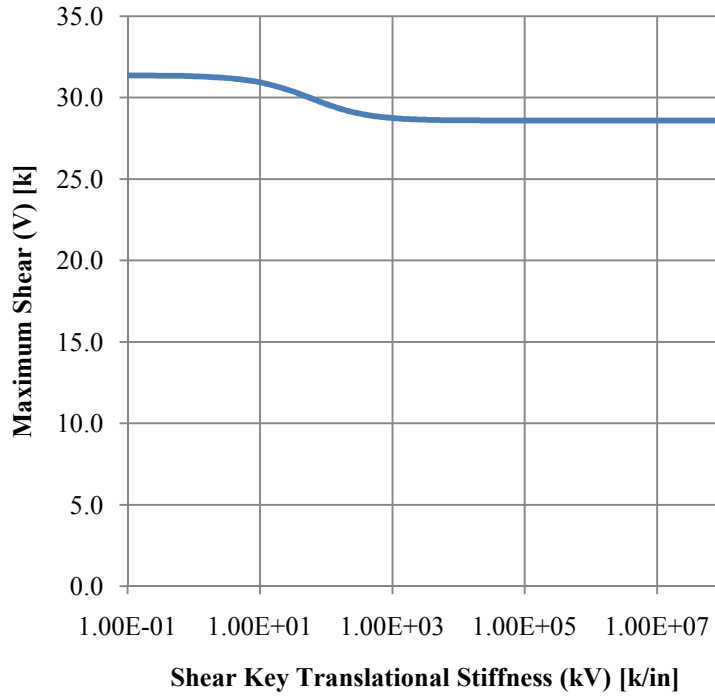


Figure H.18: Max Shear (Shear Key) vs. Key Translational Stiffness (NEXT D8)

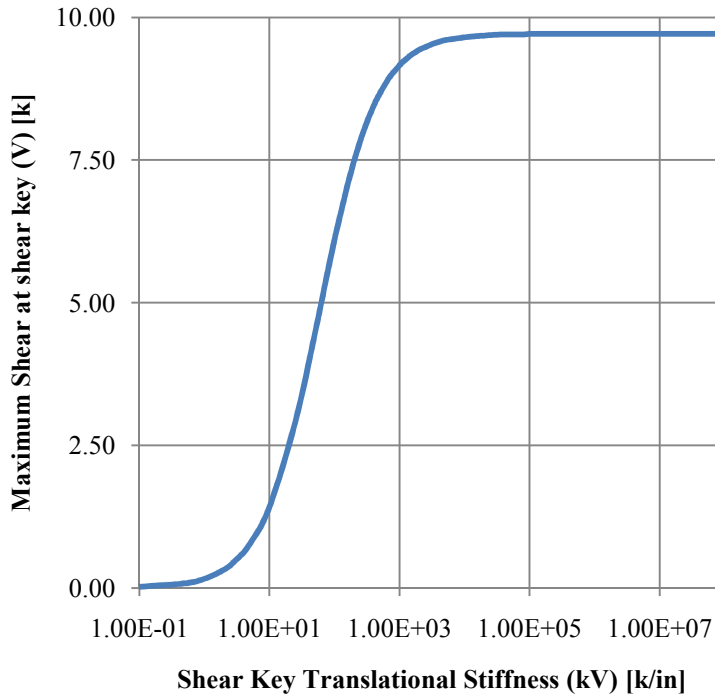


Figure H.19: Max Positive Moment vs. Shear Key Stiffness (NEXT D8)

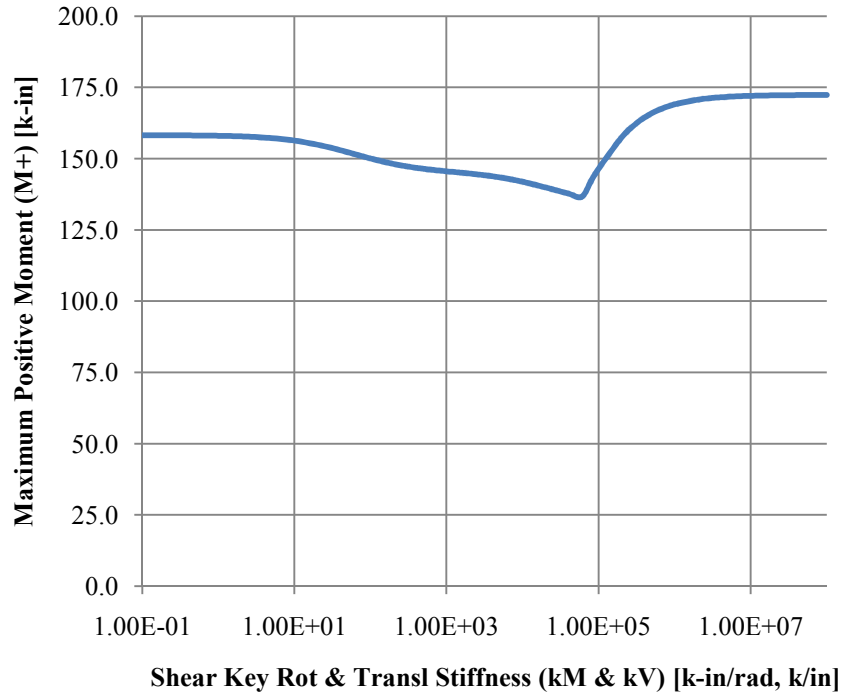


Figure H.20: Max Negative Moment vs. Shear Key Stiffness (NEXT D8)

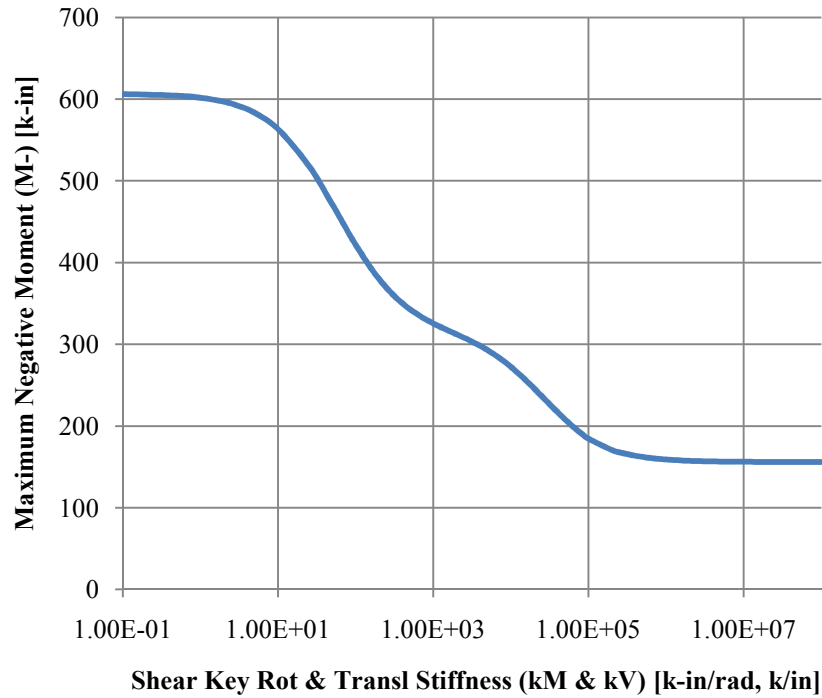


Figure H.21: Max Pos. Moment (Shear Key) vs. Shear Key Stiffness (NEXT D8)

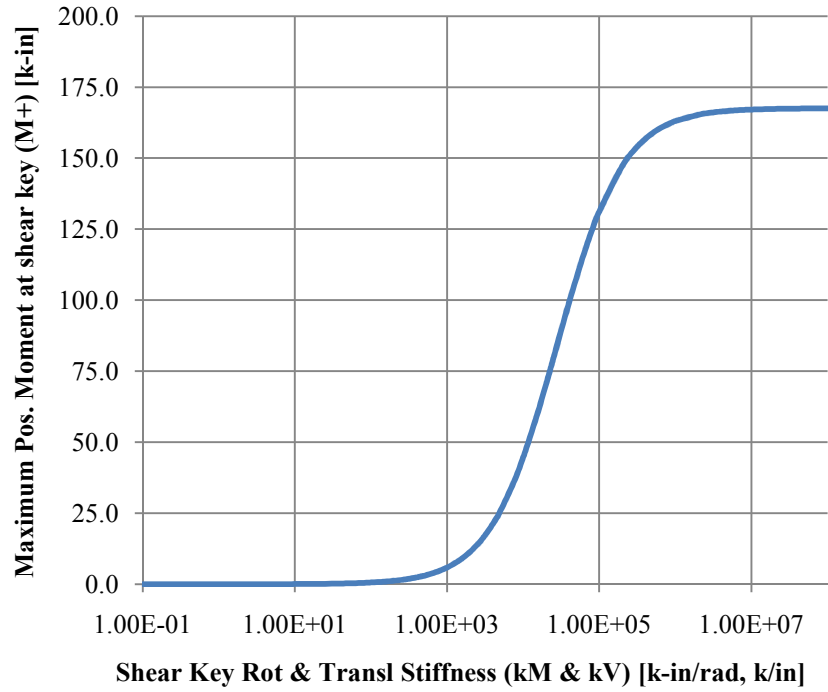


Figure H.22: Max Neg. Moment (Shear Key) vs. Shear Key Stiffness (NEXT D8)

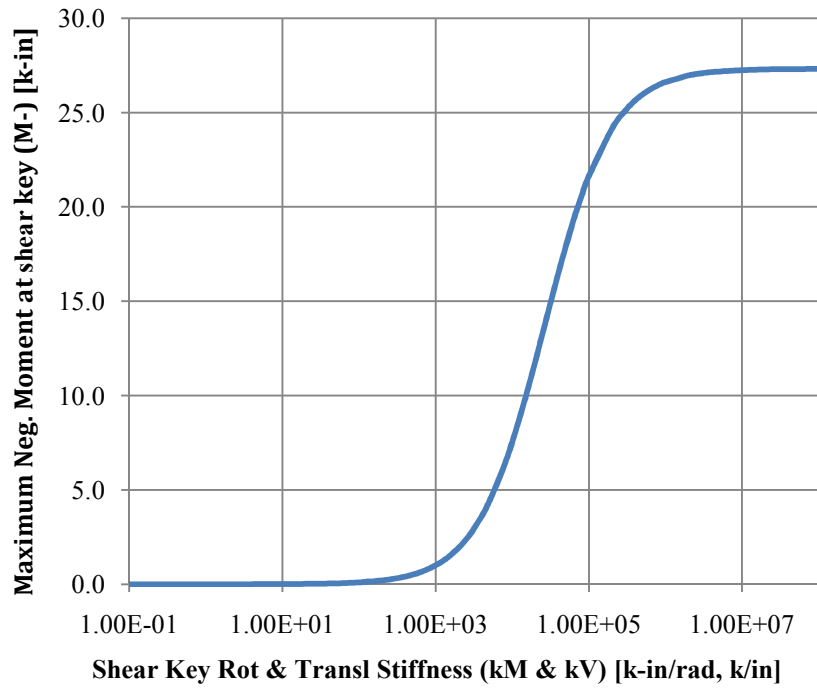


Figure H.23: Max Shear vs. Shear Key Stiffness (NEXT D8)

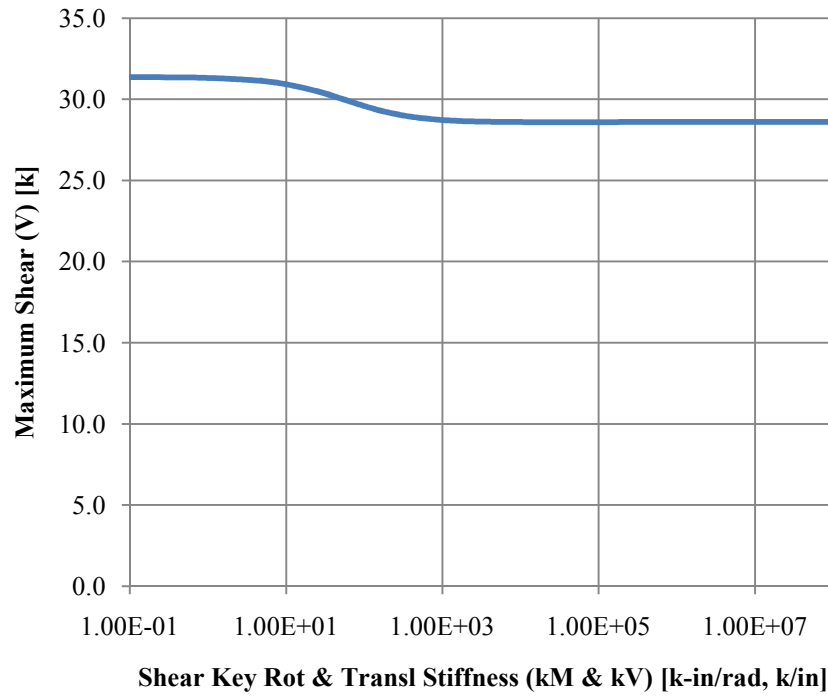


Figure H.24: Max Shear (Shear Key) vs. Shear Key Stiffness (NEXT D8)

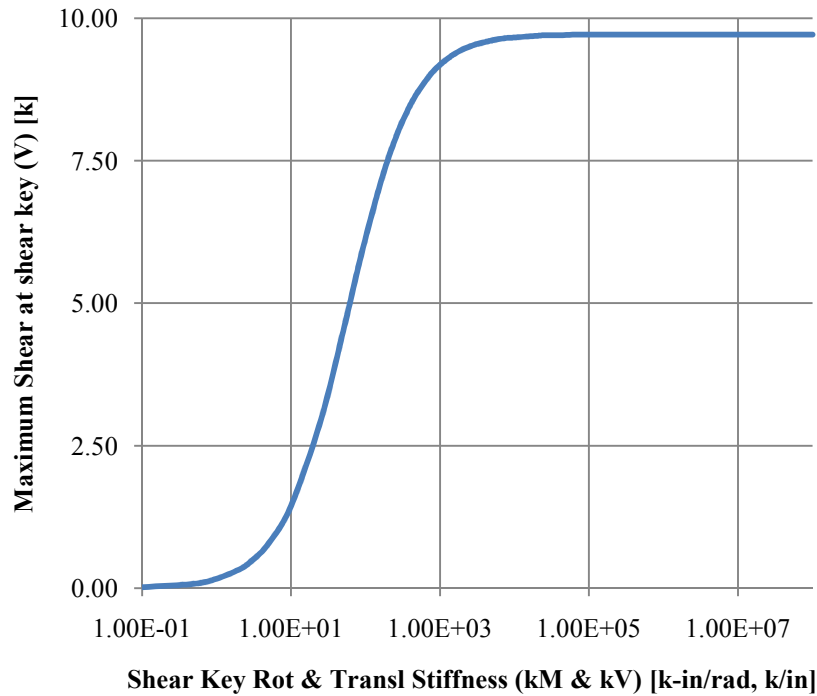


Figure H.25: Max Positive Moment vs. Support Stiffness (NEXT D6)

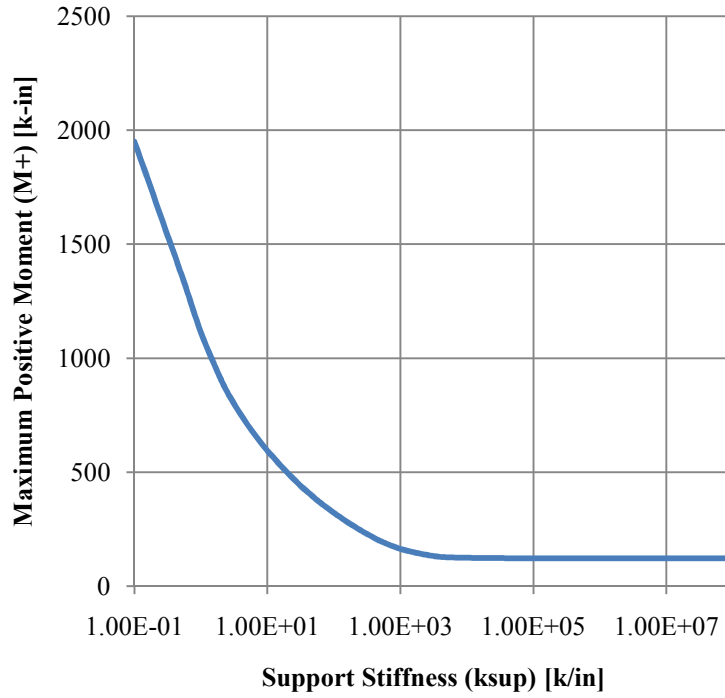


Figure H.26: Max Negative Moment vs. Support Stiffness (NEXT D6)

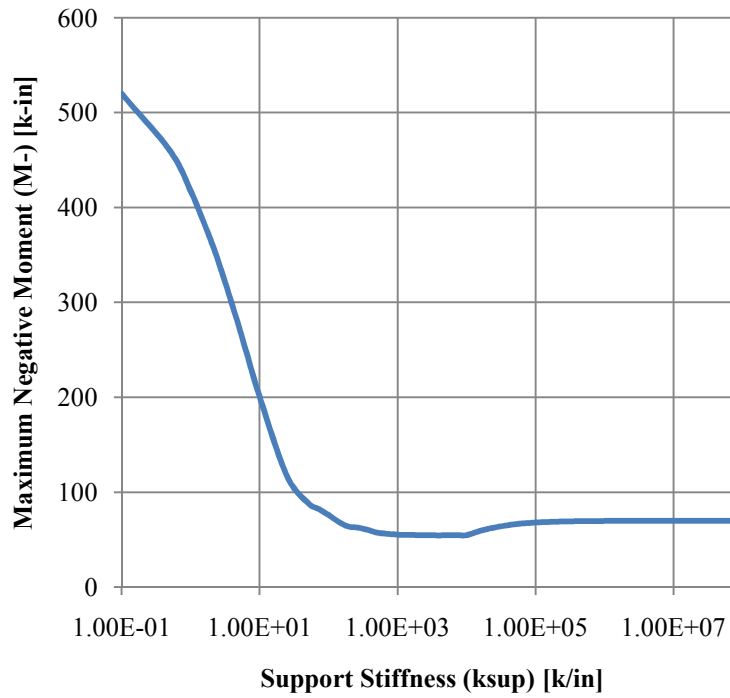


Figure H.27: Max Positive Moment (Shear Key) vs. Support Stiffness (NEXT D6)

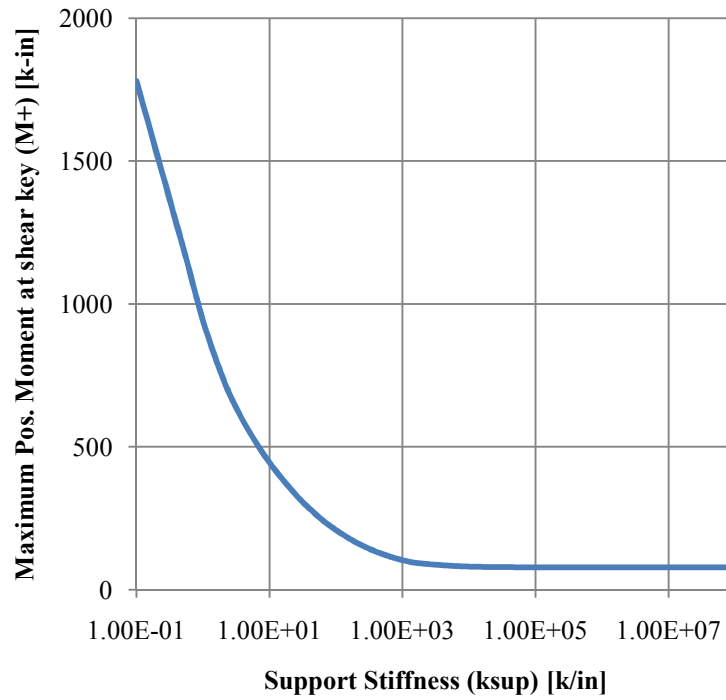


Figure H.28: Max Negative Moment (Shear Key) vs. Support Stiffness (NEXT D6)

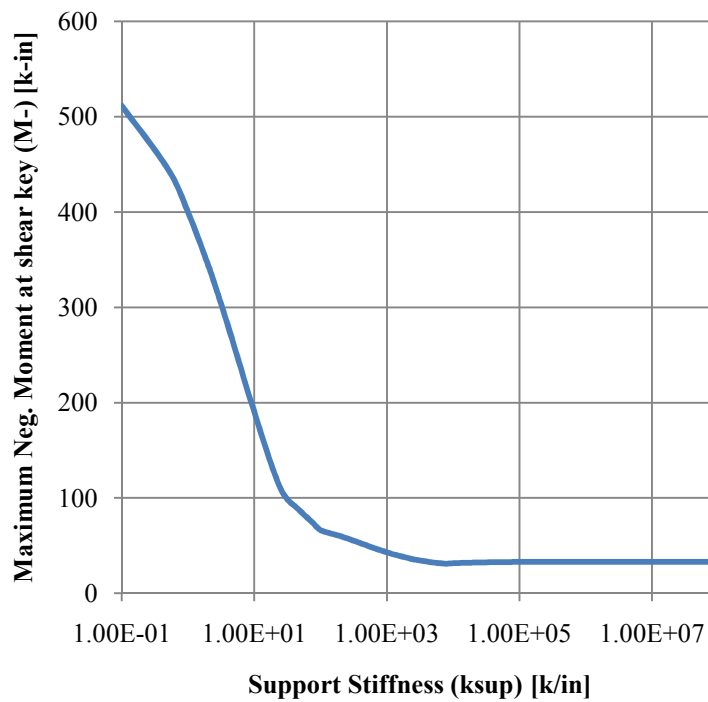


Figure H.29: Max Shear vs. Support Stiffness (NEXT D6)

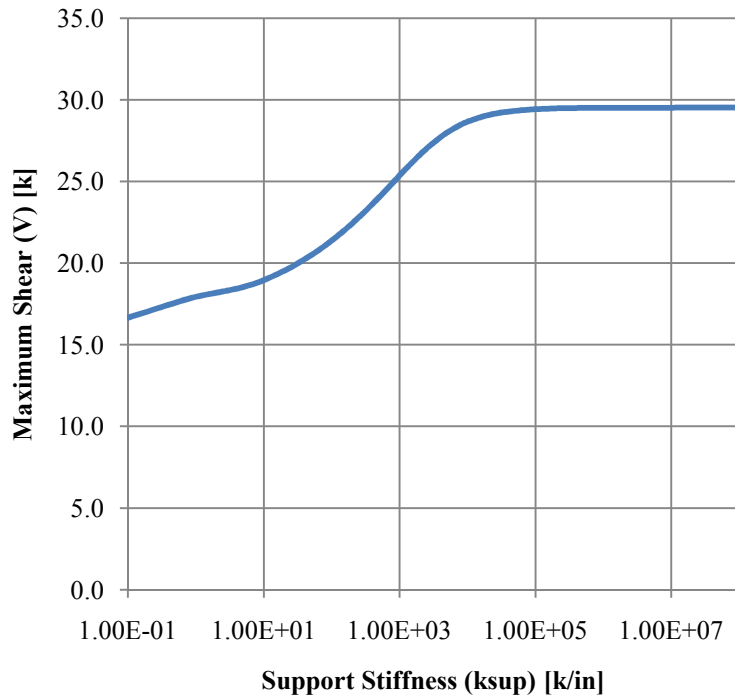


Figure H.30: Max Shear (Shear Key) vs. Support Stiffness (NEXT D6)

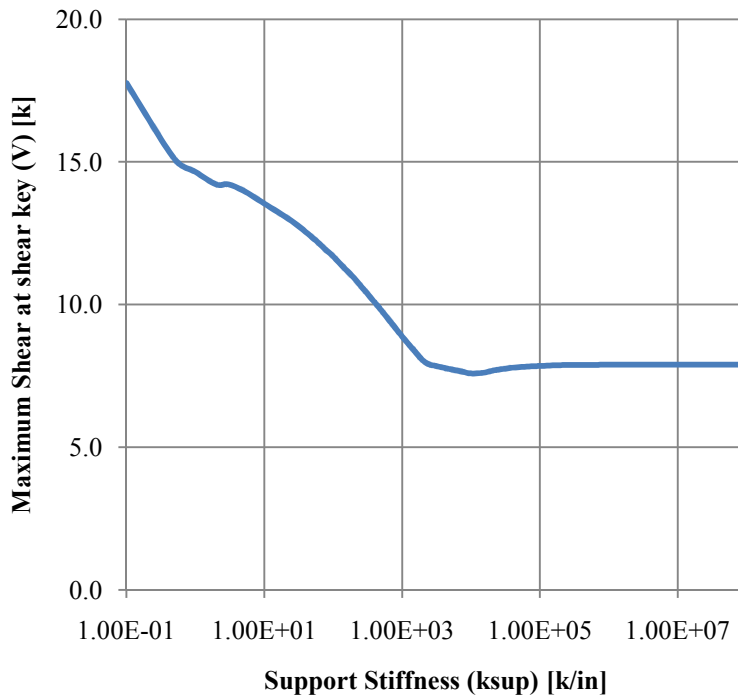


Figure H.31: Max Positive Moment vs. Key Rotational Stiffness (NEXT D6)

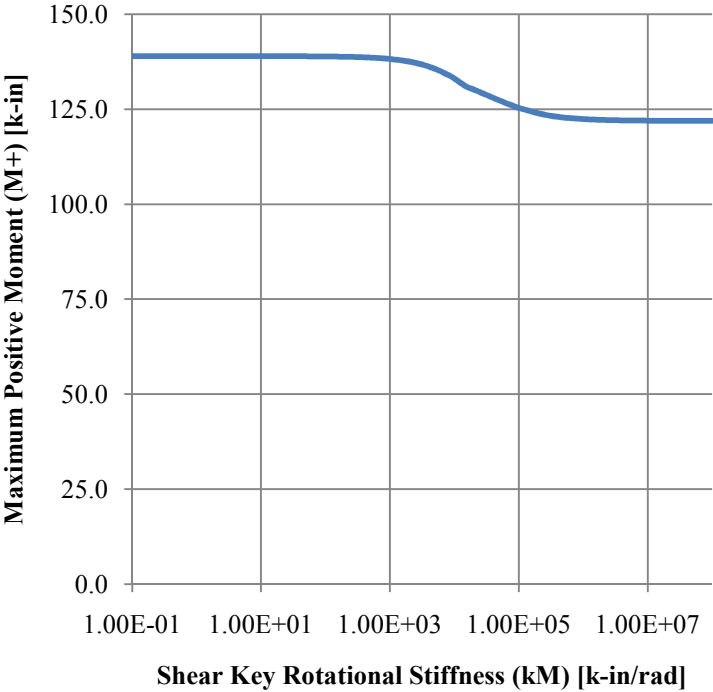


Figure H.32: Max Negative Moment vs. Key Rotational Stiffness (NEXT D6)

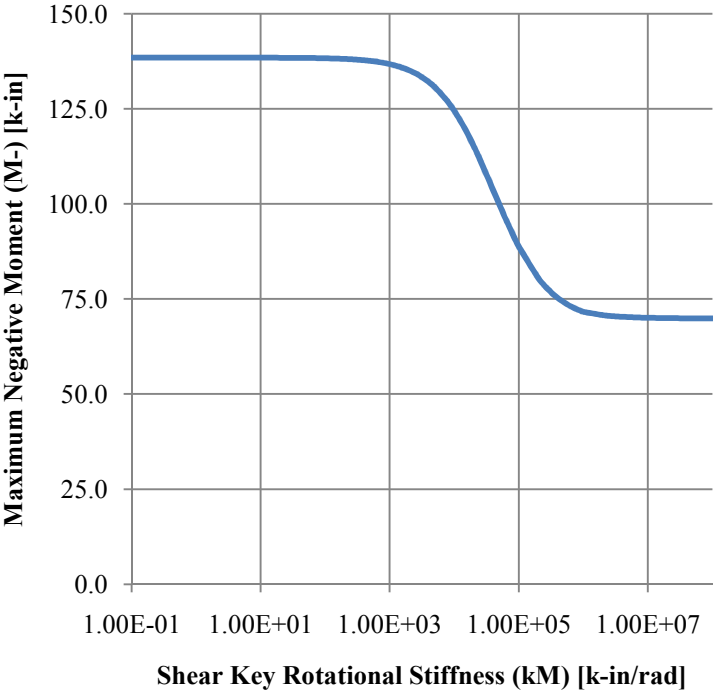


Figure H.33: Max Pos. Moment (Shear Key) vs. Key Rotational Stiffness (NEXT D6)

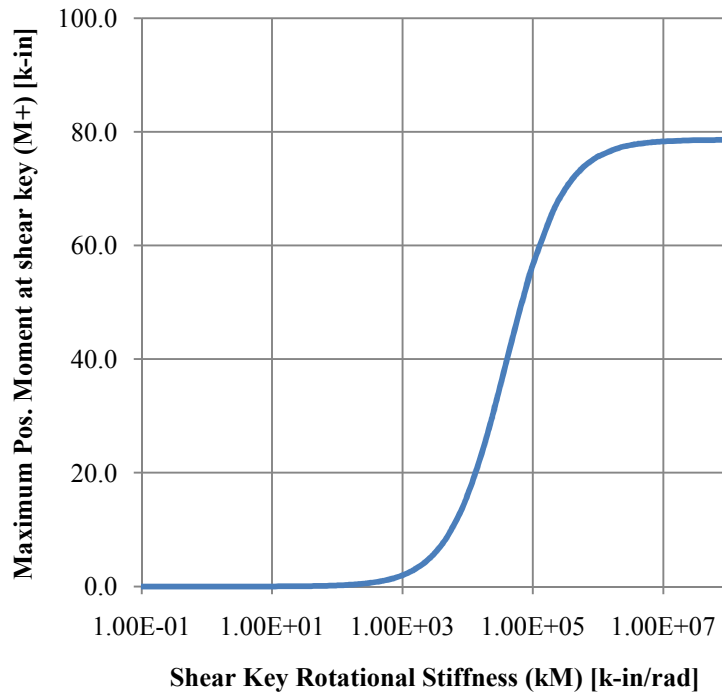


Figure H.34: Max Neg. Moment (Shear Key) vs. Key Rotational Stiffness (NEXT D6)

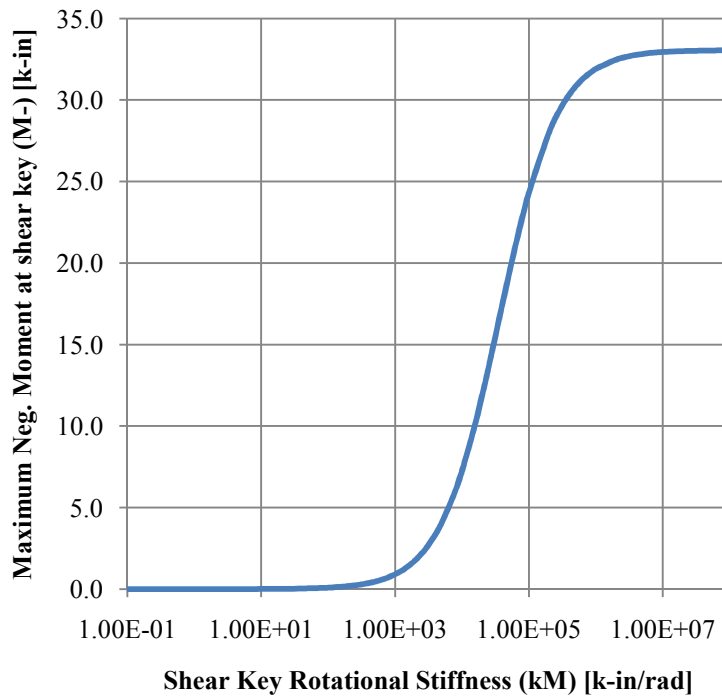


Figure H.35: Max Shear vs. Key Rotational Stiffness (NEXT D6)

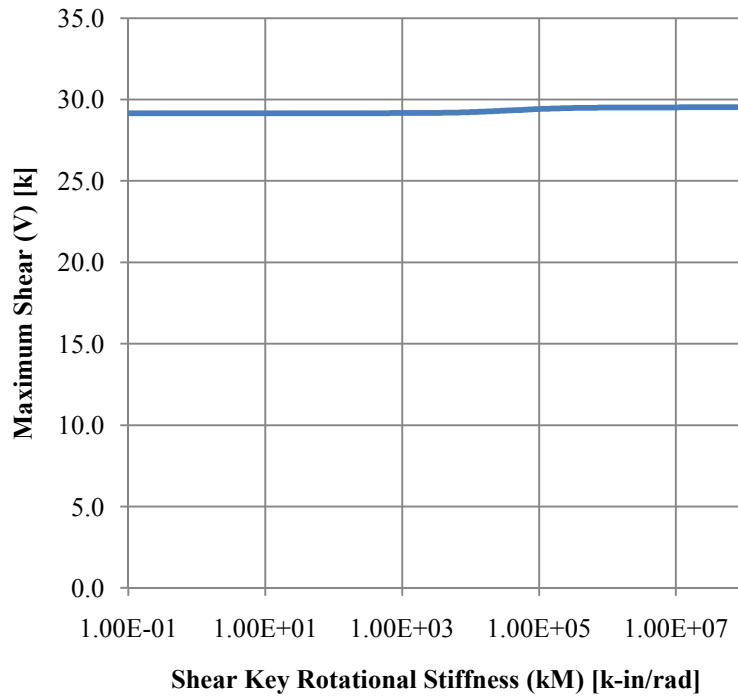


Figure H.36: Max Shear (Shear Key) vs. Key Rotational Stiffness (NEXT D6)

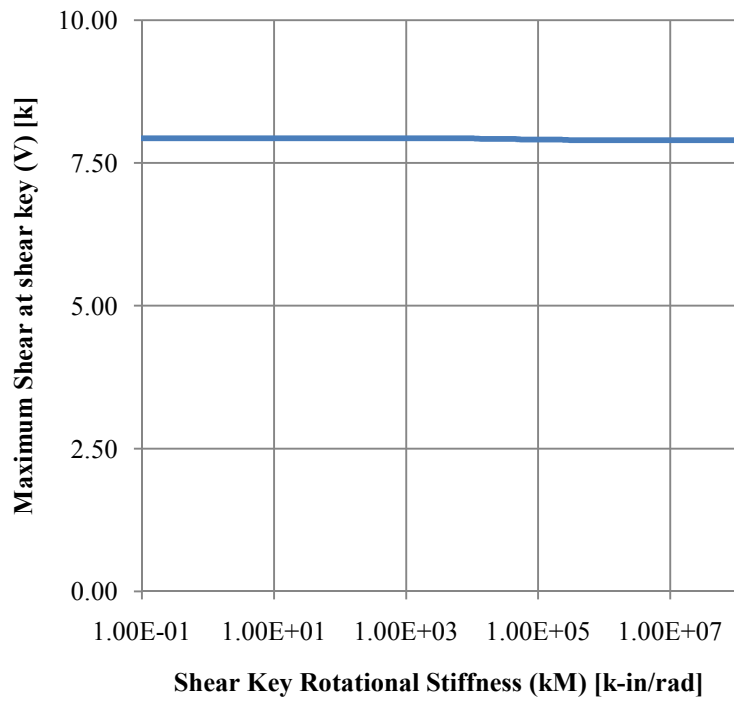


Figure H.37: Max Positive Moment vs. Key Translational Stiffness (NEXT D6)

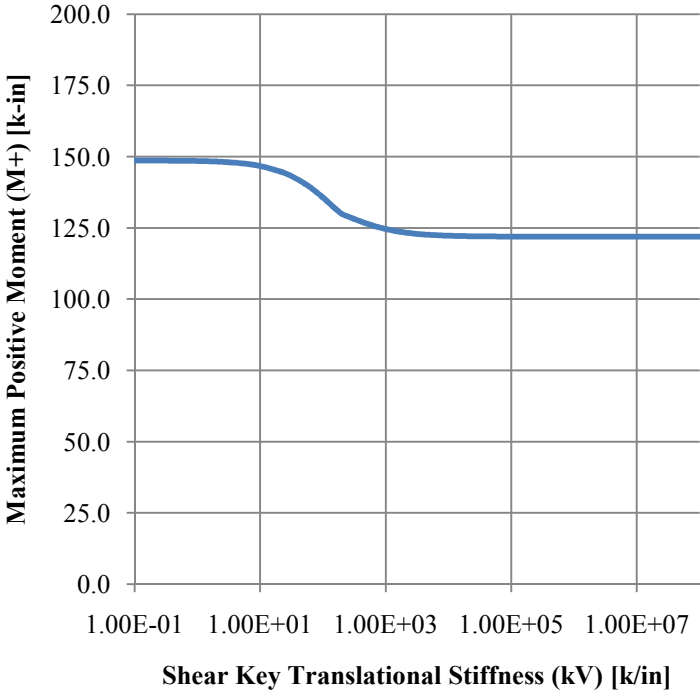


Figure H.38: Max Negative Moment vs. Key Translational Stiffness (NEXT D6)

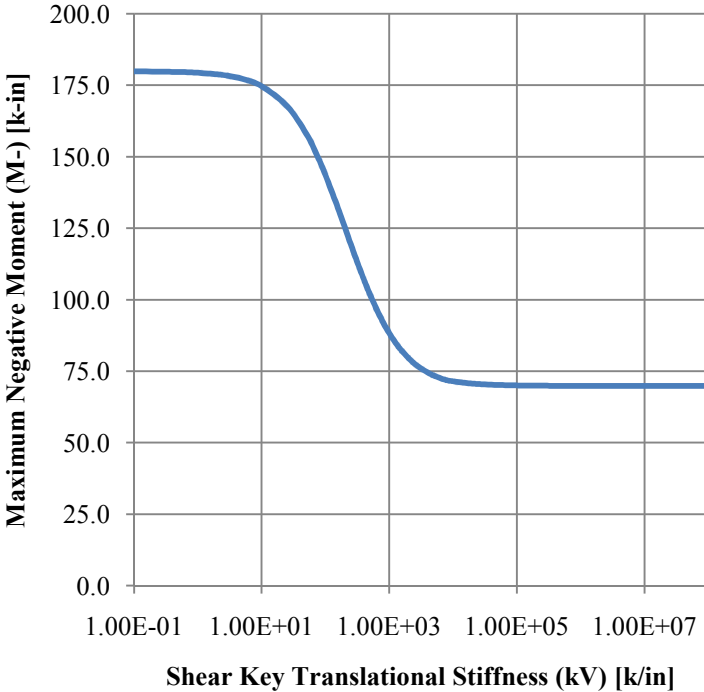


Figure H.39: Max Pos. Moment (Shear Key) vs. Key Translational Stiffness (NEXT D6)

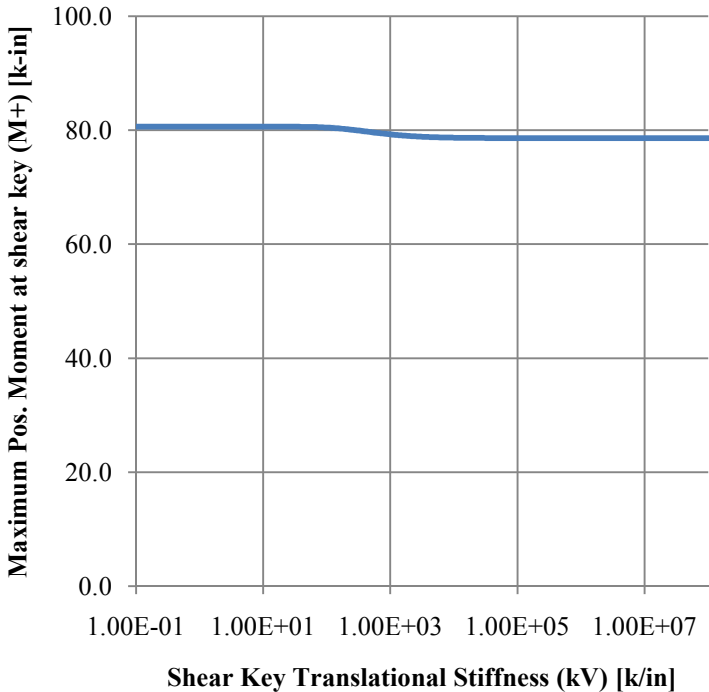


Figure H.40: Max Neg. Moment (Shear Key) vs. Key Translational Stiffness (NEXT D6)

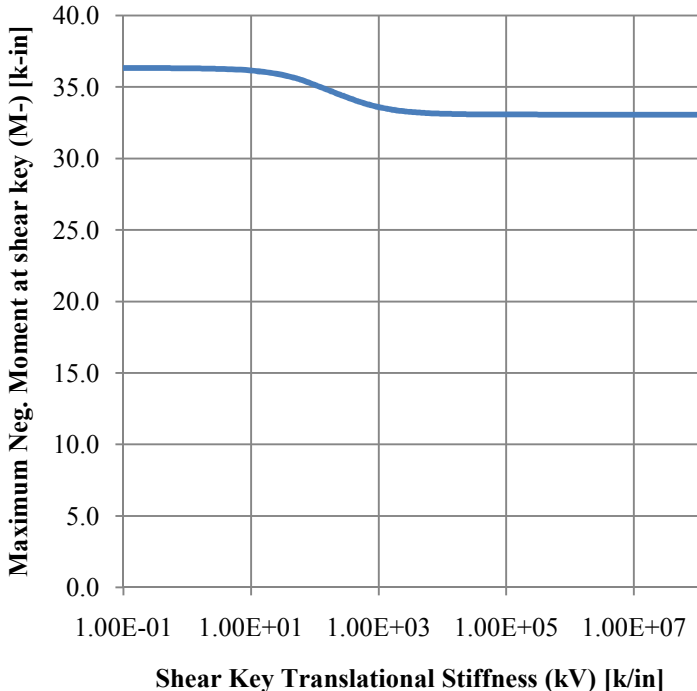


Figure H.41: Max Shear vs. Key Translational Stiffness (NEXT D6)

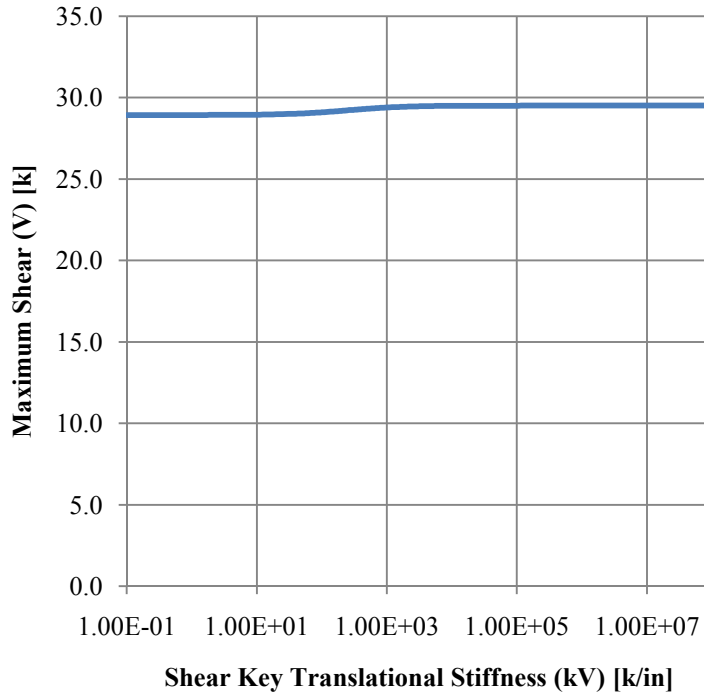


Figure H.42: Max Shear (Shear Key) vs. Key Translational Stiffness (NEXT D6)

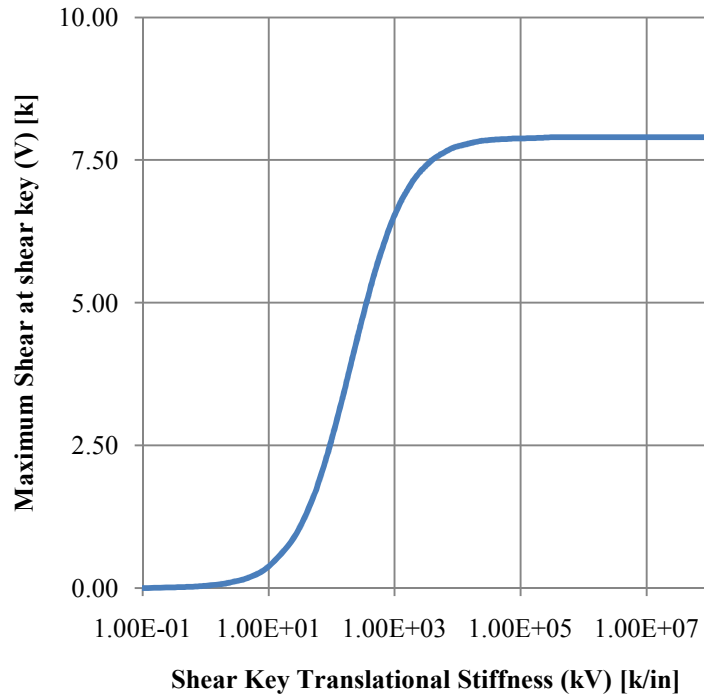


Figure H.43: Max Positive Moment vs. Shear Key Stiffness (NEXT D6)

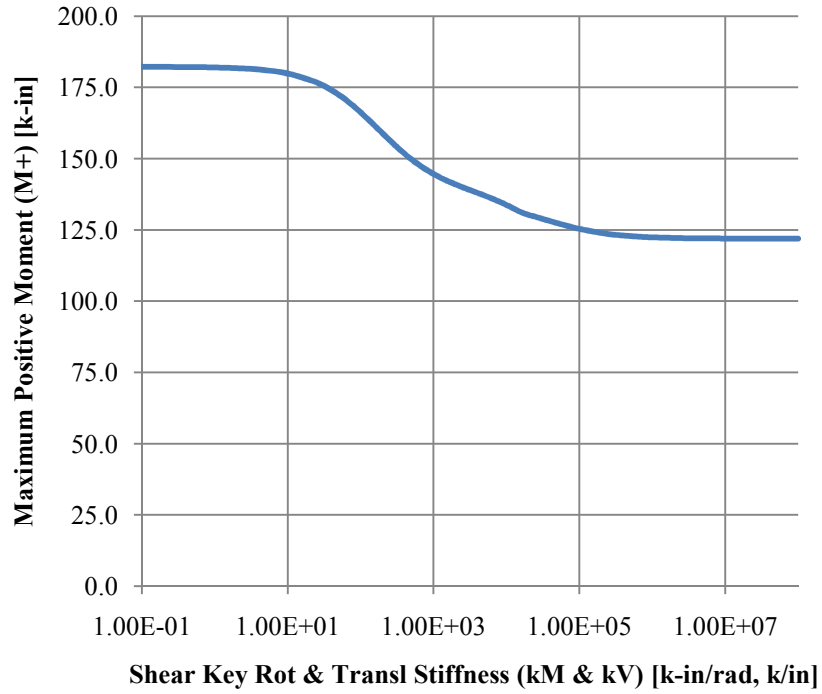


Figure H.44: Max Negative Moment vs. Shear Key Stiffness (NEXT D6)

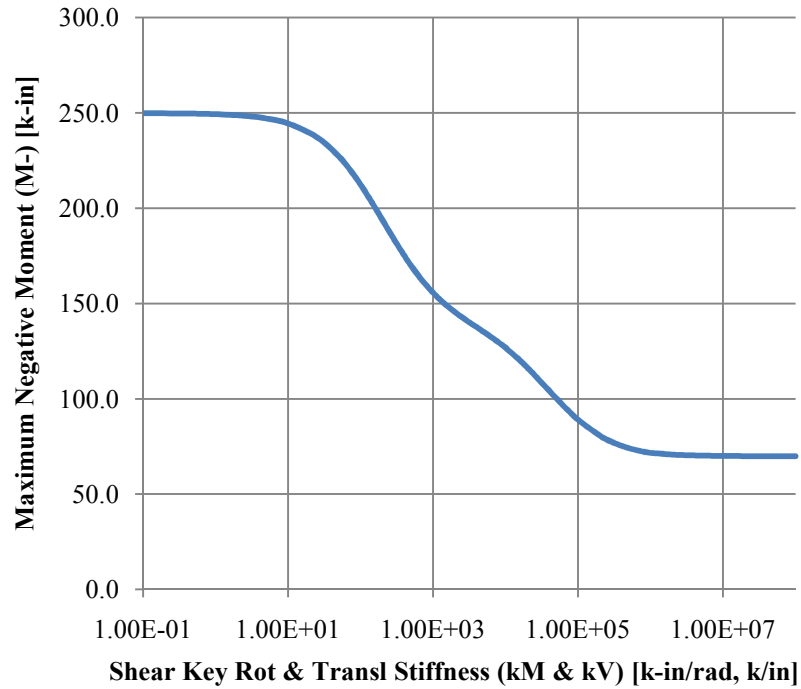


Figure H.45: Max Pos. Moment (Shear Key) vs. Shear Key Stiffness (NEXT D6)

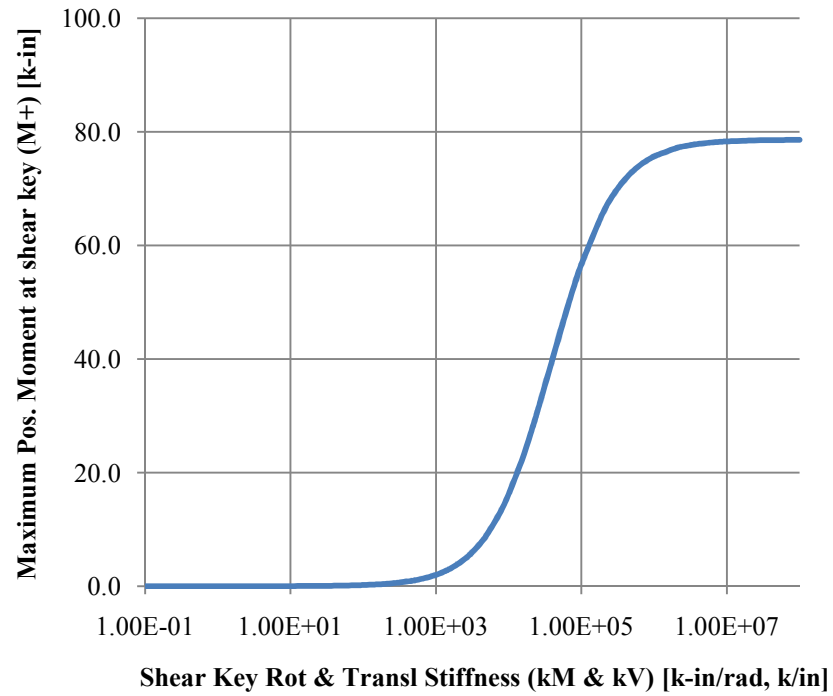


Figure H.46: Max Neg. Moment (Shear Key) vs. Shear Key Stiffness (NEXT D6)

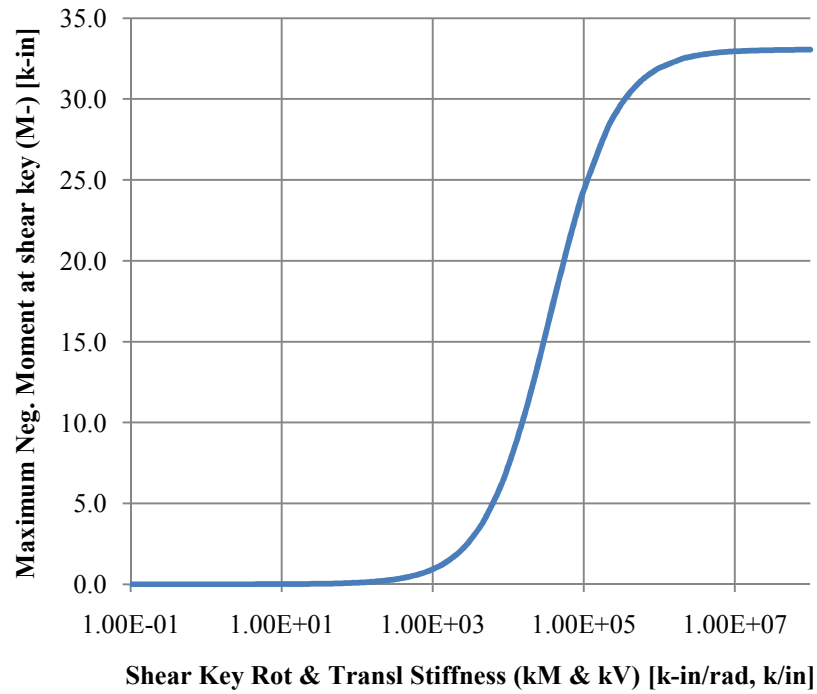


Figure H.47: Max Shear vs. Shear Key Stiffness (NEXT D6)

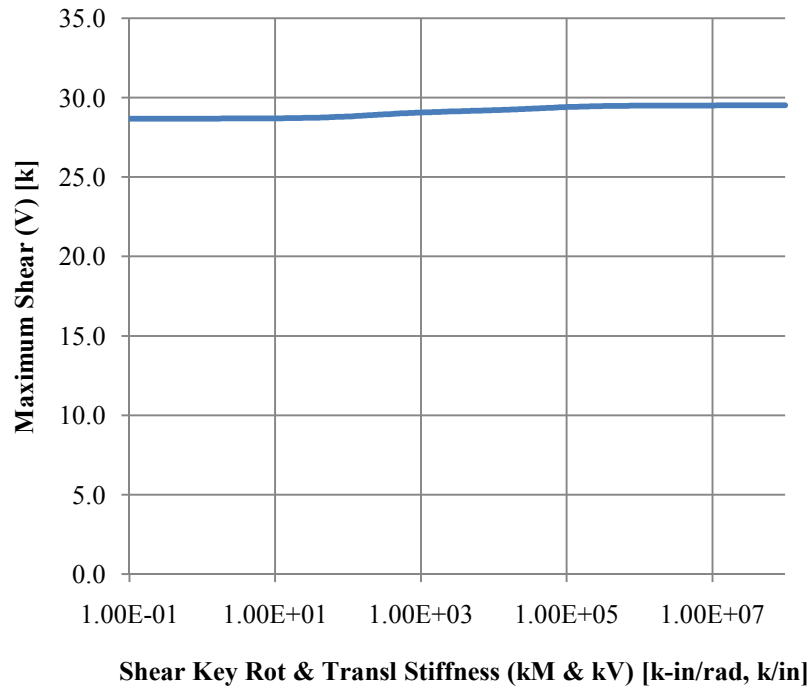


Figure H.48: Max Shear (Shear Key) vs. Shear Key Stiffness (NEXT D6)

