NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP Report 407

Rapid Replacement of Bridge Decks

Transportation Research Board
National Research Council

REPORT DOCUMENTATION PAGE		Form Approved OMB No. 0704-0188	
Public reporting burden for this collection of information is estimated to average 1 hour response, including the time for reviewing instructions, searching existing data sources, gathering and maintaining the data needed, and completing and reviewing the collection of information. Send comments regarding this burden estimate or any other aspect of this collection of information, including suggestions for reducing this burden, to Washington Headquarters Services, Directorate for Information Operations and Reports, 1215 Jefferson Davis Highway, Suite 1204, Arlington, VA 22202-4302, and to the Office of Management and Budget, Paperwork reduction Project (0704-0188), Washington, DC 20503			
1. AGENCY USE ONLY (Leave blank)	2. REPORT DATE 1998	3. REPORT TYPE AND DATES COVERED Final Report	
4. TITLE AND SUBTITLE NCHRP Report 407: Rapid Replacement of Bridge Decks			5. FUNDING NUMBERS C12-41
6. AUTHOR(S): Maher K. Tadros and Mantu C. Baishya		PB99-126807	
7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES) Center for Infrastructure Research University of Nebraska Lincoln, NE			8. PERFORMING ORGANIZATION REPORT NUMBER HR 12-41
9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES) American Association of State Highway and Transportation Officials 444 North Capitol Street, N.W. Suite 249 Washington, D.C. 20001			10. SPONSORING/MONITORING AGENCY REPORT NUMBER
11. SUPPLEMENTARY NOTES Sponsored in cooperation with the Federal Highway Administration			
12a. DISTRIBUTION/AVAILABILITY STATEMENT Available from: Transportation Research Board 2101 Constitution Avenue, N.W., Washington, D.C. 20418			12b. DISTRIBUTION CODE: unlimited
13. ABSTRACT (Maximum 200 words) This report contains the findings of a study that was performed to evaluate existing rapid bridge-deck replacement methods and to develop better procedures and new superstructure designs for future rapid deck replacement. The report provides a comprehensive description of the research, including the details of a continuous precast prestressed stay-in-place concrete system and girder-to-deck connections that would substantially reduce bridge-deck construction and replacement time. The contents of this report will be of immediate interest to bridge and construction engineers and others involved in the design, construction, and rehabilitation of bridge structures.			
14. SUBJECT TERMS Bridges, Other Structures, and Hydraulics and Hydrology; Materials and Construction			15. NUMBER OF PAGES
			16. PRICE CODE
17. SECURITY CLASSIFICATION Unclassified	18. SECURITY CLASSIFICATION OF THIS PAGE Unclassified	19. SECURITY CLASSIFICATION OF ABSTRACT Unclassified	20. LIMITATION OF ABSTRACT

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Report 407

Rapid Replacement of Bridge Decks

MAHER K. TADROS
MANTU C. BAISHYA
Center for Infrastructure Research
University of Nebraska
Lincoln, NE

Subject Areas

Bridges, Other Structures, and Hydraulics and Hydrology
Materials and Construction

Research Sponsored by the American Association of State Highway and Transportation Officials in Cooperation with the Federal Highway Administration

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY PRESS
Washington, D.C. 1998

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state and local governmental agencies, universities, and industry; its relationship to the National Research Council is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the National Research Council and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are the responsibilities of the National Research Council and the Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

NCHRP REPORT 407

Project C12-41 FY'94

ISSN 0077-5614

ISBN 0-309-06260-8

L. C. Catalog Card No. 98-60089

NOTICE

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The members of the technical committee selected to monitor this project and to review this report were chosen for recognized scholarly competence and with due consideration for the balance of disciplines appropriate to the project. The opinions and conclusions expressed or implied are those of the research agency that performed the research, and, while they have been accepted as appropriate by the technical committee, they are not necessarily those of the Transportation Research Board, the National Research Council, the American Association of State Highway and Transportation Officials, or the Federal Highway Administration, U.S. Department of Transportation.

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Published reports of the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Transportation Research Board
National Research Council
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

and can be ordered through the Internet at:

http://www.nas.edu/trb/index.html

Printed in the United States of America

FOREWORD

By Staff
Transportation Research Board

This report contains the findings of a study that was performed to evaluate existing rapid bridge-deck replacement methods and to develop better procedures and new superstructure designs for future rapid deck replacement. The report provides a comprehensive description of the research, including the details of a continuous precast prestressed stay-in-place concrete system and girder-to-deck connections that would substantially reduce bridge-deck construction and replacement time. The contents of this report will be of immediate interest to bridge and construction engineers and others involved in the design, construction, and rehabilitation of bridge structures.

With increasing traffic on roadways, motorists are becoming less tolerant of delays during rehabilitation of bridge decks. In addition to the travel delays and increased risk of accidents, both users and owners incur financial losses because of the interruption of service during deck replacement or rehabilitation.

Under NCHRP Project 12-41, "Rapid Replacement of Bridge Decks," the University of Nebraska-Lincoln was assigned the task of developing optimized systems for rapid deck replacement and recommending details for new superstructures that can facilitate future rapid replacement. To accomplish this objective, the researchers reviewed relevant domestic and foreign literature, surveyed U.S. departments of transportation, bridge designers, and contractors; performed analytical studies and laboratory tests; and recommended bridge-deck systems and girder-to-deck details that would reduce bridge-deck construction and replacement time. The report documents the work performed under Project 12-41 and discusses several means for expediting the removal of existing bridge decks.

The recommended continuous stay-in-place and full-depth precast prestressed concrete deck systems and girder-to-deck connections, described in this report, will facilitate deck reconstruction and replacement, result in reduced total reconstruction time, and lead to improved public acceptance. This information will be particularly useful to highway agencies and is recommended for consideration and adoption by AASHTO as recommended practices.

CONTENTS

AUTHOR ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 12-41 by the Center for Infrastructure Research of the University of Nebraska, the principal contractor for this study, jointly with HDR Engineering, Inc. and Kiewit Construction Company. Maher K. Tadros, Cheryl Prewett Professor of Civil Engineering, is the principal investigator, and Mantu C. Baishya, Research Assistant Professor, was a prime participant in the research.

Other participants in the project are Sameh Badie and Hussam Kakish of the University of Nebraska; William Dowd, Philip E. Rossbach, Hussein Khalil, and Darin Splittgerber of HDR Engineering, Inc.; Sharad Mote, Frank Watt, Jerry Thoendal, and Gary A. Pietrok of Kiewit Construction Company; and Takashi Yamane of Kyokuto Corporation, Japan. Also, Mounir Kamel, Sherif Yehia, and Zhongguo Ma of the University of Nebraska provided assistance during the experimental phases of the project. Deborah Derrick, Communications Specialist for the Center of Infrastructure Research, contributed to the project administration and report editing.

RAPID REPLACEMENT OF BRIDGE DECKS

SUMMARY

The ability to rapidly replace deteriorated bridge decks minimizes public inconvenience, travel delays, and financial losses. Methods of bridge-deck replacement that allow repair work to be completed at night or during other periods of low traffic and methods that reduce the total time of reconstruction help to improve public acceptance, reduce accident risk, and yield economic and environmental benefits. This study was conducted to evaluate existing rapid bridge-deck replacement methods and develop better procedures and new superstructure designs for future rapid deck replacement.

The three main areas where modifications could be made to make deck systems more suitable for rapid replacement are the demolition process and equipment, the bridge deck system itself, and the bridge girder-to-deck connection. Removing concrete from around steel connectors, without damage to the connectors or the girders, is particularly expensive. The cost of removal has a major influence on the selection of the method of rehabilitation. Also, each method and the equipment used for deck removal have their advantages and disadvantages. Selection of the method and equipment used for deck removal should be made on a job-by-job basis. A set of special provisions is provided to aid owners in the task of deck removal. Because of rapid advancement in equipment technology and the unique abilities of contractors, owners are advised to adopt performance-based specifications that allow for maximum freedom without compromising structural and environmental concerns.

The second area of study to improve the speed of deck replacement deals with the deck system itself. Seventy percent of U.S. bridges have cast-in-place (CIP) reinforced concrete decks over steel or concrete girders. A reduction in the amount of reinforcement in CIP systems can enhance deck replacement. Because the most recent *AASHTO LRFD Bridge Design Specifications* yield a significantly reduced amount of reinforcement in the deck, it should be used wherever applicable. Also, the use of welded wire fabric (WWF) as a replacement for conventional reinforcing bars can considerably reduce the duration of construction. The laboratory time log shows that it took 30 percent less time to place the reinforcement on a deck with WWF than a conventional system.

An innovative continuous precast prestressed stay-in-place (SIP) system was developed. The system, shown in Figure 1, extends over the full width of a bridge deck and is continuous both in the transverse and the longitudinal directions to eliminate reflective cracks. The portion over the girder line is kept open to accommodate shear studs, and the overhang form is a part of the SIP system. The construction time for this system was 20 percent less than that for a conventional SIP system and 60

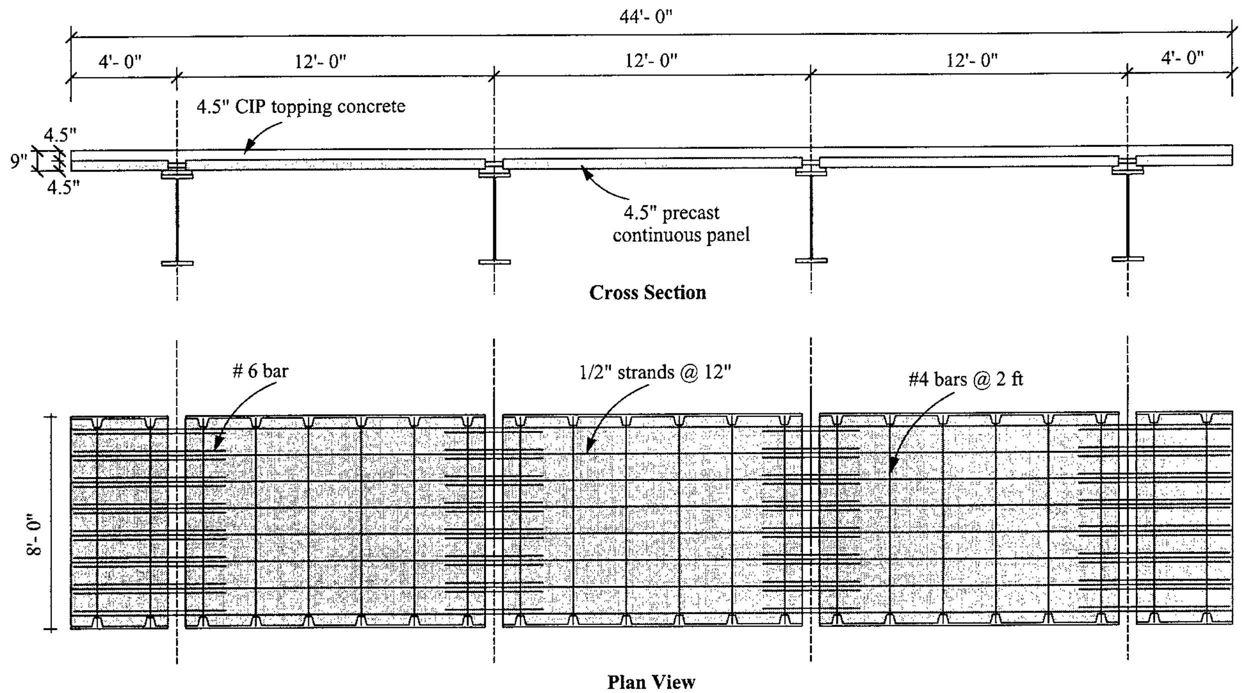


Figure 1. Continuous SIP subpanel bridge deck system.

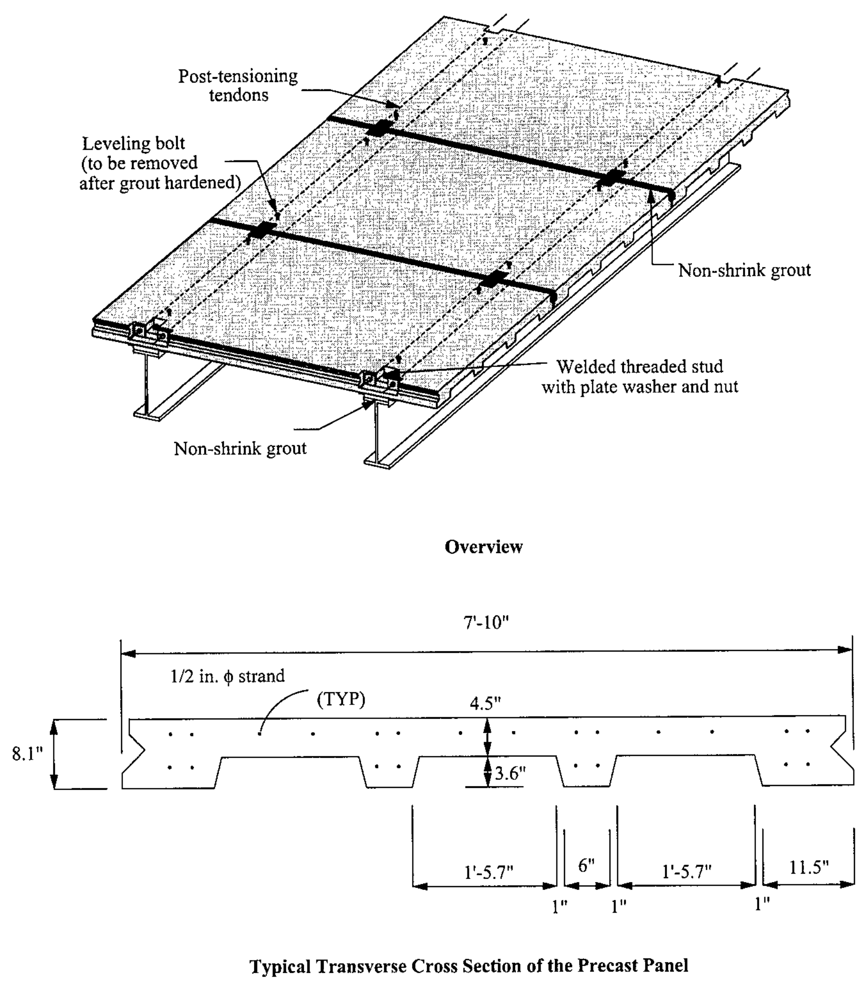


Figure 2. Full-depth precast prestressed concrete bridge deck system.

percent less than for a conventional CIP system. Cost estimates show that this system is comparable to the CIP deck system. Also, a full-depth precast system was developed. The system, shown in Figure 2, is transversely pretensioned and longitudinally post-tensioned. This system is about 10 percent thinner and 20 percent lighter than conventional CIP or precast reinforced concrete sections. The full-depth precast panel system was found to require the least construction time of all systems studied. Further, two-way prestressing controlled the occurrence of transverse and longitudinal cracking. Cost estimates show that this system compares very favorably with other full-depth systems and is ideally suited for new construction as well as renovation.

The third area of study to improve the speed of deck replacement examined the connection system of concrete decks to concrete or steel girders. Demolition of bridge decks that are compositely connected with either structural steel I-girders or precast concrete I-girders is one of the major time-consuming tasks in deck replacement. The time required for deck demolition can be reduced by constructing bridges with connections that provide composite action and allow for easier deck removal. Two new connection systems were developed, one for concrete girder-to-concrete deck connections and the other for steel girder-to-concrete deck connections. For concrete girders, a debonded shear key system, shown in Figures 3 and 4, was developed. This system provided excellent results for composite action as well as deck removal. For steel girder-to-concrete deck connections, a 1¼-in. (32-mm) diameter shear stud system, shown in Figure 5, was developed to replace the commonly used ¾-in. (19-mm) and ⅞-in. (22-mm) shear studs. The 1¼-in. stud provides approximately twice the capacity

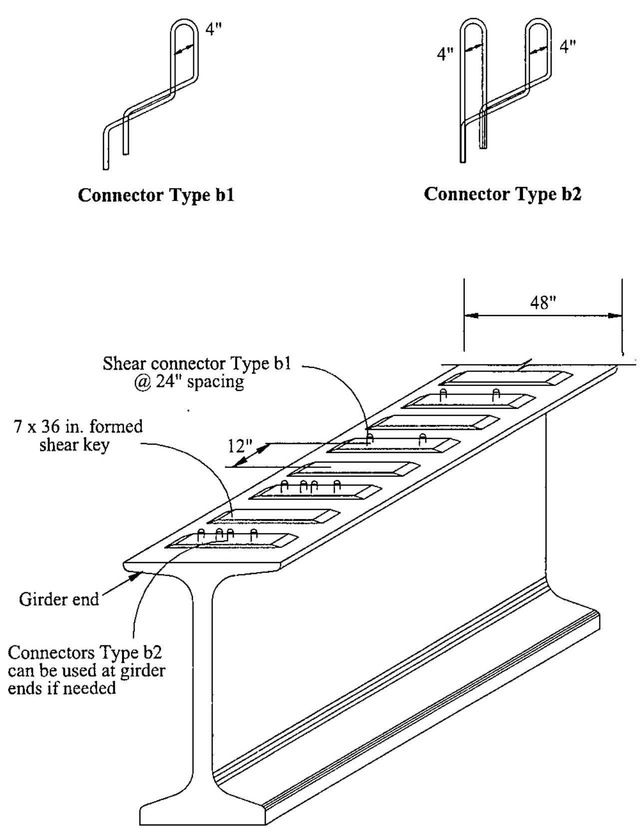


Figure 3. Proposed connectors for wide flanged concrete girders.

Figure 4. Shear keys on concrete I-girder.

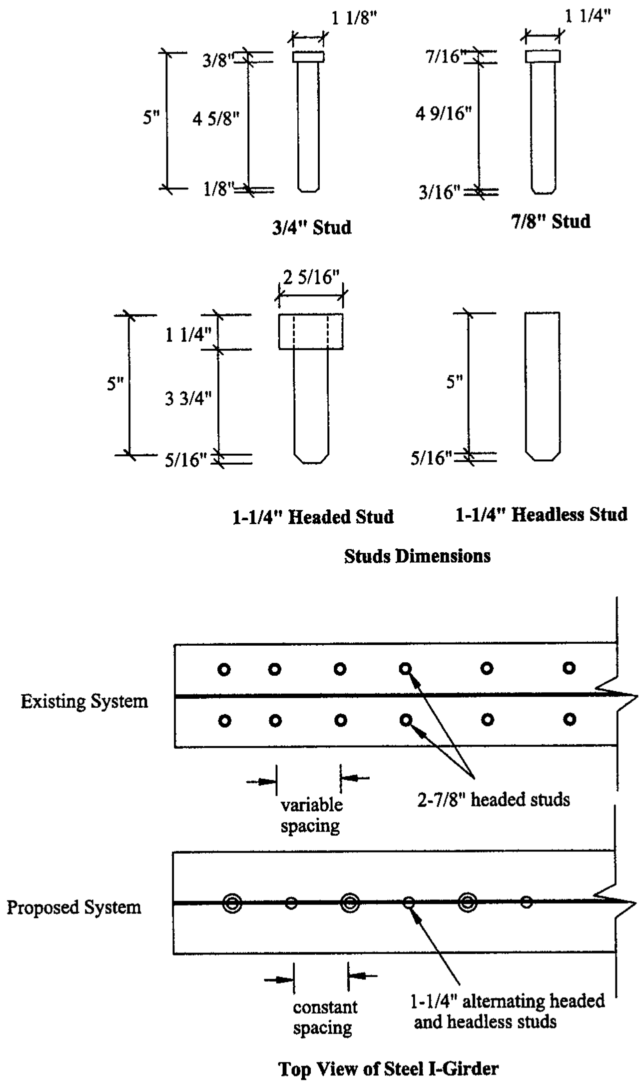


Figure 5. Proposed 1¼-in. shear stud.

of a ⅞-in. stud and would allow positioning in a single row over the girder web. Also, the research team found that alternating headed and headless studs was adequate for anchorage to the concrete deck. This further facilitates deck removal.

Timelines were produced for different bridge deck systems to compare the speed of construction. Finally, a cost analysis was done to compare the cost-effectiveness of the systems investigated in this project.

CHAPTER 1

INTRODUCTION

PROBLEM STATEMENT

The number of bridges owned and operated on public roads in the United States exceeds 600,750, of which 206,904 are over 40 years old and have become substandard. One of the most critical substandard aspects of these bridges is the deteriorated decks. Rehabilitation of the deteriorated bridge decks causes public inconvenience, travel delay, and economic impact. Maintaining minimum interference with traffic flow during bridge repair is often a difficult task for highway agencies and requires extensive planning and coordination. Therefore, it is desirable to adopt deck replacement techniques that allow repair work to be completed rapidly during periods of low traffic volume, such as nights and weekends, thereby reducing the risk of accidents and minimizing interference with traffic, financial losses, and environmental impact.

Over the past years, many strategies have been developed to enhance the efficiency of bridge deck replacement. For example, incorporating prefabricated elements would provide cost and time savings by reducing on-site construction time and staff. Because each of these methods has its own advantages and disadvantages, a need was identified to conduct a comprehensive study to synthesize and evaluate existing rapid deck replacement methods. Efforts should also be made to combine the benefits of previous experience into refined procedures that account for new developments in materials, design, and construction technologies.

In new bridge designs, most bridge designers do not consider the fact that decks would need to be replaced at intermediate points in the life of a bridge. The public deserves a product that provides the lowest possible life cycle cost, where the initial cost and costs of maintenance and rehabilitation are considered. In that sense, it is important to adopt new bridge designs that would allow for future rapid deck replacement.

To enhance the efficiency of rapid deck replacement, a comprehensive study was conducted to develop optimum systems for rapid deck replacement of existing CIP reinforced concrete bridge decks on steel or prestressed girders. The major aspects of this study were to examine the practices currently used for concrete deck replacement, develop approaches for improved procedures, and make recommendations for new bridge superstructure designs that will enhance future rapid deck replacement.

OBJECTIVES AND SCOPE OF THE RESEARCH

The objectives of this research were to develop optimum systems for rapid replacement of concrete bridge decks and to recommend design guidelines for new bridge superstructures that will facilitate future rapid deck replacement. The following specific tasks were pursued:

- Review recent domestic and foreign literature that deals with field procedures and performance of modern replacement products.
- Review and evaluate the bridge deck removal and replacement techniques currently used by federal, state, county, and municipal agencies. The evaluation considered the speed of deck replacement, safety, economic considerations, and environmental impact. Consideration was also given to the evaluation of the structural integrity and capacity of bridge superstructures when the deck is partially removed for reconstruction.
- Develop and evaluate conceptualized optimum systems for rapid deck replacement, considering both composite and noncomposite connection systems. These systems should be cost-effective and provide the required structural integrity, stability, and long-term performance for existing and new bridges.
- Develop and evaluate, in a laboratory test program, shear connectors that facilitate rapid deck replacement and provide composite action of bridge deck slabs with concrete or steel girders.
- Develop design details for the optimized systems, and construct and evaluate prototype specimens as well as full-scale concrete and steel girders in a laboratory testing program to evaluate structural integrity, stability, and fatigue.
- Develop design guidelines and details for systems, to be used for both existing and future superstructures, that facilitate future rapid deck replacement.
- Prepare a user's manual for recommended improvements and concepts, designs, detailed drawings, construction procedures, and specifications, and develop a plan to facilitate implementation of the systems recommended for rapid bridge deck replacement.

RESEARCH APPROACH

This study included a literature review of domestic and foreign literature, a questionnaire survey of transportation agencies, personal interviews with staff from 10 selected state departments of transportation (DOTs) and with Japanese engineers, and analytical studies.

The research team performed a detailed literature review to determine the state of the art of the rapid replacement of bridge decks. Abstracts of pertinent domestic and foreign findings have been reviewed to identify the published and unpublished data most relevant to the project objectives. Article searches have also been made utilizing key word searches of directories for technical publications.

A nationwide survey of bridge owners, designers, and contractors was performed to identify the current practices for existing concrete deck replacement, to obtain ideas for improving deck replacement procedures, and to seek recommendations for new bridge superstructure designs to enhance future rapid deck replacement. Questionnaire surveys were distributed: 250 to owners, 100 to designers, and 60 to contractors. The research team received 93 completed responses, i.e., a 22.7 percent response rate. In addition, the research team interviewed bridge engineers who were involved in bridge deck replacement projects in Japan.

The research team visited 10 state DOTs to review current practices for existing concrete deck replacement, to seek out new ideas for improvements to deck replacement procedures, and to solicit recommendations for incorporating rapid deck replacement in the new bridge superstructure designs. These DOTs are California, Georgia, Illinois, Minnesota, Missouri, Nebraska, New York, Texas, Virginia, and Washington.

The research team identified the following items for simplification and/or modification to facilitate rapid replacement of bridge decks:

- New or replacement of prefabricated bridge deck systems;
- New or replacement of CIP concrete bridge decks;
- Modification or improvement of girder-to-deck connections; and
- Provisions for deck demolition.

These topics were investigated through literature review, experience, analytical studies, and laboratory tests.

ORGANIZATION OF THE FINAL REPORT

This report consists of four chapters. This chapter provides the introduction and research approach, describes the problem statement and research objective, and outlines the scope of study. Chapter 2 describes the findings for this research. Based on the findings from the literature review, responses to the national survey, and interviews with DOT officials and Japanese engineers, a work plan was prepared to evaluate potential approaches for rapid deck replacement. Chapter 3 describes the various systems and test results and includes special provisions for the deck demolition process and evaluation. Chapter 4 summarizes the conclusions and recommendations of this research.

APPLICABILITY OF RESULTS TO HIGHWAY PRACTICE

This project was structured to improve bridge design and construction features. The design and construction of each rapid replacement component was analyzed and laboratory tested. The findings provided in Chapter 4 could be directly applied to construction activities and would definitely reduce construction time for bridge deck replacement.

CHAPTER 2

RESEARCH FINDINGS

INTRODUCTION

The literature review determined the state of the art of the rapid replacement of bridge decks. In assembling the review, the research team collected domestic and foreign literature, research findings, and performance data. Most of the literature reviewed was related to different types of decks, girder-to-deck connections, and construction practices. Similar articles were grouped together and reported under appropriate headings for the reader's convenience.

A nationwide survey was also conducted to investigate the current practices for replacing existing concrete bridge decks. Several conceptual details for the rapid replacement of bridge decks were introduced in the survey to stimulate ideas from responding agencies. The surveys were sent to 50 state agencies, 50 port authorities, public works departments for the 50 largest U.S. cities, 150 design consultants, and 110 contractors.

In addition to the survey, personal interviews were conducted with the state DOTs of California, Georgia, Illinois, Minnesota, Missouri, Nebraska, New York, Texas, Virginia, and Washington.

LITERATURE REVIEW

The review of domestic and foreign literature was performed to determine the state of the art of rapid bridge-deck replacement. The review identified modifications that can be made to make systems suitable for rapid replacement of bridge decks. The literature studied pertained to demolition processes and equipment, bridge deck systems, and bridge girder-to-deck connections.

Demolition Processes and Equipment

Contracts for bridge rehabilitation often include the removal of the entire concrete deck. Concrete removal is a tedious and expensive task that often controls the rate of progress and results in most of the cost overruns on a rehabilitation contract. The service life of the rehabilitated structure is strongly influenced by the quantity of concrete removed, the cleanliness of the reinforcing steel, and the quality of the surface that is prepared. Removing concrete from around reinforcing steel, without damage to the rebars or the concrete left in place, is particularly expensive, and the costs of removal have a major influence on the selection of the method of rehabilitation. As more concrete is removed, the costs may increase to the point at which complete removal and replacement of the component becomes more cost-effective than partial removal and rehabilitation. Also, complete removal permits the use of techniques with higher production rates, which results in an overall cost saving and longer service life of the new component.

Among the many factors affecting bridge deck rehabilitation projects are the availability of suitable equipment, traffic flow considerations, and project planning and scheduling.

The time allotted for work completion controls the selection of equipment and traffic control method used during the deck replacement. Project conditions, such as traffic, schedule constraints, and site conditions, control the selection of the technique to be used for deck removal.

Removal refers to the removal of deteriorated concrete that can be either partial or complete. Partial removal involves the removal of deteriorated concrete before preparing the surface and placing concrete (e.g., by patching or overlay). The objectives in removing part of a concrete member are to salvage the sound portions and to remove the concrete quickly and economically without damage to the concrete to be left in place. The studied literature dealt mainly with the complete removal of bridge decks; partial removal was not considered in this research. However, equipment used for partial removal and full removal is discussed in Appendix A.

For reasons of economy and efficiency, there is a natural tendency to use much larger equipment for the complete removal of concrete than for partial removal (*1*). Pneumatic breakers have been used for almost a century, whereas other equipment, such as hydrodemolition equipment, is relatively new. Although the range of available equipment is vast, it is rare that different types of equipment can be used interchangeably. The optimum choice of equipment for a particular application is determined by the (1) quantity and quality of concrete to be removed, (2) time available to complete the work, (3) type of concrete component and its accessibility, (4) cover to the reinforcement, and (5) restrictions with respect to vibration, noise, dust, and containment and disposal of the debris.

Equipment used for the complete removal of a concrete component includes saws, drills, breakers, splitters, crushers, ball and crane, and blasting. Saws are often used to cut the deck into sections, but care must be taken not to cut the top flanges of bridge girders. Breakers are usually machine-mounted and can be used on most components. Restrictions are often placed on the size and manner for the operation of these breakers because of the high risk of damage to the remainder of the structure.

Preparation of the girder's surface is required following concrete deck removal. A point that has recently received attention is that the concrete surface can be damaged by some methods of concrete removal and surface preparation. This is particularly true of percussive tools and scarifiers, which can fracture the girder deck interface without removing particles of aggregate and paste, and thus leave a surface that is extensively microcracked.

The problem of microcracking has been explored in a variety of circumstances. Hindo (*2*) investigated the effects of pneumatic hammers and hydrodemolition in in-place bond tests and found that, in most cases, there was a damaged or "bruised" area with numerous microcracks immediately below the bond line. Tayabji (*3*) reported on tests on a full-bridge deck panel which showed that the shear bond of low-slump and latex-modified concrete overlays applied to a surface prepared by a hydrodemolisher was comparable to the shear strength of the base concrete. Silfwerbrand (*4*) conducted laboratory pull-off tests on concrete overlays placed on surfaces prepared by water jetting and by pneumatic breakers. The results showed a difference between the mean strength of the slabs with the water-jetted surface and the slabs with the chipped surface attributed to microcracks in the chipped surface of the concrete. Concrete typically failed just below the bond interface when bond tests were conducted on latex-modified concrete overlays placed on scarified surface (*5*).

The other most common damage reported has been saw cuts in the top flange of the steel girders when full-depth sawing was used to remove bridge decks. Because it is difficult to ensure that the repair is satisfactory, most agencies have prohibited full-depth saw cuts over structural steelwork in contact with the deck soffit to avoid such damage.

Another common form of damage reported has been to the top flanges of steel and concrete girders by rig-mounted breakers. This concern has led a number of agencies to prohibit or place restrictions on the use of rig-mounted breakers.

The whiphammer has been used by a few agencies, and most have experienced damage to beams and excessive vibration.

Many agencies have reported using a wrecking ball as a standard procedure in deck removal but provided no information with respect to limitations on the size of the ball. Four agencies (Kansas, Texas, Wisconsin, and Wyoming) have reported on instances where damage occurred. Iowa does not permit the use of a drop hammer when beams are to be reused. Wrecking balls and blasting have often been used in the removal of substructure components, and only Arkansas has reported damage from blasting. Newer techniques, such as splitting or the use of hydraulic cutters or nibblers, have been used infrequently—on an experimental basis. Florida has reported that chemical splitting was not successful on a number of occasions because of deficiencies in the materials.

Some of the most detailed requirements have been compiled in special provisions developed by Maryland that limit full-depth sawing to within 2 in. (50 mm) of the steel structure. Alabama has disallowed rig-mounted breakers where reinforcement or girders will be reused. Illinois has limited breakers to those having a rated striking energy of not more than 1,200 ft-lbf (1,600 J). Maryland has limited rig-mounted breakers to within 6 in. (150 mm) on each side of the edges of flanges and requires that hand-held hammers no heavier than 90 lb (40 kg) be used to remove the remaining concrete. Ontario has restricted the size of breakers to those having a striking energy of not more than 440 ft-lbf (600 J) and also prohibits rig-mounted breakers within 12 in. (300 mm) of the concrete that will remain in place and requires removal of the final 12 in. (300 mm) with hand-held hammers no heavier than 30 lb (13 kg).

Bridge Deck Systems

Full-Depth CIP Deck Systems

Slab design provisions in AASHTO's current *Standard Specifications for Highway Bridges* (*6*) and *AASHTO LRFD Bridge Design Specifications* (*7*) are based on considering a transverse strip of the bridge as a continuous beam supported by longitudinal girders. The girders are considered as rigid supports ignoring the settlement due to deflection of the girders in the longitudinal direction of the bridge. Because of the effect of the deicing salts on the deck slab reinforcement, some state agencies, such as Missouri DOT (*8*), specify the use of epoxy-coated bars.

AASHTO LRFD specifications (*7*) give an alternative empirical method for the design of full-depth CIP bridge decks, if the deck satisfies certain conditions. The reinforcement amount provided by the empirical method is less than that provided by the strip method. Recent research (*9*) confirmed the validity of the empirical design method. It also indicated that the tensile bending stresses developed at the top of a bridge deck subjected to traffic loads were relatively low. As a result, the top reinforcement needed for sustaining the negative bending moment induced by traffic loads can be eliminated or reduced, which reduces the chance of bridge deck deterioration because of reinforcement corrosion. Top reinforcement is only needed over the piers in continuous spans.

Researchers have confirmed that a significant arching system develops in reinforced bridge deck slabs when subjected to concentrated loads (*10–13*). The in-plane compressive stresses generated because of this arching action cause slabs to fail in punching shear, rather than in flexure, and at a much higher load. The *Ontario Highway Bridge Design Code* (*14*) took advantage of this behavior by requiring that these components be designed for failure by punching shear instead of flexure. It introduced provisions for empirical design of reinforced concrete bridge decks which lead to a considerable reduction in reinforcing steel.

Researchers at the Technical University of Nova Scotia, Canada, developed a fiber-reinforced concrete deck slab without conventional steel reinforcement (*15*). Corrosion in concrete deck slabs is eliminated entirely by replacing the steel reinforcement with polypropylene fibers. To maintain the arching action inside the deck slab, a lateral restraint is attached to the deck slab by welding a transverse steel strap to the top flanges of the girders. The Nova Scotia Department of Transportation and Communications used this system in the Salmon River Bridges, Kemptown, Nova Scotia, Canada.

A full-depth CIP bridge deck system has the advantage of allowing for field adjustment of the profile of the riding surface. Although modern concrete placement and finishing systems make field casting of concrete cost-effective, there are several drawbacks to this system. These include (1) low construction speed, (2) high cost of field forming and placing of reinforcement, (3) need for field quality control, and (4) likelihood of cracking due to differential shrinkage between the deck and the underlying girder. To reduce the time and cost of placing reinforcement and corrosion potential, some state agencies allow the use of epoxy-coated welded wire fabric which has the advantage of covering large areas in a shorter time than when using reinforcing bars (*16*). However, there are some problems encountered with the use of welded wire fabric reinforcement, such as splicing, especially with the large size of bars, and the effect of epoxy coating on the development and anchorage of the reinforcement. Recent tests conducted on a series of pull-out specimens and one-way slabs showed that epoxy coating has little or no effect on the development and anchorage of welded plain wire fabric and welded deformed wire fabric (*17*). The tests also indicated a similar cracking behavior of slabs reinforced with uncoated and epoxy-coated welded wire fabric. However, there was a significant difference in cracking behavior between slabs reinforced with welded plain wire fabric and welded deformed wire fabric.

Precast Deck Subpanel Systems

SIP precast panels for CIP deck systems can reduce construction time and improve cost-effectiveness. The precast panels are 2.5 to 3.5 in. (62.5 to 87.5 mm) in thickness and prestressed at their centerlines with ⅜-in. (9.5-mm), 7/16-in. (11-mm), or ½-in. (12.5 mm) strands. These panels are produced in a width that ranges from 4 to 8 ft (1,220 to 2,450 mm). The thickness of the CIP topping slab ranges from 3.5 to 6.0 in. (87.5 to 150 mm), depending on the girder spacing. The topping slab is reinforced to resist the negative bending moment over the supporting girders resulting from superimposed dead loads and the live loads. Figure 6 gives complete details of this system.

Some designers believe that the minimum thickness of panels should be 3.5 in. (87.5 mm) to conform with AASHTO requirements, which require 1.5-in. (37.5-mm) clear cover. The 3.5-in. (87.5-mm) thickness would reduce the possibility of panel cracking due to handling or longitudinal splitting at strands near panel edges. However, some state agencies, such as in Missouri (*8*), use 3-in. (75-mm) thick precast panels pretensioned with ⅜-in. (9.5-mm) diameter strands. Occurrence of cracks at edges over the prestressing strands can be minimized by (1) placing additional reinforcing steel perpendicular to the strands near the transverse edges, (2) using gentle release of prestressing strands, (3) spacing the strands uniformly, and (4) accurately locating the strands in the midheight of the panel.

One of the most important factors that needs to considered is the bearing of the precast panels on the supporting girders. In bridges built in the 1960s, it was customary to use a soft material under the portion of the panels bearing on the girders. Research on deck cracking (*18,19*) has demonstrated that the panels must be firmly bedded on grout or concrete on the supporting girders. A number of satisfactory details have been used to provide elevation adjustment and grout space between the top of the supporting girder and the panel. Figures 7 and 8 give details for precast panel bearings over steel and concrete girders, respectively.

In the case of skewed bridges, skewed spans are often cast in place for the full depth of the deck. On lightly skewed spans (15 deg or less), the panels are sawed to match the skew (*20*). A minimum bearing length of 1 ft (300 mm) on

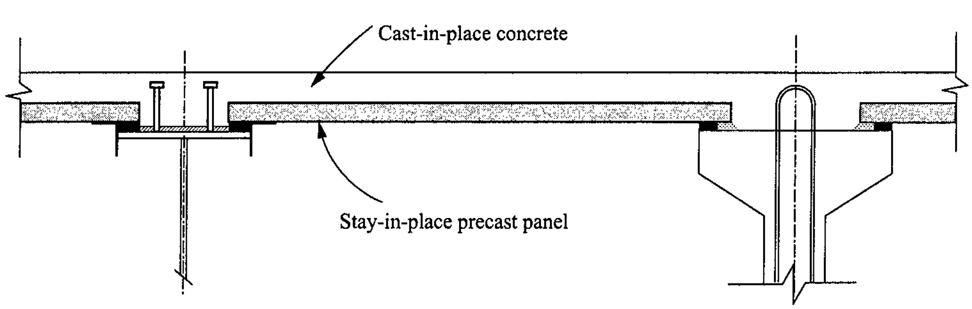


Figure 6. Precast deck subpanel system.

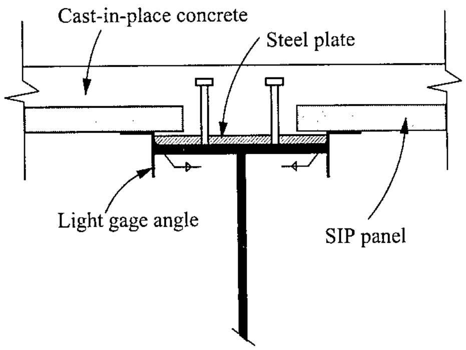


Figure 7. Details of precast panel bearing over steel girders.

the short side of the panel is sometimes required. Normal curvatures can be accommodated by slight offsets in the standard rectangular panels or by occasional trapezoidal-shaped panels in areas with a sharper curvature. Cantilever sections are most often formed with standard formwork and cast full depth in place.

Detailing is a crucial aspect of the performance of the system. It addresses questions concerning the type of joint between the deck precast panels, strand development length, the need for mechanical shear connectors, and transverse panel strand extension.

Research at Pennsylvania State University (*21*) showed that the type of joint used between precast deck panels affects neither deck behavior nor wheel load distribution. It also showed that a 6-in. (150-mm) projection of prestressing strands from the ends of the panel is needed to anchor the panel to the CIP topping. However, research at the University of Texas at Austin (*22*) showed that under cycling load there was no significant difference in cracking pattern or cracking width between bridges built with precast panels, with or without strand projection.

Research at the University of Florida (*18*) showed that in most cases the provided development length is less than the AASHTO requirements. Therefore, the research recommended that reinforcing steel perpendicular to the prestressing strands should not be less than #3 bars at 12 in. (300 mm)

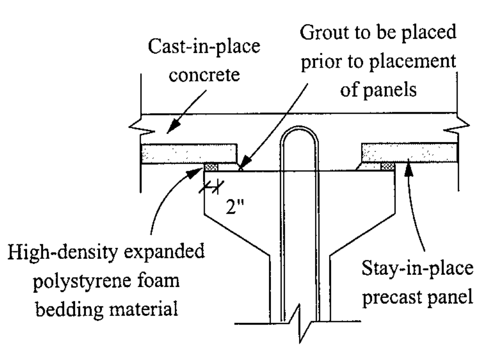


Figure 8. Details of precast panel bearing over concrete girders.

on centerlines, and the strands should be clean of any trace of oil or any other substance that may adversely affect the bond between concrete and the strands.

The Texas DOT examined three bridges built with the SIP precast prestressed panel system after they had been in service for 10 years (*23*). The examination showed that there was no evidence of distress or noncomposite action. Some transverse cracking was found in the CIP topping of two of the more heavily traveled structures. With few exceptions, these cracks coincided with the transverse butt joints between the panels. Results of laboratory tests showed that (1) the bond at the interface between the precast prestressed panels and the CIP topping performed without any distress under cyclic design loads and static failure loads, and it is feasible to design for the composite action; (2) wheel loads were transferred and distributed across the transverse panel joints satisfactorily; (3) average development lengths of 22 in. (550 mm) and 34 in. (850 mm) were required for the ⅜-in. (9.5-mm) and ½-in. (12.5-mm) diameter strands, respectively; (4) the type of concrete (i.e., normal or lightweight) had little effect on the development length; and (5) cyclic loading had a negligible effect on the strand development length.

A recent study at Purdue University (*24*) evaluating interface horizontal shear strength in composite decks indicated that specimens with shear connectors were stiffer near ultimate load than those without shear connectors. The researchers reported that shear connectors were not strained prior to reaching ultimate load. At failure, with load higher than the predicted flexural capacity, interface slip occurred suddenly, and an increase in strain of the shear connectors was observed. The report concluded that for precast panels with a broom-finished surface, horizontal shear connectors are not required if the stress at interface is less than 116 psi (0.8 MPa). Maybe a minimum of four shear connectors can be provided for shipping and handling purposes.

Full-Depth Prefabricated Deck Systems

Many full-depth precast deck systems have been used to replace deteriorated bridge decks in high traffic areas where it is required to minimize the construction time. Because each project has its own constraints (e.g., girder type and spacing, shear connector type, and skew), each bridge has its own customized details. Details of precast deck systems include blockouts for shear connectors, tie-down connections, and transverse connections between the precast panels (shear keys).

The full-depth precast system may be conventionally reinforced, transversely prestressed, or biaxially (transversely and longitudinally) prestressed. Transverse prestressing is used to optimize the cross-section dimensions of the panels and consequently to reduce its own weight. Longitudinal prestressing is usually provided by means of post-tensioning to eliminate transverse cracks between panels. Certain types of the full-depth precast panels, such as the Inverset, are self-prestressed by use of the upside-down casting technique.

Other full-depth precast deck systems include the steel grid system, exodermic bridge deck system, Inverset deck system, IKG Greulich bridge flooring system, and Effi-Deck system. All of these systems have a combination of concrete and structural steel members, such as tubes or special bars.

Bridge Girder-to-Deck Connections

Concrete Girder-to-Deck Connections

The American Concrete Institute (ACI) Building Code (*25*) introduced the first design provisions for steel crossing the interface between CIP slabs and precast beams. These provisions were based on research by Hanson (*26*) and Kaar et al. (*27*) in which they conducted horizontal shear tests using push-off specimens and composite flexural beams. The push-off tests were found to be a relatively simple and inexpensive way of determining the horizontal shear strength when compared to constructing composite beams. The shear strength at the interface was assumed to vary directly with the amount of crossing reinforcement. The first parabolic equation for shear transfer strength was introduced by Birkeland and Birkeland in 1966 (*28*); other parabolic equations were introduced by Raths (*29*) and Loov (*30*). In 1970, the concept of "shear-friction" was modified in the ACI Building Code on the basis of push-off tests by Birkeland and Birkeland (*28*), Mast (*31*), Kriz and Raths (*32*), and Hofbeck et al. (*33*). The effects of concrete strength and clamping stress on the strength of the composite connection were correlated by Loov and Patnaik (*34*).

Horizontal Shear Failure and Critical Slip Limit. A large slip or separation of the slab relative to the precast beam before the beam fails in flexure or in vertical shear can cause a horizontal shear failure to the composite beam, resulting in loss of composite action and a significant reduction of stiffness and flexural capacity. To develop clamping stresses across the shear interface, which is essential in developing horizontal shear capacity, a certain amount of relative horizontal slip has to occur. A slip of 0.005 in. (0.127 mm) was found by Hanson (*26*) to be the critical value beyond which the composite action will be lost. Patnaik (*35*) found that if a slip of 0.02 in. (0.51 mm) was permitted, shear strength in beam tests, reported by Hanson (*26*) and Mattock and Karr (*36*), would have been increased by 50 percent. Loov and Patnaik (*34*) recommended that a relative slip value of 0.02 in. (0.51 mm) be permitted in order for the steel reinforcement to develop the clamping stress.

Code Equations for Horizontal Shear Design. The specifications used in the United States for horizontal shear design are compared in this section. It also includes the Canadian Code (*37*), which is based on the Loov and Patnaik equation but without the cohesion term. All design equations are for nominal shear strength without load factors and for concrete placed against bonded, roughened concrete surfaces with an amplitude of 0.25 in. (6.25 mm). In this comparison, the equations have been manipulated to give the total horizontal shear stress, v_{nh}, directly.

- According to AASHTO Standard Specifications (*6*):

  No ties: $v_{nh} = 80$ psi

  Minimum ties: $v_{nh} = 350$ psi, $A_{vh} = \dfrac{f_{cl} b_v s}{f_y}$

  Ties > minimum: $v_{nh} = 350 + 0.4(f_{cl} - 50)$ psi.

- According to AASHTO LRFD Specifications (*7*):

  $$v_{nh} = c + \mu f_{cl} + \frac{\mu P_c}{b_v s} \le 0.2 f'_c, \le 800\text{psi}$$

  For the indicated roughened surface condition: $c = 100$ psi and $\mu = 1.0$.

- According to ACI 318-95 (*38*):

  No ties: $v_{nh} = 80$ psi

  Minimum ties: $80 < v_{nh} = \lambda(260 + 0.6 f_{cl}) \le 500$ psi

  Ties > minimum: $v_{nh} = \mu f_{cl}, \le 0.2 f_c', \le 800$ psi, >500 psi

  For the indicated roughened surface conditon: $\lambda = 1.0$ and $\mu = 1.0$.

- According to Canadian Code CSA-A23.3-94 (*37*):

  $$v_{nh} = 0.5\sqrt{f_{cl} f'_c} \le 0.25 f'_c \text{ psi}$$

  where:

  v_{nh} = total horizontal shear stress;
  A_{vh} = area of horizontal shear reinforcement;
  b_v = girder top flange width;
  s = spacing of horizontal shear reinforcement;
  f_{cl} = clamping stress;
  f_y = yield strength of shear reinforcement;
  f_c' = compressive strength of concrete;
  c = cohesion between girder top and deck;
  μ = coefficient of friction;
  P_c = permanent net compressive force on the interface; and
  λ = lightweight concrete reduction factor (1.0 for normal weight concrete).

Figure 9 compares the horizontal shear strength–clamping stress relationship according to the various code equations for a 4,000 psi compressive strength and coefficient of friction of 1.0. The AASHTO Standard and ACI Code equations provide similar relationships and both yield more conserva-

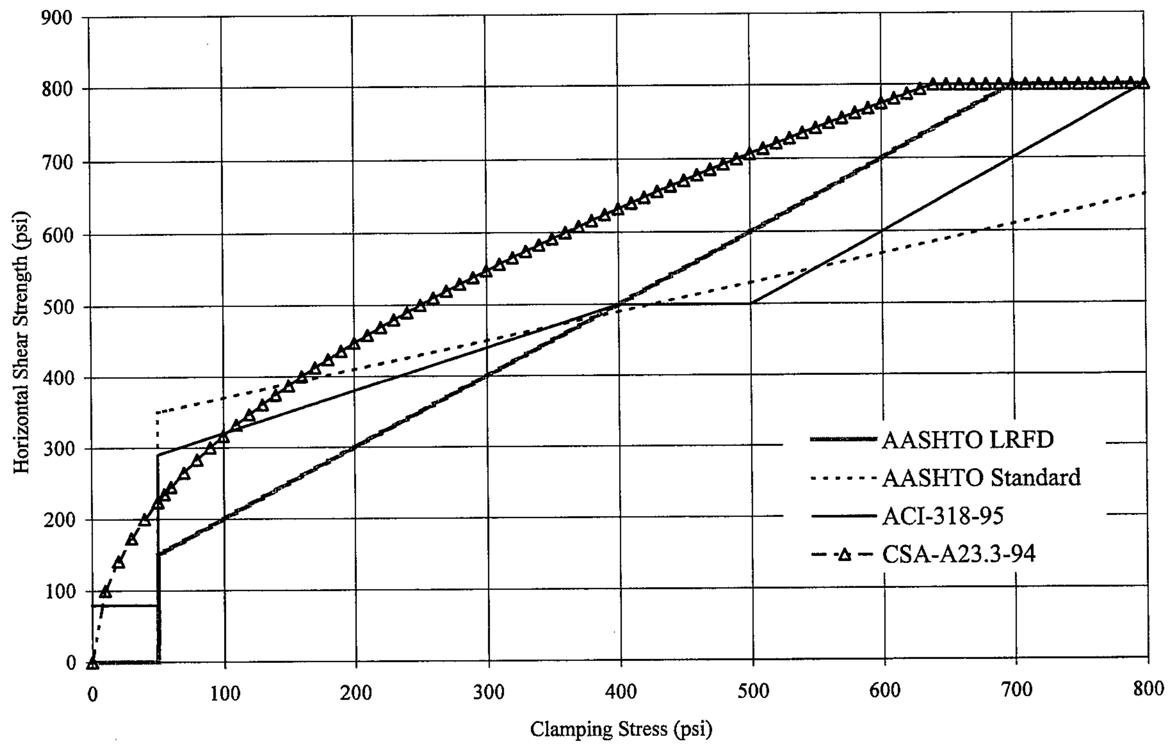


Figure 9. Horizontal shear strength versus clamping stresses.

tive values when compared to the Canadian Code except in areas where the clamping stresses are between 80 and 100 psi (0.55 and 0.69 MPa). Of all the other equations, the LRFD equation yields the most conservative values for a clamping stress up to 400 psi (2.76 MPa).

Steel Girder-to-Deck Connections

The most common horizontal shear connector used today is the shear stud, which consists of a smooth shank with a flat round head. The stud is welded or "shot" to the top flange of the steel girder using an efficient arc welding process.

The use of welded shear studs did not begin until the early 1950s when research by Viest (*39*) began paving the future for this new connector. The first bridge to use shear studs was the Bad River Bridge in Pierre, South Dakota, completed in 1956 (*40*). The initial design equations for welded studs were based only on strength. It was not until the 1960s that stud fatigue strength was considered in the design. There has been extensive research conducted on the ultimate and fatigue strength of welded shear studs, using both composite beams and push-off tests.

In most cases, the diameter of the studs tested were: ½ in. (12.7 mm), ⅝ in. (15.9 mm), ¾ in. (19.1 mm), 1 in. (25.4 mm), and a few 1¼ in. (31.8 mm).

Critical Slip. Viest (*39*) recommended a critical slip of 0.02 in. (0.51 mm) to be considered in determining the ultimate strength of the 1¼-in. (31.8-mm) shear studs. He indicated that at higher loads residual slip increased greatly with small increments of load, suggesting that large inelastic deformation, caused by plastic deformation of concrete and yielding of the stud steel, probably occurred at these loads. However, the capacity of the stud did increase to about 50 kips (222.4 kN) with 0.10-in. (2.54-mm) slip.

Slutter and Driscoll (*41*) indicated that the magnitude of slip will not reduce the ultimate moment in beam tests if both the equilibrium condition is satisfied and the magnitude of slip is not greater than the lowest value of slip at which an individual connector might fail.

Shear Strength of the Welded Shear Stud. Different equations were presented to evaluate the shear strength of the different diameters of the welded shear stud.

Viest (*39*) presented the following equations for determining the horizontal shear strength or "critical load" for different diameters of the welded shear stud:

For $d_s < 1$ in.:

$$Q_{cr} = 5.25d^2f'_c\sqrt{\frac{4{,}000}{f'_c}} \tag{1}$$

For $d_s \geq 1$ in.:

$$Q_{cr} = 5d^2f'_c\sqrt{\frac{4{,}000}{f'_c}} \tag{2}$$

where:

Q_c = critical load, lb;
d_s = stud diameter, in.; and
f'_c = concrete strength, psi.

Slutter and Driscoll (*41*) recommended the following equation to determine the ultimate strength of the shear stud, for concrete strengths not higher than 4,000 psi or stud diameters not larger than ⅞ in.:

$$q_u = 930 d_s^2 \sqrt{f'_c} \quad (3)$$

where:

q_u = ultimate strength, lb;
d_s = stud diameter, in.; and
f'_c = concrete strength, psi.

The AASHTO Standard Specifications (*6*) give the following equation for the nominal strength of welded shear studs:

$$S_u = 0.4 d_s^2 \sqrt{f'_c E_c} \quad (4)$$

where:

S_u = ultimate strength of individual shear connector, lb;
d_s = stud diameter, in.;
f'_c = concrete strength, psi; and
E_c = modulus of elasticity of the concrete, psi.

The AASHTO LRFD Specifications (*7*) give the strength equation for the nominal strength of welded shear studs as

$$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} \leq A_{sc} f_u \quad (5)$$

where:

Q_n = nominal shear resistance of one connector;
A_{sc} = cross-sectional area of a stud shear connector, in.^{2};
f'_c = specified 28-day compressive strength of concrete, ksi;
E_c = modulus of elasticity of concrete, ksi; and
f_u = specified minimum tensile strength of stud shear connector, ksi.

Fatigue Strength of Shear Connectors. Fatigue tests can be conducted on push-out specimens similar to those used for ultimate tests. The applied stress ranges, the minimum level of stress, and stress reversal are the main variables for such fatigue tests. The range of stress, rather than minimum stress, is the more important variable, if the stress reversal is ignored.

Slutter and Fisher (*42*) and Naithani et al. (*43*) conducted fatigue tests on push-out specimens containing ¾-in. (19-mm) and ⅞-in. (22-mm) diameter welded shear studs. Test results showed that ¾-in. (19-mm) stud connectors failed in fatigue at a lower stress than ⅞-in. (12.7-mm) stud connectors, and the strength of concrete had only a minor effect on cycle life.

Based on test data, Slutter and Fisher (*42*) proposed the following design formula for the allowable range of load:

$$Z_r = \alpha d_s^2 \quad (6)$$

where:

Z_r = allowable range of shear force per stud, lb;
d_s = diameter of the stud, in.; and
α = 13,800 for 100,000 cycles; 10,600 for 500,000 cycles; or 7,850 for 2,000,000 cycles.

Equation 6 could also be applied to smaller diameter shear studs; however, the authors made no recommendations for studs larger than ⅞ in. (22 mm) in diameter.

The allowable range of horizontal shear force, according to the AASHTO Standard Specifications (*6*), is the same as Equation 6 except that the value of α for 100,000 cycles is 13,000; α values for the other cycles are the same.

The AASHTO LRFD (*7*) design equation for the allowable range of horizontal shear force is

$$Z_r = \alpha d_s^2 \geq 5.5 d_s^2 \text{ (kips)} \quad (7)$$

where:

d_s = stud diameter (in.) and
α = 34.5 – 4.28 Log N

where N is the number of load cycles.

Thus, α = 13.1 for 100,000 cycles; 10.1 for 500,000 cycles; and 7.5 for 2,000,000 cycles.

QUESTIONNAIRE SURVEYS AND INTERVIEWS

A nationwide survey of bridge owners, designers, and contractors was (1) to investigate the current practices for replacement of existing concrete decks and the possible improvements of the deck replacement procedures and (2) to solicit recommendations for new bridge superstructure designs to enhance future rapid deck replacement.

A total of 410 questionnaire surveys were sent to bridge owners, designers, and contractors. The survey included three different sets of questions targeted to the owners, designers, and contractors. The research team received 93 completed surveys: 49 from owners; 26 from designers; and 18 from contractors. Results of the surveys, provided in Appendix B, are summarized as follows:

- Visual inspection is the main determining factor for replacement of a bridge deck. Other methods used are chloride content analysis, chain dragging, sounding by hammer, deck coring, and half-cell potential testing.

- The average life of a bridge deck is approximately 25 years. The general trends indicate that full removal of the deck is required after 25 years, and partial removal or repair of the deck may also be required as early as 10 years after placement in service.
- The most common tools used in the removal of existing bridge decks include boom-mounted breakers, saws, and hand-held hammers. Hydraulic jaws are used mainly for steel girder bridges. Other common bridge deck removal procedures include water jets, pressure bursting, crane and ball, and blasting. Less popular procedures include mechanical hammers, planing machines (grinders), shot blasting, roto-mills, and whiphammers.
- There are certain problems associated with each deck removal method. These include the following:
  - –The boom-mounted breaker is easy to use but produces extensive vibration and excessive noise, can damage the top flange of the steel girder, presents safety concerns to the vehicle and worker, and may necessitate an expensive cleanup of debris.
  - –Hydraulic jaws can damage the top flange of the girders and, therefore, require skilled equipment operators to minimize such damage.
  - –Sawing has the potential of gouging the top flange of a steel girder and requires a controlled supply of coolant fluid for best performance.
  - –Hand-held hammering is a very expensive and labor-intensive method.
  - –Deck removal by water jet is expensive. This method can be used effectively for partial removal of the deck, but the environmental concerns associated with the run-off water must be addressed. Also in the case of partial removal, control of the water jet is critical to avoid blow-out through the full depth of the deck.

The factors that must be considered when evaluating the efficiency of a bridge-deck replacement system were identified and rated according to importance, based on the survey results, on a scale of 1 to 10, with 10 being the most important and 1 being the least important. These factors and their importance indicators (in parentheses) are structural performance (10), safety requirements (7.9), traffic control during construction (7.9), measures to protect new deck (7.0), deck material (6.9), volume of traffic and importance of crossing (6.9), life cycle cost (6.9), method of removal and installation (6.8), equipment and level of skill required (6.0), relative initial cost (6.0), cost of bridge partial/full closure (5.9), contractor's availability and experience (5.4), composite and noncomposite design (4.9), possible future replacement (4.6), girder material (4.3), sources of deterioration (4.1), contractor's incentives to accelerate work (3.5), environmental restrictions (3.4), night construction (1.9), and innovative features (1.0).

To develop an optimized system for rapid deck replacement, further research is required in the areas of modular deck systems, deck surface protection methods, shear connectors, and pretensioning or post-tensioning in decks.

The research team also visited the DOTs of California, Georgia, Illinois, Minnesota, Missouri, Nebraska, New York, Texas, Virginia, and Washington. The purpose of the visits was to study current practices for existing concrete deck replacement, to seek out new ideas for improving deck replacement procedures, and to solicit recommendations for new bridge superstructure designs to enhance future rapid deck replacement. Because of climatic conditions and other variables, there are variations in the practices of rapid deck replacement in different states. States with higher population and large cities are very much concerned with rapid replacement of bridge decks. A detailed analysis of the DOT interviews is given in Appendix C.

In addition, to investigate the practices and trends of using rapid replacement of bridge decks in foreign countries, the research team participated in the 12th International Federation of Prestressing (FIP) Congress held in Washington, D.C., from May 29 to June 2, 1994, and interviewed bridge engineers currently involved in bridge-deck replacement projects in Japan. A summary of these interviews is provided in Appendix D.

The research team concluded from the questionnaire surveys that the quality of the deck construction is the top priority, and DOTs were not willing to sacrifice the quality of a bridge deck for future rapid replacement considerations. Another conclusion of the survey is that a higher initial investment may result in a longer life of the bridge deck. It was evident from the DOT interviews that rapid deck replacement is highly important to almost all states because of the need to minimize traffic interruptions.

SUGGESTED AREAS OF RESEARCH

On the basis of the literature review, interviews, and survey results, it was concluded that to enhance bridge-deck replacement, research is required to develop (1) a precast prefabricated deck system for both new construction and replacement; (2) a simplified CIP system because this system is very popular in the United States; (3) a connection system for concrete girders and concrete decks that can ease the deck removal; (4) a connection system for steel girders and concrete decks that can ease the deck removal; (5) timelines for various decking systems that can adequately inform owner, designer, and contractor of the length of time required for a particular system and an evaluation method to aid in selecting the proper system; and (6) special provisions for deck demolition.

CHAPTER 3

RESEARCH RESULTS

INTRODUCTION

A review of the literature and the questionnaire results of bridge owners, designers, and contractors has revealed that prefabricated full-depth panel systems would improve the speed of deck replacement. However, the majority of bridges built in the United States have a CIP concrete slab or incorporate a CIP topping over SIP concrete or steel forms. CIP concrete allows the adjustment of the bridge riding surface during construction. Also, modern concrete placement and finishing methods make field placement of concrete cost-effective. In Phase I of the research, several new ideas for CIP concrete deck replacement evolved that focused on the speed of construction. These ideas were incorporated in three CIP systems that were laboratory tested. These are a 9-in. (225-mm) thick CIP deck reinforced with conventional reinforcement, a 9-in. (225-mm) thick CIP system reinforced with epoxy-coated WWF, and a 3-in. (75-mm) thick precast prestressed SIP panel with 6-in. (150-mm) thick CIP deck reinforced with WWF.

The specimens tested for these three systems were 20 × 8 ft (6,000 × 2,400 mm) in size supported on two girders placed 12 ft (3,600 mm) apart. There were 4-ft (1,200-mm) cantilevers on either side. In this experimental evaluation, the conventional CIP system was used as a benchmark for comparison with the other systems.

After careful study of the test results of the SIP system, the research team designed a new continuous SIP panel system. This system has a 4.5-in. (137-mm) thick continuous sub-panel incorporating integral cantilever sections and a 4.5-in. (137-mm) reinforced CIP topping. In addition, a full-depth precast system was developed, designed, and tested. This system, transversely pretensioned and longitudinally post-tensioned, is 10 percent thinner and 20 percent lighter than a conventional full-depth CIP deck. Three 20- × 8-ft (6,000- × 2,400-mm) panels were designed and fabricated in a local prestressing company and load tested by constructing a 20- × 24-ft (6,000- × 8,000-mm) bridge.

The purpose of testing the various systems was to determine the speed of construction, the structural behavior under cyclic load, and the ultimate capacity of each system in comparison with design values.

Another purpose of the test program was to evaluate new connection systems for composite action between steel or concrete girders and concrete decks that would allow for rapid replacement of deteriorated bridge decks. For steel girder-to-concrete deck connections, the main emphasis was placed on the development of 1¼-in. (32-mm) shear studs that would replace the popular ⅞-in. (22-mm) shear studs. Because the strength of the proposed 1¼-in. stud is about twice that of the ⅞-in. stud, fewer studs will be required along the length of the steel girder. This reduction in the number of studs would reduce the effort required for deck removal from around these studs and the probability of damaging the studs and the girder top flange.

For concrete girder-to-concrete deck connections, the main emphasis was placed on the development of a debonded interface with a shear key placed in the top flange of the concrete girder. Test results showed that this debonded surface in conjunction with extended vertical shear stirrups provides adequate horizontal shear transfer to ensure composite action. The debonded surface with less congestion of reinforcement enhanced the deck removal.

In addition to the laboratory tests, other tasks were conducted to correlate the experimental results to field applications. The research team prepared special provisions for removal of existing bridge decks. Also, a series of timelines was produced to compare the speed of construction for various deck replacement systems and determine how certain systems may yield time-saving benefits. Finally, cost estimates were produced for the various systems to provide a means of comparison.

BRIDGE DECK SYSTEMS

Five different bridge deck systems were tested in the Structures Laboratory of the University of Nebraska–Omaha. These were two full-depth CIP deck systems, two precast deck sub-panel systems, and one full-depth precast deck system.

One of the full-depth CIP systems tested was reinforced with conventional reinforcement. This system was designed by the strip method given in the AASHTO LRFD specifications (*7*) to study the effect of reinforcement reduction compared to AASHTO standard specifications (*6*). The test results of this system provided a baseline for comparison with the other four deck systems. The other full-depth CIP system was reinforced with epoxy-coated welded wire fabric and was designed in accordance with AASHTO standard specifications (6). To minimize the amount of reinforcement over the girder lines to expedite replacement of the deck, the bottom layer of welded wire fabric (WWF) was not continuous over the girder lines.

One of the two precast deck subpanel systems tested used 3-in. (75-mm) thick conventional SIP panels with a 6-in. (150-mm) thick CIP topping. After careful evaluation of the test results of this system, the research team designed a continuous SIP panel system that included slab cantilevers. The other full-depth precast system, designed and tested in response to the desire to develop improved full-depth precast systems, incorporates a 4.5-in. (114-mm) precast prestressed panel and a 4.5-in. (114-mm) CIP topping. Also, it is 10 percent thinner and 20 percent lighter than a conventional full-depth CIP deck.

Appendixes E through O provide detailed information on design assumptions and procedures; specimen details; and test setup, procedures, and results. Tables 1 and 2 summarize the observations and data obtained during fatigue and ultimate load tests, respectively, and include comparisons with calculated values.

Full-Depth CIP Deck Systems

Full-Depth CIP Deck Systems with Conventional Reinforcement

Structural Behavior at Ultimate Load. Initially, the load was applied at four points and was increased until it reached 400 kips (1,780 kN), i.e., 100 kips (445 kN) per point—the maximum capacity of the hydraulic jack. No signs of failure were observed, but hairline cracks were observed both at the top and the bottom surface of the deck. The test setup was then rearranged to apply the load at the two central points between the girders. When the jacking load reached 140 kips (623 kN), or 70 kips (311 kN) per point, a compression failure occurred at the top surface near the midpoint between the girders.

Comparison with Theory. Three modes of failure were checked in order to compare the theory with actual test results: (1) two-way shear failure, (2) one-way shear failure, and (3) flexural failure. For each mode, three values were evaluated: (1) design value based on the material design assumptions ($f_c' = 4.0$ ksi and $f_y = 60$ ksi), (2) predicted value computed from the actual strength of the materials used ($f_c' = 6.5$ ksi and $f_y = 95$ ksi), and (3) failure value obtained from the test. Slab design according to the AASHTO standard and LRFD is based on flexure only; one- and two-way shear are not considered. For calculations of the design and predicted values, the 0.5-in. (12-mm) integral wearing surface, provided for profile adjustment, is not considered.

Full-Depth CIP Deck System with Welded Wire Fabric

Structural Behavior Under Cyclic Load. It was noted that after 100,000 cycles, the top surface of the panel started to crack over the girders. It is believed that the plain WWF activates the crack formation at early stages of cyclic loading. A transverse crack was also observed at the middle of the bottom surface of both cantilevers after 700,000 cycles, as shown in Figure 10. These cracks could be seen on the side face of the panel extending vertically from the bottom to a height of 3 in. (76 mm). The research team believed that these bottom cracks were due to the discontinuity of the test panels and that these cracks could be eliminated, if the panels were continuous. No cracks were observed on the bottom surface between the girders (positive moment area).

Figures 11 and 12 show the relationship between the applied monotonic service load and the resulting concrete stresses in the panel deck before and after the cyclic loading. The strains were measured and the stresses calculated at locations of maximum positive and maximum negative moments. For the maximum positive moment zone, load-stress curves obtained before and after the cyclic load were almost identical. However, for the maximum negative moment zone, a 175-percent stress increase was recorded between the initial and final cycles of cyclic load, because of the concrete cracking at the top surface and the reduction of the concrete area subjected to compression.

Structural Behavior at Ultimate Load. As the load increased, the crack width over the top surface started to increase, and cracks were formed in the bottom surface of the panel between the girders. A one-way shear crack was seen at a jacking load of 375 kips (1,668 kN), or 94 kips (418 kN) per point, at the left support edge. Eventually, when the jacking load reached 400 kips (100 kips/point), a one-way shear failure occurred at the same location. It was also noted that the shear cracks had started from the end of the positive moment reinforcement, which was discontinued over the girder. This discontinuity created a stress concentration zone that helped to start the one-way shear crack.

Precast Deck Subpanel Systems

Conventional Precast Deck Subpanel Systems

Structural Behavior Under Cyclic Load. Cracks occurred after 1,400,000 cycles at the top surface of the panel over the girder lines. One crack formed near the centerline of each girder and covered the entire width of the panel. A transverse crack at the bottom surface of the cantilevers was observed after 1,500,000 cycles, as shown in Figure 13. These cracks extended to the sides of the panel to a height of 4 in. (100 mm) and were consistent with the field observations of actual

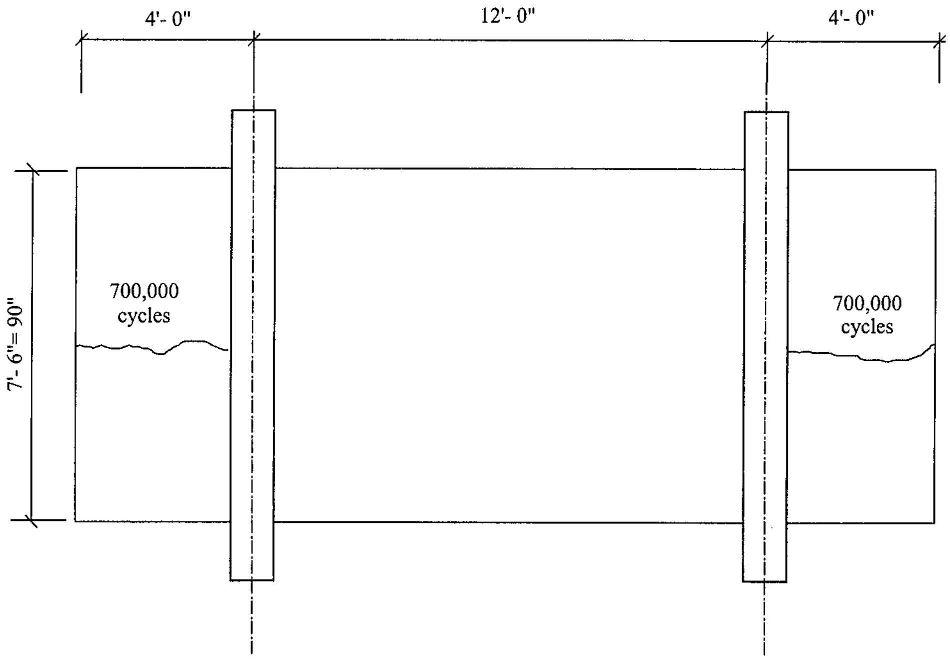


Figure 10. Cracks at bottom surface due to cyclic load.

bridges where the joint between the precast SIP panels extends as a reflective crack into the cantilevers. No cracks were observed on the bottom surface of the 3-in. (75-mm) prestressed precast panels between the girders in the positive moment area. Also, no slippage between the 3-in. prestressed precast panel and the 6-in. CIP concrete topping was observed.

Figures 14 and 15 show the relationship between the applied monotonic service load and the resulting concrete stresses in the panel deck before and after the fatigue loading. The strains were measured and the stresses calculated at locations of maximum positive and maximum negative moments. For the maximum positive moment zone, the load-stress curves obtained before and after the fatigue load are almost identical. For the maximum negative moment zone, stresses after cyclic loading were about 50 percent higher than those obtained initially because of the reduction in the compression area following the cracking of the section.

Structural Behavior at Ultimate Load. As the load increased, the cracks at the top surface over the girders started to increase, while no crack was observed at the bottom surface between the girders. At a load of 70 kips (311 kN) per point, a curved crack occurred over the top surface near the left girder, as shown in Figure 16. The research team believed

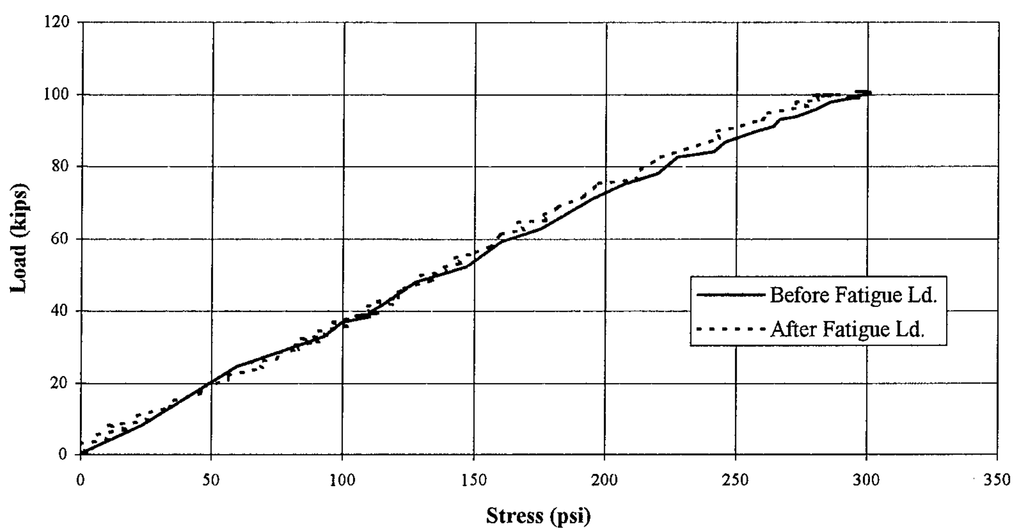


Figure 11. Load-stress relationship at maximum positive moment section.

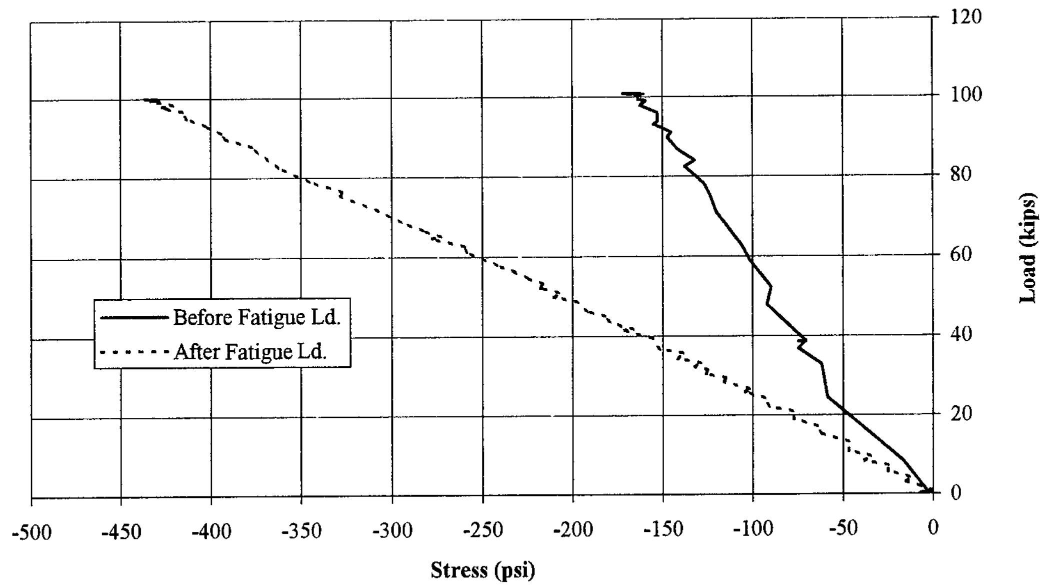


Figure 12. Load-stress relationship at maximum negative moment section.

that this crack initiation was due to the effect of kickout of the prestressed panels because they were not connected together, which is fundamental in the design, and also because the whole system was not connected in the longitudinal direction. At this stage, no slippage between the prestressed panel and CIP concrete was observed. At a load of 75 kips (333 kN) per point, a sudden one-way shear failure and delamination took place near one of the girder supports, as shown in Figures 17 and 18, respectively. The beam shear failure occurred because of the lack of development length of the prestressed strands in the precast panel, which was adequate for service loads but not for the ultimate load test. The research team believes that the combination of the loss of bond of the prestressed strands and the large difference in concrete strength between the two layers, 9,500 psi (65.5 MPa) for the prestressed panels and 6,500 psi (44.8 MPa) for the CIP concrete topping, precipitated the delamination and the beam shear failure. Because delamination between the prestressed panel and the CIP slab took place after the slippage of the strands, the CIP slab alone had to carry the applied shear force, which exceeded the capacity of the concrete section, thus leading to the failure of the system.

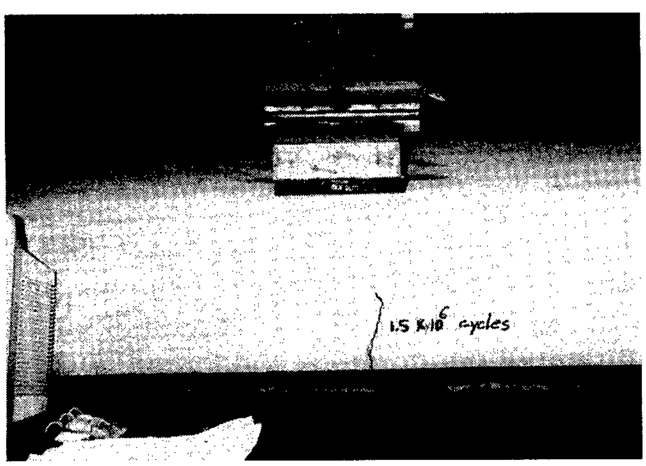


Figure 13. Bottom surface crack extended to midheight of the deck.

Continuous Precast Deck Subpanel Systems

The research team recognizes that systems using precast subpanels with a CIP topping have certain inherent disadvantages. These include the need for forming for the overhangs and handling a large number of precast panels, the formation of reflective cracks over the transverse joints between the SIP panels, and the system's low capacity because prestressing reinforcement is not fully developed. A continuous precast deck subpanel system was recommended to avoid these disadvantages.

Description of the System. To provide a detailed description of the system, a two-lane bridge with shoulders on both sides was considered. The deck was continuous over three 12-ft (3,658-mm) spans, with a full width that included two 4-ft (1,219-mm) overhangs, thus yielding a 44-ft (13,411-mm) wide bridge. It was assumed that the four supporting steel girders would have 12-in. (305 mm) wide top flanges.

The deck system would consist of 4.5-in. (114-mm) precast prestressed subpanels; leveling devices and a grout stop; and a 4.5-in. (114-mm) thick CIP concrete topping. Figure 19 shows details of the deck system.

As shown in Figure 19, the precast subpanel is 44-ft (13.4-m) long and covers the entire width of the bridge. The

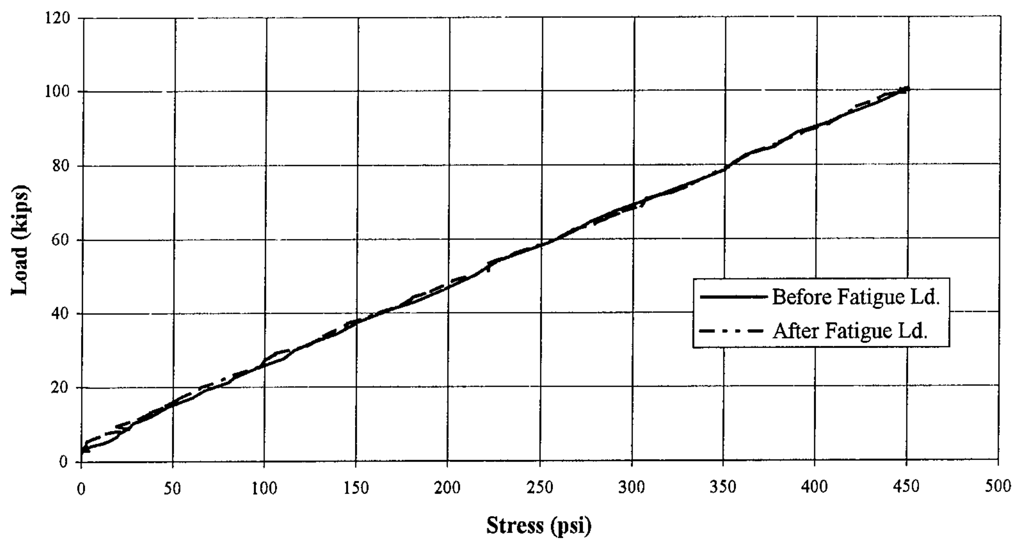


Figure 14. Load-stress relationship at maximum positive moment section.

panel width can range from 4 to 12 ft (1.2 to 3.65 m), depending on the transportation and lifting equipment available. In this study, an 8-ft (2.4-m) width was chosen. An 8-in. (203-mm) wide gap is maintained along the centerline of the girder to accommodate the shear connectors. For the purpose of designing this subpanel system, high-strength concrete is assumed with a specified concrete strength of 4.0 ksi (27.58 MPa) at release and 10.0 ksi (68.95 MPa) at service. The panel is pretensioned with sixteen ½-in. diameter, 270 ksi (1,862 MPa) low relaxation strands. The strands are provided in two layers and uniformly spaced at 12 in. (305 mm), as shown in Figure 19. A minimum clear concrete cover of 1 in. (25 mm) is provided for both the top and bottom layers of strands.

To maintain the 8-in. (203-mm) gap over the girder and to transmit the prestressing force over the gaps, 28 #6, 44-in. (1,118-mm) long reinforcing bars are provided in two layers, as shown in Figure 19. These bars provide 18 in. (457 mm) of embedment length to transmit the compression force over the gaps from one part of the panel to the adjacent part.

To maintain continuity in the longitudinal direction between the adjacent precast panels, shear keys and reinforced pockets are provided at 2 ft (610 mm) on center. Figure 20 shows the details of the shear key and reinforced pockets. To avoid additional forming in the field, 20-gage, 6- × 6-in. (152- × 152-mm) sheet metal stock is used as an SIP form at the pockets. The panel is reinforced longitudinally with #4 bars spaced at 2 ft (610 mm) at the locations of the pockets. To provide for ten-

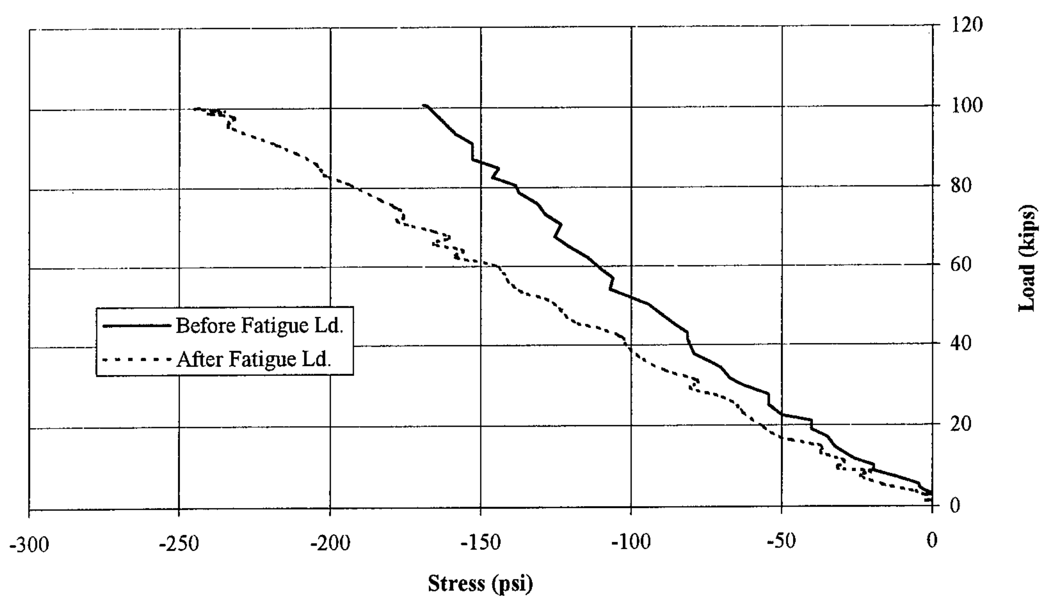


Figure 15. Load-stress relationship at maximum negative moment section.

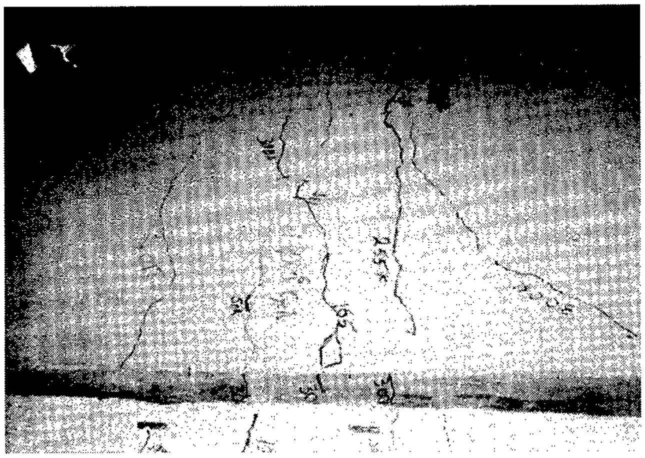


Figure 16. Cracks on top surface over the girder line.

sion development, these bars are spliced using a loose 9-in. #4 bar and a spiral, as shown in Figure 21. This technique was evaluated separately with a tension specimen and found to produce the full bar yield strength of 60 ksi (414 MPa).

Figure 22 shows the details of a simple leveling device that was proposed to level the panels when set over the supporting girders. The device consists of a ½- × 2- × 4½-in. (12- × 50- × 115-mm) plate with a ¾-in. (19-mm) nut welded to its bottom surface. A ¾-in. (19-mm) bolt is inserted through a hole in the plate and threaded through the nut to allow for leveling of the panel by turning the bolt. The plate is mounted between the top flange of the girder and the lower layer of reinforcement in the precast panel. At least two assemblies should be provided in each gap. The leveling device could be inserted in the 1-in. (25-mm) space between the reinforcing bars and the strand to save room for the shear connectors. A grout barrier should be installed along the girder edges before the precast panels are set on the supporting girders. This grout stop could be formed from light gauge metal sheets

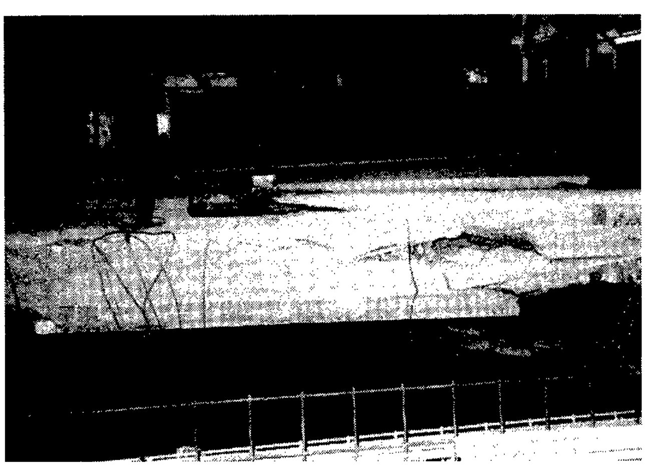


Figure 17. One-way shear failure.

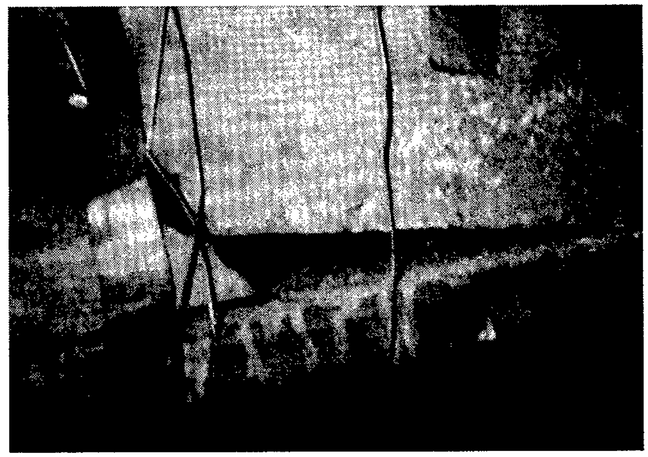


Figure 18. Separation of precast panel from CIP concrete topping.

attached to the top surface of the girder, using a construction adhesive.

Once the panels are placed over the girders and adjusted with the leveling devices, the gaps over the girders are grouted with a flowable mortar mix to about 1.5 in. (38 mm) below the top surface of the panel. The mortar mix should have a compressive strength of 4,000 psi (27.58 MPa) at the time of casting the topping slab. The mortar would provide the compression block needed to handle the negative moment over the girders due to loads imposed by the deck finishing machine and the concrete topping self weight. The mortar would also provide bedding for the precast panel over the girders.

As shown in Figure 19, the system includes a 4.5-in. (114-mm) thick CIP topping slab placed over the precast subpanel. This slab is reinforced with epoxy-coated welded wire fabric.

Deck construction involves the following steps:

- Clean the girder surface by grit blasting.
- Glue the grout barrier to the edges of the top surface of the girders.
- Install the precast panels, insert the leveling device, and adjust the panels with the leveling devices.
- Install a backer rod between adjacent panels to prevent leakage during the casting of the CIP topping slab.
- Fill the 8-in. (203-mm) gaps over the girders with a flowable mortar mix or a rapid-set nonshrink grout.
- Install the #4 bar splices in the pockets and adjust the spiral bars into position.
- Install the welded wire fabric reinforcement for the CIP topping slab.
- Place the CIP topping concrete and fill the shear keys and the pockets.

Structural Behavior Under Cyclic Load. After 700,000 cycles, two hairline cracks were noted on the top surface of

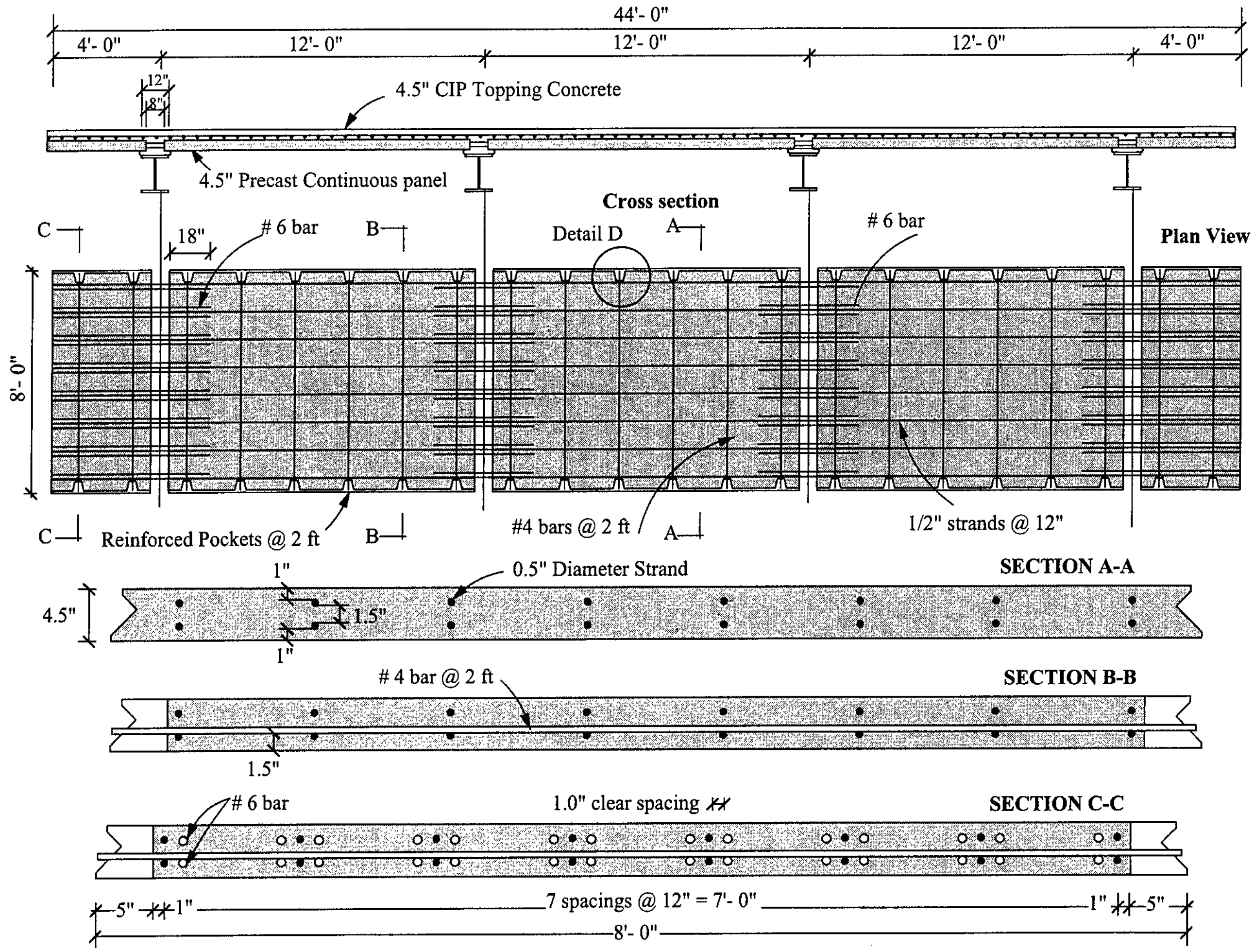


Figure 19. Details of the continuous precast prestressed panel.

the panel, one crack over each girder, as shown in Figure 23. These cracks started over the transverse joint between the precast panels and spread out in the longitudinal direction. After 2,000,000 cycles, both cracks extended to about two-thirds of the panel width, and no cracks were observed either on the top surface of the overhang or on the bottom surface between the girders (positive moment area). The number, size, and length of cracks observed in this system were much less than those observed during tests on the conventional precast deck subpanel with a 3-in. (75-mm) SIP panel. The research team felt that this improvement occurred because the connections between the 4.5-in. (112-mm) precast SIP panels stiffened the system and led to better load distribution. It was also noticed that the cracks which had been formed during fatigue loading were closed after removing the load. This was attributed to the continuity of the prestressed SIP panel in the transverse direction over the girder lines.

Figures 24 and 25 show the relationship between the applied monotonic service load and the resulting concrete stresses in the panel deck in the maximum positive and maximum negative moment zones, respectively. The figures also show the load-stress relationship before and after the fatigue loading.

For the maximum positive moment zone, load-stress curves obtained before and after the fatigue load were almost identical. However, for the maximum negative moment zone, the stress decreased between the initial and final cycles of fatigue load. This could be attributed to the self-healing abilities of the prestressed precast concrete.

Structural Behavior at Ultimate Load. The load was applied over the transverse joint between the precast panels and was increased until it reached 400 kips (1,780 kN), i.e., 100 kips (445 kN) per point—the maximum capacity of the hydraulic jack. No signs of failure were observed, but hairline cracks were observed both at the top and the bottom surface of the deck. Deflections at midspan and at the end of the overhang were 0.35 in. (9 mm) downward and 0.56 in. (14 mm) downward, respectively. The test setup was then rearranged to apply the load at the two central points between the girders. When the jacking load reached 140 kips (623 kN), or 70 kips (311 kN) per point, a compression crushing occurred at midspan between the gird-

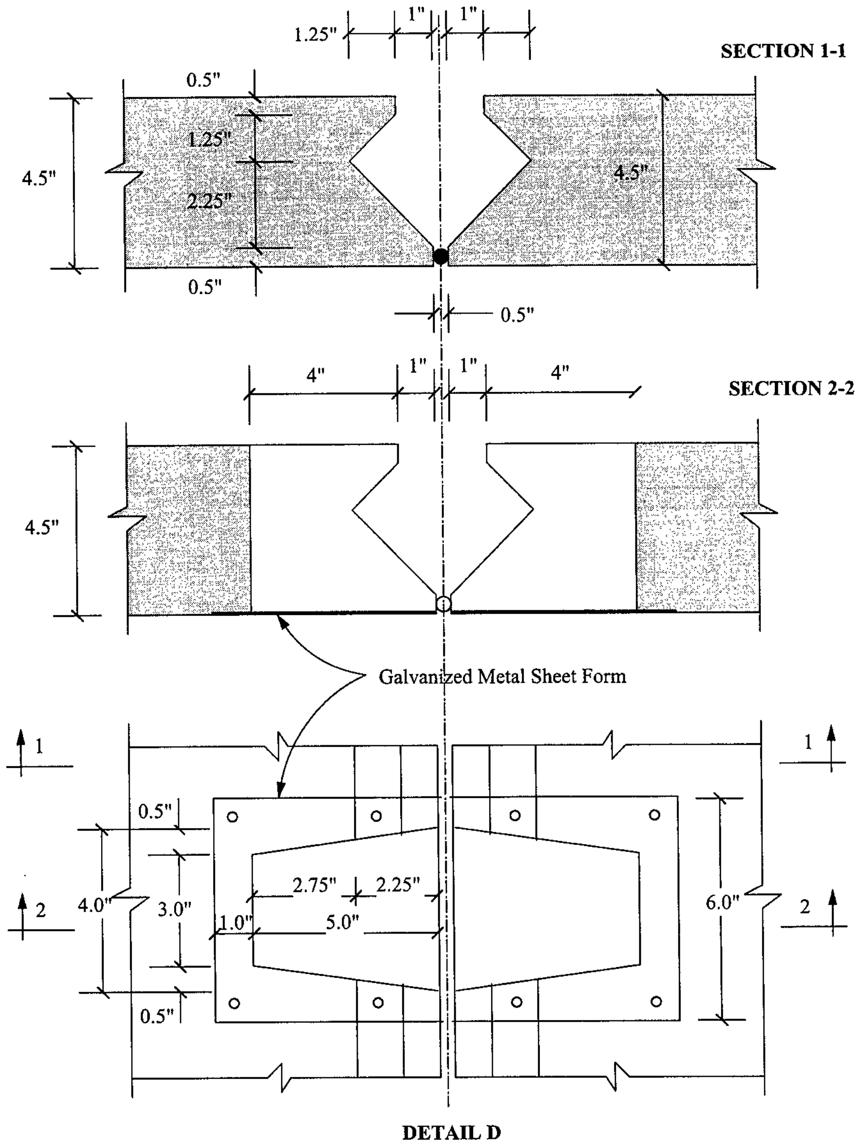


Figure 20. Transverse joint detail.

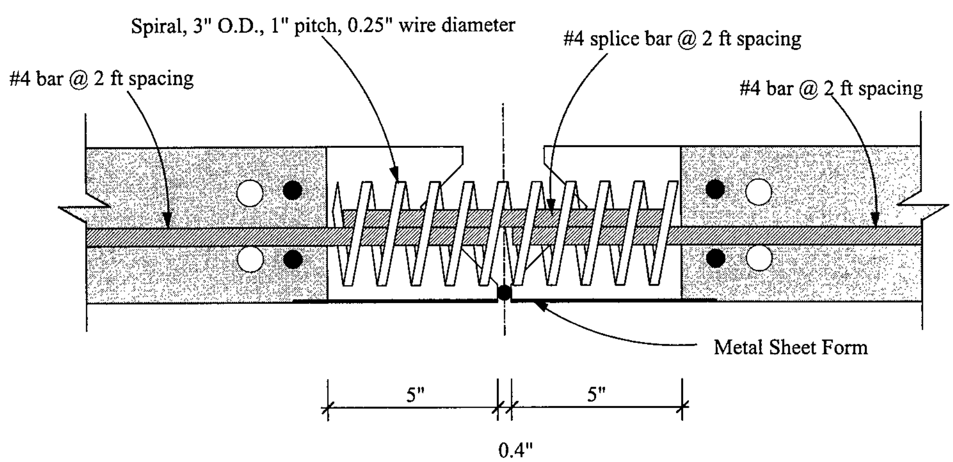


Figure 21. Reinforced pocket detail.

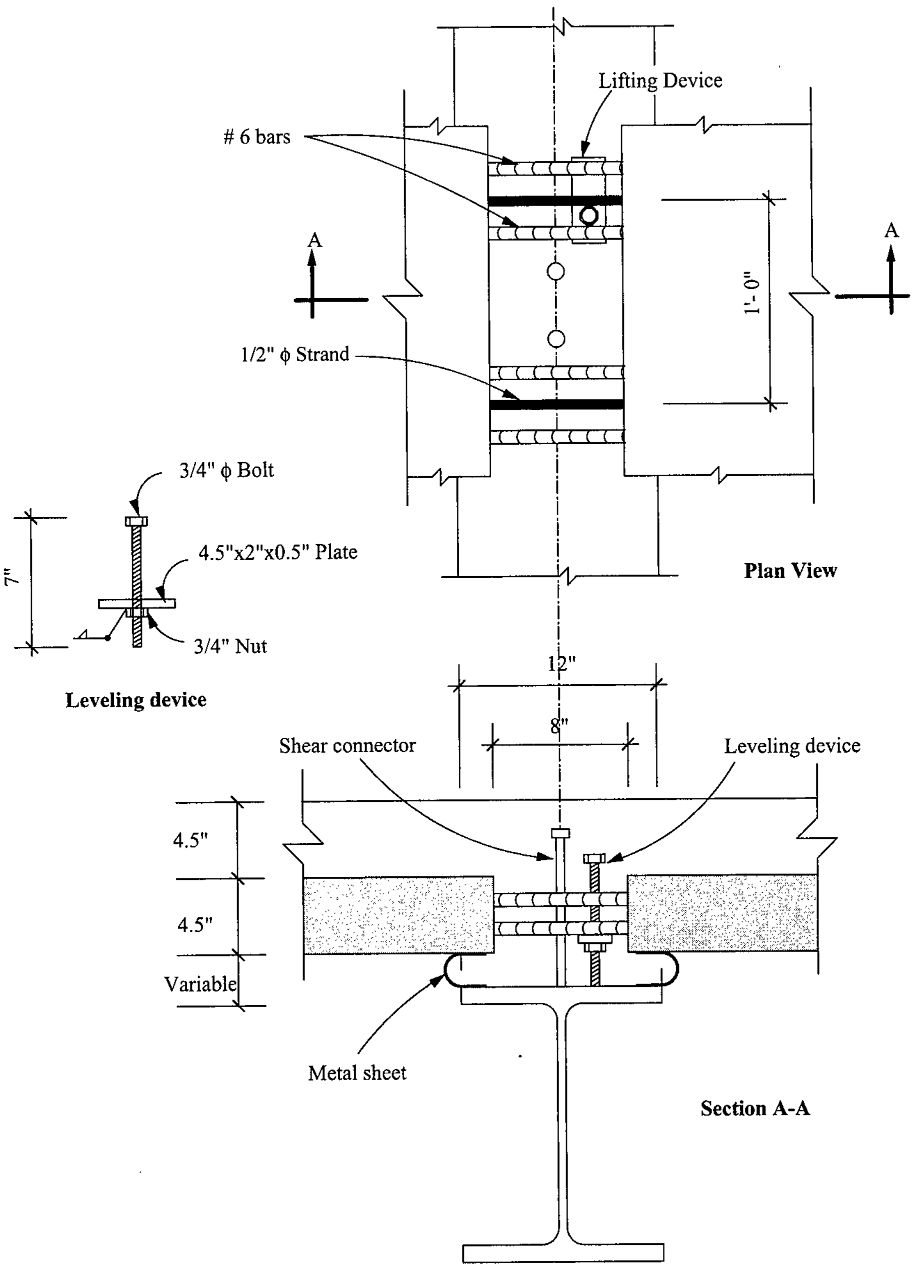


Figure 22. Leveling device.

ers at one side and one-way shear cracks were observed at the other side. Examination of the precast panel and the CIP slab revealed that the shear cracks took place in the thin nonreinforced concrete that was filling the transverse shear key. Deflections at midspan and at the end of the overhang, at failure, were 1.60 in. (40 mm) downward and 0.88 in. (22 mm) upward, respectively. After removing the load, the deck returned to its original shape, and no residual deflection was noticed.

Advantages of Continuous Precast Deck Subpanel Systems. Compared with the conventional SIP precast panel system, the continuous precast deck subpanel system has the following advantages:

- The materials used in producing the precast panels are readily available, not expensive, and not proprietary.
- Field forming for deck overhangs is eliminated, and fewer precast panel pieces would have to be handled.
- SIP panels could be crowned to form the cross slope of the bridge.
- Continuity of the SIP panels in the longitudinal direction results in a better load distribution and eliminates reflective cracks at the transverse joints.
- Continuity of the transverse reinforcement over the girder lines leads to better performance.
- Under cyclic loading, the system exhibits only minor top cracks over the girder.

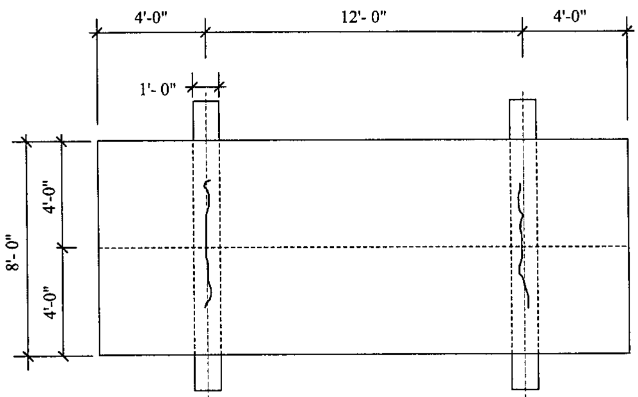


Figure 23. Cracks on top surface.

- The system allows full prestress transfer at maximum positive moment sections, thus preventing sudden one-way shear failure.
- The system exhibits a substantially higher capacity than that of the conventional SIP concrete panel system.

Comparison of Different CIP Deck Systems

Table 1 summarizes the observations made during fatigue test. Table 2 compares the capacity of the different deck systems. In this comparison, three modes of failure—one-way shear, two-way (punching) shear, and the flexure—were considered in evaluating the different systems. For each mode of failure, the behavior was estimated using assumed material properties and also using actual material properties and was determined from test results. Figure 26 compares the flexural capacity of the different systems as determined from the tests. Figure 27 compares the duration of construction for these systems based on the construction time of the laboratory test specimens. From these comparisons, the following conclusions can be made:

- Using welded wire fabric reinforcement instead of conventional reinforcement in the full-depth CIP deck slabs reduces reinforcement placement time by about 30 percent.
- Using the conventional SIP panel or the continuous SIP panel systems with the welded wire fabric reinforcement reduces the construction time by about 50 percent, as compared with the full-depth CIP deck system reinforced with conventional reinforcement.
- Under fatigue loading, the continuous SIP panel system shows a superior performance over the conventional SIP panel system and the full-depth CIP with WWF. The most important features are the self-healing of the cracks following the load and the relatively smaller width and length of cracks compared with those of other systems.
- Predicted capacities are in good agreement with test values.
- For full-depth CIP systems, anchoring the positive moment reinforcement over the girder lines by extending it at least up to the tension development length helps the development of arching action in decks.
- For the conventional SIP panel system, it is necessary to develop strands by means of mechanical anchors or by extending the strands outside the panel and embedding them in the CIP slab.
- The continuous SIP panel system has twice the capacity of the conventional SIP panel system and has shown more ductile behavior.

Full-Depth Precast Deck Systems

Description of the System

The full-depth precast deck system developed as part of this project incorporates precast prestressed concrete panels, welded headless studs, welded threaded studs, grout-filled

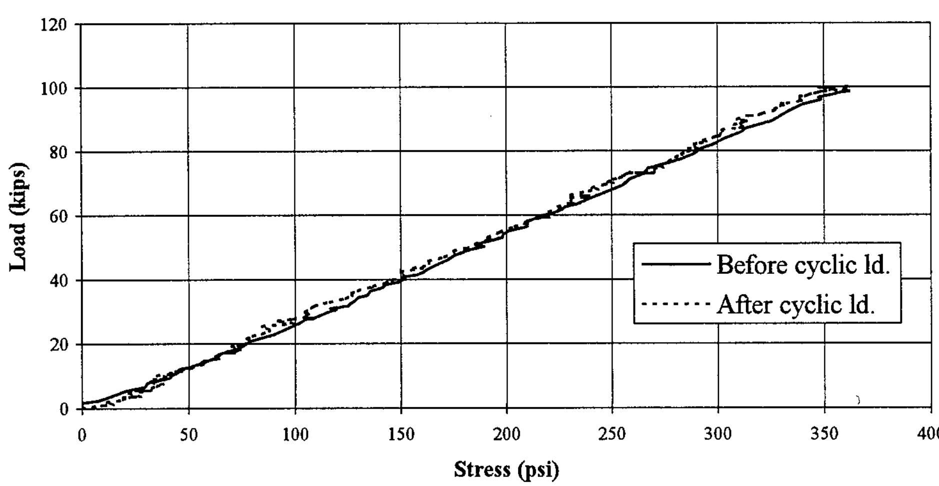


Figure 24. Load-stress relationship at maximum positive moment section.

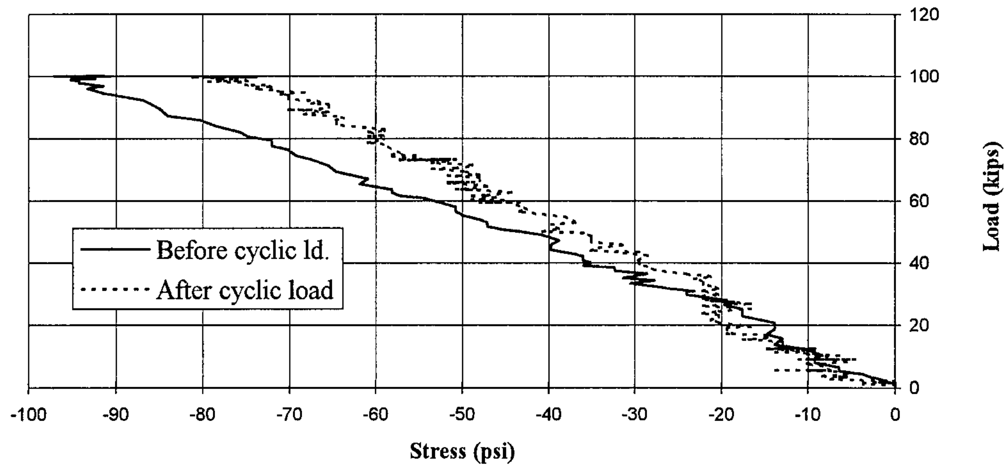


Figure 25. Load-stress relationship at maximum negative moment section.

shear keys, leveling bolts, and threaded bars for post-tensioning. The panels were extended to the full width of a bridge and transversely pretensioned and longitudinally post-tensioned. This system was about 10 percent thinner and 20 percent lighter than conventional reinforced CIP or precast prestressed solid concrete systems. An overview of the system is shown in Figure 2.

The specified 28-day compressive strength of concrete for the precast panels was 5,000 psi at transfer of prestress and 7,500 psi (51.71 MPa) at 28 days. Prestressing was provided with ½-in. diameter, 270 ksi, low relaxation indented wire strands. Mild reinforcing and welded wire fabric for the panels use Grade 75 steel.

Figure 28 shows a transverse cross section of the precast panel and steel arrangement. The primary advantages of a multi-stemmed section were to reduce self weight and the amount of longitudinal post-tensioning. The solid slab thickness was determined to accommodate top strands and welded wire fabric with the required clear cover specified by AASHTO standard specifications (*6*). Dimensions of the cross section were optimized to provide adequate room for the prestressing strands and adequate blockout for the post-tensioning anchorage devices. As shown, three 8- × 19.7-in. (203- × 480-mm) blockouts were provided over the girder lines to accommodate headless shear connectors. These blockouts were grouted from the top surface of the panel through 1-in. (25-mm) diameter tubes.

The precast deck consists of an 8.1-in. (205-mm) thick solid portion at each girder location and a 4.5-in. (115-mm) thick portion between the girders, as shown in Figure 28. As shown in Figure 29, the thick portion at the girder location was necessary to accommodate post-tensioning steel and to eliminate eccentricity of post-tensioning forces. The width of the voids between the girders bounded by the transverse

TABLE 1 Observations during fatigue load

Deck System			
Full-Depth with conventional reinforcement	**Full-depth with epoxy-coated welded wire fabric**	**Conventional precast SIP panel**	**Continuous Precast SIP panel**
Not tested	• Two line cracks over each girder line on the top surface covering the entire width of the panel; started at 0.1×10^6 cycles. • One transverse line crack at the middle of the bottom surface of each overhang; started at 0.7×10^6 cycles and extend to a height of 3 in. on the side of the slab. • Cracks remain visible after removing the load. Stress over the girder lines after the 2×10^6 cycles was 175% higher than those before fatigue test.	• One line crack over each girder line on the top surface covering the entire width of the panel; started at 1.4×10^6 cycles. • One transverse line crack at the middle of the bottom surface of each overhang; started at 1.5×10^6 cycles and extend to a height of 4 inches on the side of the CIP slab. • Cracks remained visible after removing the load. Stress over the girder lines after the 2×10^6 cycles was 50% higher than those before fatigue test.	• One line crack over each girder line on the top surface covering about 2/3 of the panel width; started at 0.7×10^6 cycles • No bottom cracks • Cracks closed after removing the load. Stress over the girder lines after the 2×10^6 cycles was 20% lower than those before fatigue test.

TABLE 2 Results of ultimate load tests

		Deck System			
		Full-Depth with conventional reinforcement system	**Full-depth with epoxy-coated welded wire fabric system**	**Conventional precast SIP panel system**	**Continuous Precast SIP panel system**
Mode of failure		compression due .to flexure	one-way shear	one-way shear	compression due to flexure
One-way shear (kips/ft)	Design[1]	9.0	12.2	9.1	17.6
	Predicted[2]	11.5	**15.2**	**11.6**	**18.8**
	At failure	10.0	**14.0**	**10.0**	**9.4**
Two-way shear (kips)	Design[1]	141.7	163.3	180.3	168.7
	Predicted[2]	180.6	208.2	210.7	190.6
	At failure	70	100	75	70
Flexural at maximum positive moment section (ft-kips/ft)	Design[1]	24.6	42.9	28.6	28.7
	Predicted[2]	**37.2**	47.9	29.6	**29.9**
	At failure	**38.4**	27.8	21.1	**36.1**
Notes	Designed According to	AASHTO LRFD	AASHTO Standard	AASHTO Standard	AASHTO Standard
	Design truck	HL-93	HS25	HS25	HS25
			Positive reinforcement is not continuous over girder lines.	-	
		Ductile failure.	Sudden failure due to loss of anchorage of the positive moment reinforcement.	Sudden failure due to slippage of strands.	Ductile failure, one-way shear cracks on one side and compression failure on the other side.

[1] With assumed material properties
[2] With actual material properties and no reduction factor

stems depended on the spacing between the girders, and they could be formed by using adjustable void forms.

Using a full-depth section over the girders improved the ease of construction. The flat bottom surface provided at girder locations allowed a simple grout stop to be installed between panels and girders. This was done by using oversized continuous expanded polystyrene at the edges of the top flanges.

The function of the transverse joints is to transfer the live loads between adjacent panels. A key consideration of joint details was to avoid water leakage through the joint. Many shear key shapes had been tried in previous projects, and the shear key size and shape shown in Figure 30 were chosen based on the findings of these projects. For production and construction purposes, a 0.4-in. (10-mm) wide gap was allowed between panels.

Properties of the grout material are important in reducing construction duration, in transferring live loads, and in preventing water leakage. A rapid-set nonshrink grout was considered to fill the shear keys and the shear connector pockets. Many types of rapid-set nonshrink grout were investigated

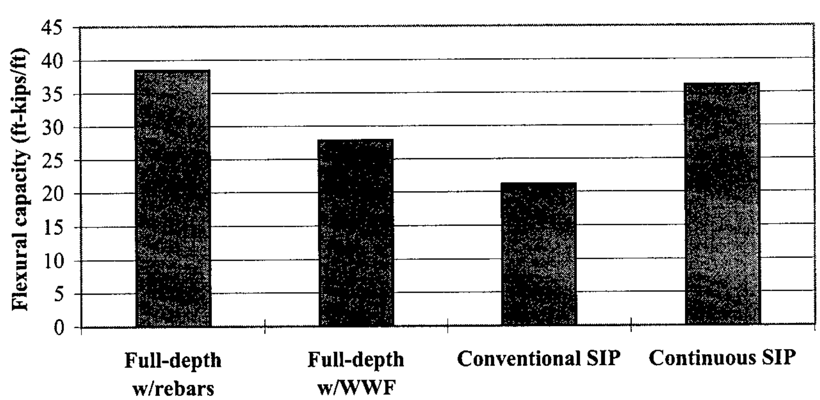


Figure 26. Flexural capacity measured from testing.

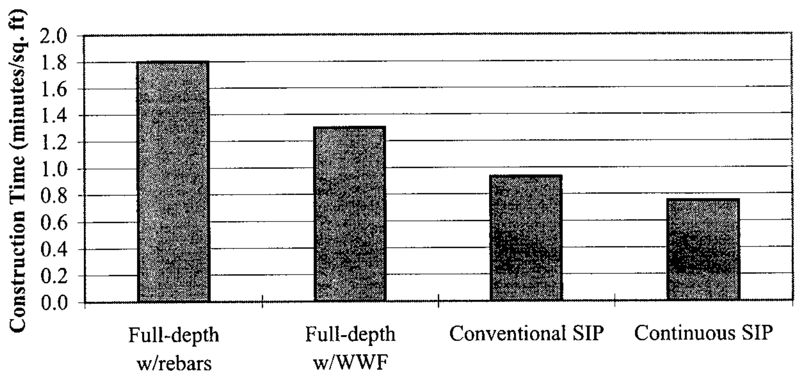


Figure 27. Comparison of construction time.

and Set 45 by Master Builders was chosen for this project. Set 45 gives compressive strength of 5,000 psi (34.5 MPa) after 3 hours.

Longitudinal post-tensioning was used to provide enough compressive force to close transverse cracks at the joint between the panels. It was provided at approximately mid-depth of the panels. To simulate field situations, staged post-tensioning was considered. As shown in Figure 31, post-tensioning tendons are located over the top flanges of girders. Blockouts were provided for anchorage and/or couplers at both transverse edges of the panels. Based on the design criteria, 200 psi (1,379 kPa) of longitudinal compressive stress was required at each girder line. This stress was provided by two 1-in. diam, 150 ksi post-tensioning bars. As shown in Figure 32, the shear connectors for this system consist of welded headless studs and welded threaded studs with nuts.

Prior to panel erection on steel girders, the short headless studs are welded onto the top flange of the girder to accommodate horizontal shear transfer from the deck panels to the girders. Also, long threaded studs are field-welded onto the top flanges, and nuts are threaded onto these studs to clamp the precast panel to the steel girders and provide a means for resisting horizontal shear and uplift. For concrete girder bridges, either inserts or drilled and grouted anchors can be used as shear connectors.

The general procedure for deck construction using the full-depth panels would occur in the following sequence. Once the old deck is removed and new shear studs are installed, the new deck panels would be erected and adjusted to grade by the use of the leveling bolts and tied down by the threaded studs. The keyways between panels would then be grouted with rapid-set grout and longitudinally post-tensioned once the keyway grout had cured. Finally, the pockets over the tops of the girders would be grouted.

Finite Element Analysis

Because of the multi-stem shape, the precast panel's stiffness in the direction of the slab span differs from the stiffness in the other direction. Therefore, the behavior of the panel is different from a solid slab, which leads to a concentration of bending moments and shearing forces around the stem on which the wheel load is applied. Because AASHTO's formulas derive the moment in the deck slab on the basis of a solid slab assumption, using these formulas for a multi-stem deck may be inappropriate. A commercial finite element analysis software package, "ANSYS 50A," was used to investigate the stress distribution in the panel. Using the results from this analysis, appropriateness of the AASHTO formulas for the developed system was confirmed and compared to experimental test results. Also, the uniform distribution of longitudinal compression in the horizontal direction was also confirmed by finite element analysis.

The test specimen was modeled as a three-dimensional structure. Because of the structure's symmetry, one quarter of the structure was modeled with linear-isoparametric 8-node 3-dimensional elements. The level of stresses and the stress distributions over the panels were checked for two conditions: after application of post-tensioning force and under wheel loads.

The resulting stress distributions in the panels are provided in Appendix E. The stresses due to longitudinal post-tensioning were distributed almost equally at the transverse joint nearest to the edge of the panels where post-tensioning forces were applied. In the transverse direction, stresses due to wheel loads were 50 to 60 percent of those obtained from AASHTO formulas. The critical stresses determined from the finite element analysis were the longitudinal tensile stresses at the bottom surface of the slab underneath the wheel load between stems. The tensile stress at this location was 617 psi (4.25 MPa), which exceeded the allowable concrete tensile stress calculated as ($6\sqrt{f_c'}$), i.e., 520 psi (3.58 MPa). Thus, welded wire fabric reinforcement, 6 × 6 - W5.5 × W5.5, was provided as reinforcement in this area to accommodate the tensile load.

Structural Behavior

Structural Behavior Under Cyclic Loading (Load Configuration 1). To test the performance of the transverse shear

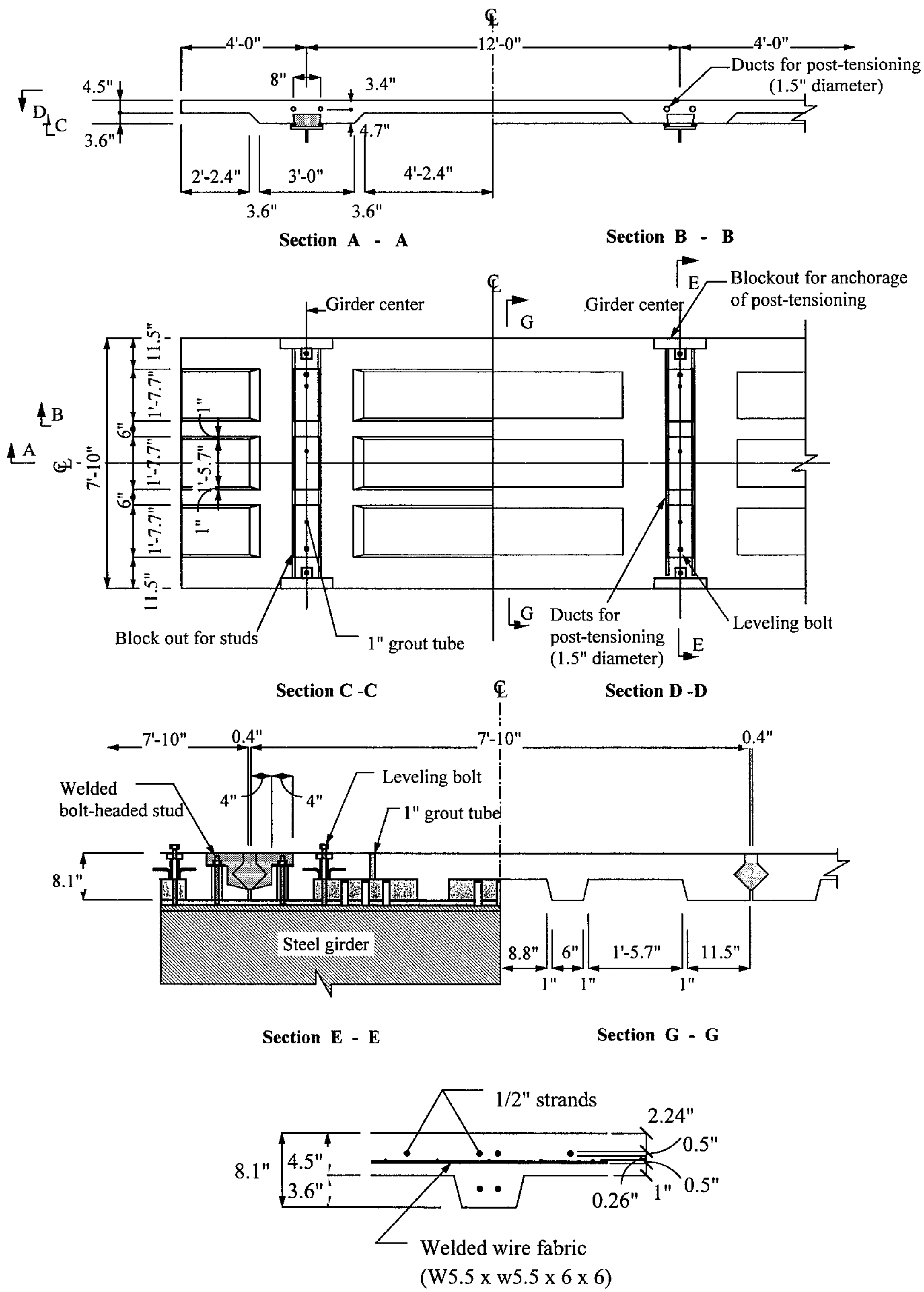


Figure 28. Details of precast prestressed concrete panel system.

key, a cyclic load was applied near the edge of the center panel, as shown in Figure 33, for 2 million cycles. Strain gages were installed on both sides of the shear key on top and bottom surfaces. Figure 34 shows the relationship between the applied monotonic service load of 25 kips (111 kN) per loading point (simulated rear wheel of HS25 load plus impact) and the resulting concrete stresses in the deck before and after the cyclic loading. The load-stress relationships before and after the cyclic loading were almost identical at most locations. In addition, there were no cracks or water leakage during the fatigue loading. Figure 33 shows stresses in the concrete due to the factored load applied after fatigue

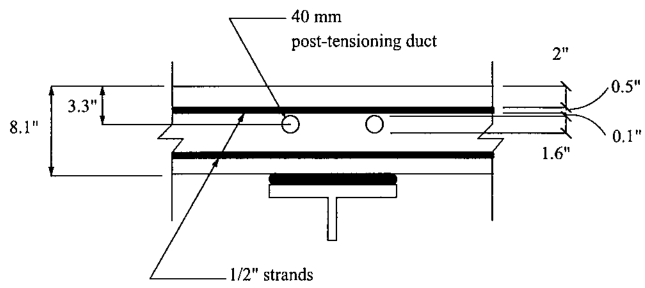


Figure 29. Typical steel arrangement.

loading. Stresses on both the loaded side and the unloaded side of the transverse joint were about the same for maximum positive and negative moment zones, indicating an effective load transfer across the joint. Deflections due to service load were 0.083 and 0.008 in. (2.11 and 0.20 mm) at midspan and at the end of the overhang, respectively. Deflection to span ratios were less than 1/1,000.

Ultimate Loading for Load Configuration 2. Figures 35 and 36 show the concrete stresses on the top and bottom deck surfaces, respectively, for Load Configuration 2. The stress distribution in the transverse direction agrees with that determined from finite element analysis. These stresses were approximately 50 percent of those computed based on the AASHTO formula. No cracking was found at this service load stage. At 45 kips/loading point, transverse cracking occurred at the bottom surface underneath the loading point between girders; however, there were no additional cracks at factored load. As shown in Figure 37, some flexural cracks occurred at 80 kips/loading point.

Longitudinal cracks at the girder locations were due to negative moment at ungrouted post-tensioning ducts. Oval-shaped cracks around two loading points between the girders and a bell-shaped crack around each loading point on the overhanging slab were because of flexure in two-way slab action. Because the specimen carried more than the capacity of the hydraulic jack, the test setup was rearranged to apply the load at the two interior points only. The panel suddenly

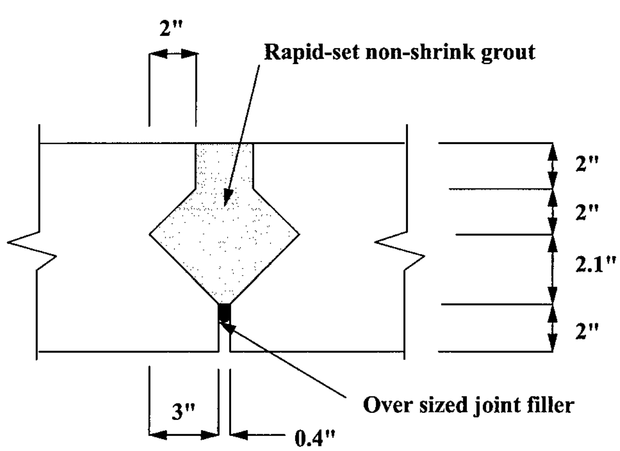


Figure 30. Transverse shear key.

failed at 122 kips (533 kN) per loading point due to punching shear at one of the loading points, as shown in Figure 38. However, the computed punching shear capacity was 70 kips (311 kN)/loading point.

From the test results, it was obvious that the AASHTO formula very conservatively predicts punching shear capacity. This formula was based on a truss model with the concrete acting as compression struts and the steel reinforcement as tension chord. Based on this mechanism, the depth of steel reinforcement controls punching shear capacity because a deeper reinforcement gives a deeper truss element.

The AASHTO formula does not account for the effects of reinforcement and prestressing. However, the shear capacity is influenced by tension steel across the shear planes and friction coefficient between the shear planes. Also, prestressing provides an added shear capacity. When the slab is heavily prestressed, like the test specimen, the punching shear capacity is significantly increased.

By inspecting the specimen after failure, the research team found that the two stems adjacent to the loading points failed in shear at about 1 ft from a girder location, indicating that the stems behaved as a beam after the transverse crack occurred between the stems. The specimen continued to carry load even after the two stems' shear failure by means of the surrounding solid slab reinforced with welded wire fabric and the other stems.

The ultimate load of the test specimen was approximately 190 percent of the predicted capacity, which indicated an excellent overall behavior of the deck system. In addition, no cracks or unusual phenomenon were found at the transverse joint during the ultimate loading.

Structural Behavior Under Edge Loading (Load Configuration 3). As shown in Figure 39, very high stress occurred at the edge of the panel under the service loading conditions. The figure also shows cracks that occurred at 80 percent of service load. These cracks occurred because of the panel discontinuity and partly due to lack of stiffness and reinforcement in the longitudinal direction. Stresses were approximately four times the amount of those for Load Configuration 1 where panels were made continuous by the grouted transverse joint. The transverse cracking at the free-end side of the second stem from the edge indicated that the tensile strength in the longitudinal direction had been reached at the location where there was a change of thickness. These results confirmed the need to maintain continuity between the old and new sections when the deck was temporarily open to traffic.

Removal of Panels. Precast panels were removed one by one after the post-tensioning bars were detensioned and removed. Nuts used with threaded studs were also removed before starting panel removal. A schematic of the panel removal is shown in Figure 40.

Panel removal was initially attempted by jacking Panel 3 at Location 1 without transverse joint demolition. However, this attempt failed because the transverse joint was strong

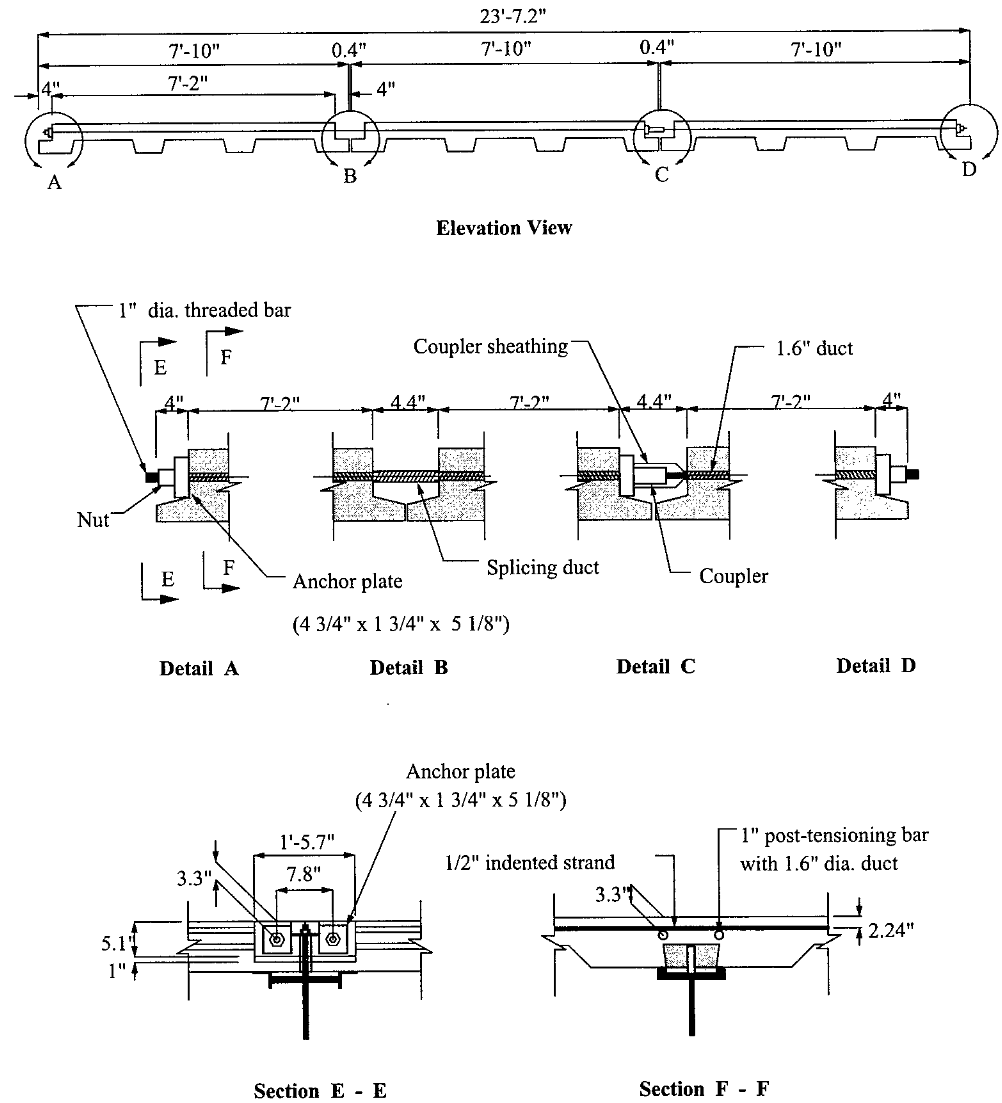


Figure 31. Longitudinal post-tensioning details.

enough and did not permit the separation of Panel 3 from Panel 2. The jacking device was then moved to Location 2, and the transverse joints between panels were demolished by jackhammer, enabling removal of Panel 3 from the steel girders. For Panel 2 removal, hydraulic jacks were applied at Location 3, and that side of the panel was lifted from the girder. The jacking location was then moved to Location 4, and the other side of the panel was also lifted. Finally, the jacking device was moved to Location 5, and Panel 1 was removed. During this removal, the jacking force was measured and the maximum force required to lift the panel from the steel girders was about 30 kips (133 kN) per jack.

Overall removal of the panels was fairly simple, and the short headless studs allowed for easy removal of the precast panels. However, it was found that threaded studs provided resistance to panel removal, which could be attributed to the confined grout in the steel pipes used for the threaded studs.

Design and Construction Details for Full-Depth Systems. The research team has developed details for multi-stage deck construction with the precast prestressed full-depth panels. Design and construction details using these precast panels are given in Appendixes F and H.

GIRDER-TO-DECK CONNECTIONS

Concrete Girder-to-Deck Connections

In a composite concrete beam system, the CIP reinforced concrete slab and underlying reinforced or prestressed precast concrete beam behave as a single unit under the superimposed load. The composite construction improves the structural efficiency of the beam and slab system, which in turn provides an economical way of reducing the depth of the member. However, in a noncomposite construction, the slab

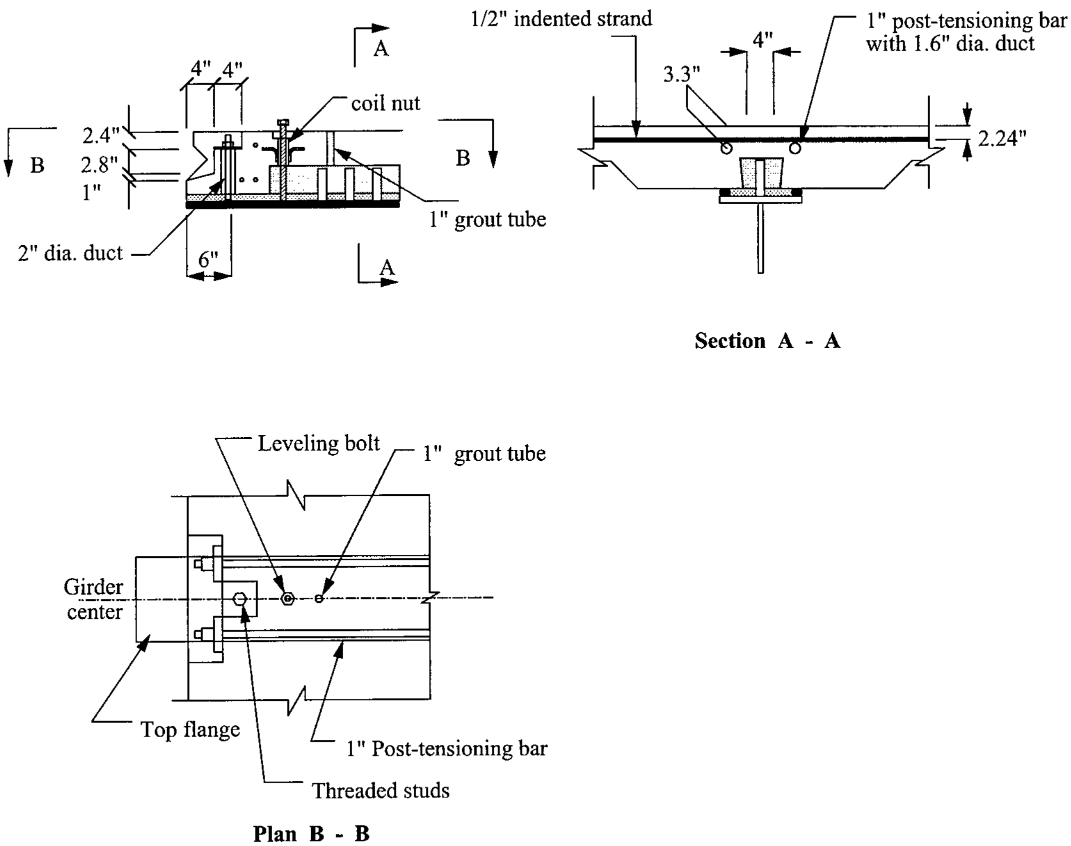


Figure 32. Shear connection details.

and the beams are not connected at the interface, and the two elements tend to separate. In such a system, each element supports only part of the total load in proportion to its respective stiffness. When horizontal shear develops between the two elements, the composite behavior significantly increases both the flexural strength and the stiffness of the joined members as compared with those of noncomposite members. The transfer of horizontal shear at the interface can be accomplished by concrete adhesion between the two surfaces, bond plus vertical steel ties, or debonded shear keys with vertical steel ties. The roughness at the top of the precast concrete beam, the amount of steel crossing the joint, and the concrete strength are the major factors affecting the horizontal shear strength of the interface.

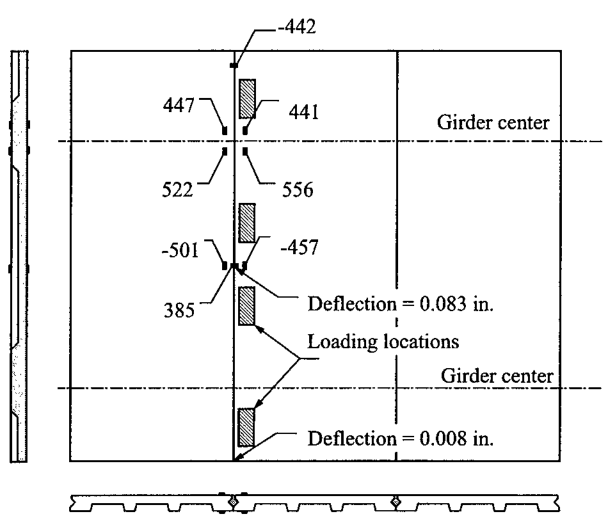


Figure 33. Stresses on top surface of deck slab (in psi).

Push-Off Tests

The test program for concrete girder-to-deck connections consisted of 73 push-off specimens. The purpose of these tests was to evaluate the performance of the different types

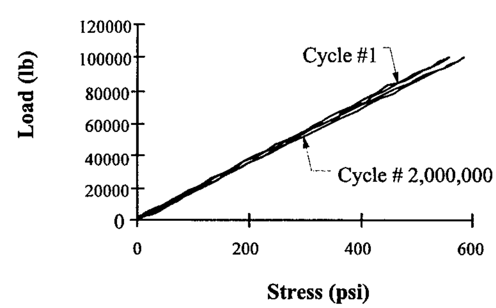
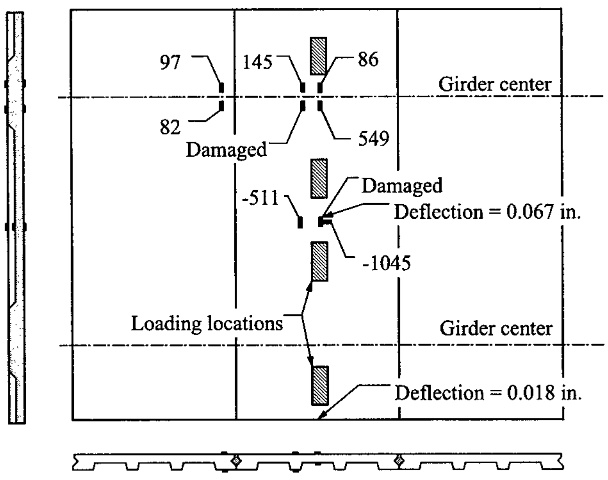
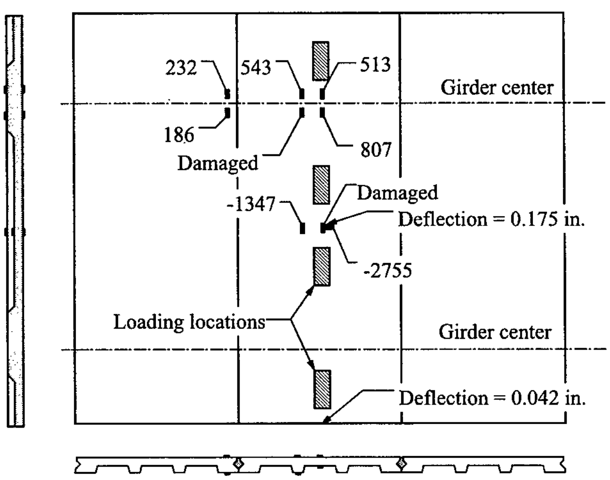


Figure 34. Load-stress relationship.

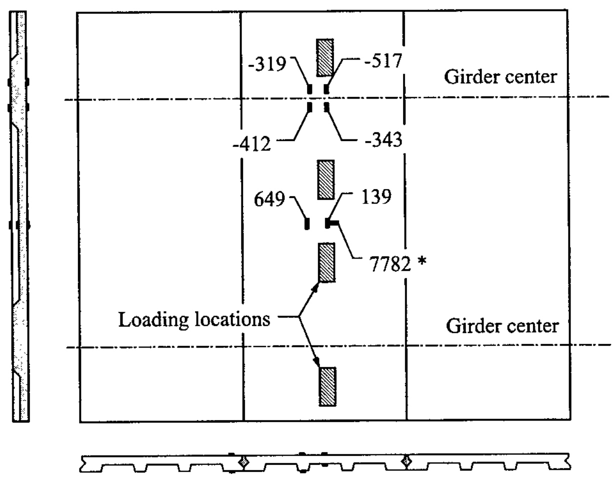


(a) At service load (25 kips/loading location)

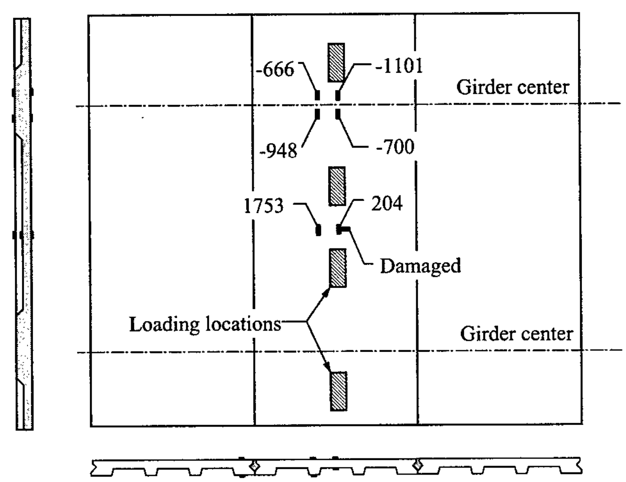


(b) At factored load (55 kips/loading location)

Figure 35. Stresses on top surface of deck slab (in psi).



(a) At service load (25 kips/loading location)
* Crack across the strain gage



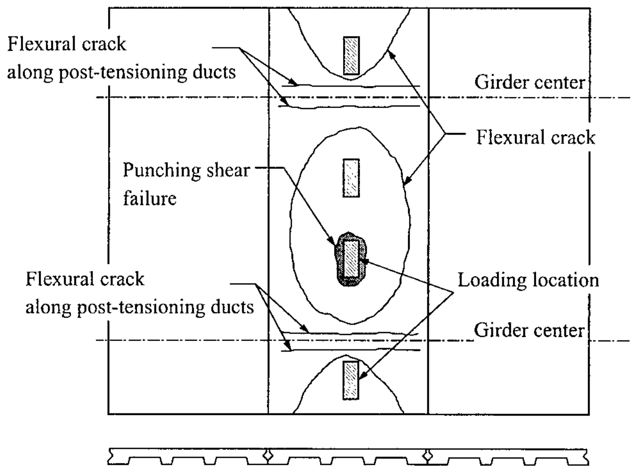
(b) At factored load (55 kips/loading location)

Figure 36. Stresses on bottom surface of deck slab (in psi).

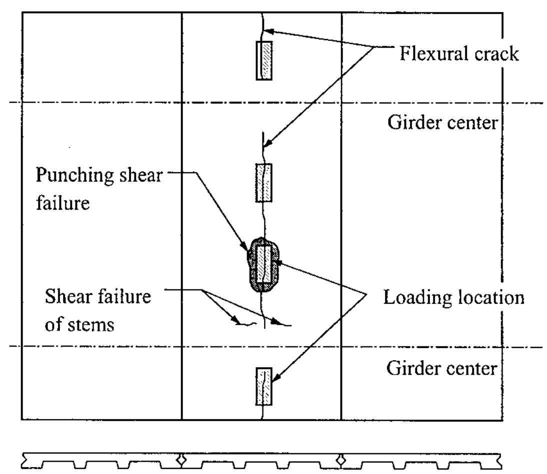
of shear connection schemes under ultimate horizontal shear stress and at fatigue. The push-off specimens consisted of a concrete slab cast on the top of another, previously cast and hardened, concrete slab. The specimens were divided into 10 series. Series 1 and 2 consisted of a double shear interface while Series 3 through 10 were single shear. Appendix I contains details of the 73 concrete girder-to-deck connection specimens, a discussion of the test variables, descriptions of testing procedures, and a summary of the observations made during the tests.

The most promising connection system consisted of a debonded interface with shear keys formed into the girder and shear connectors of reinforcement bars at wide spacing. Figure 41 illustrates the general concept of this system. Figure 42 illustrates details of the formed shear key concrete interface. The purpose of the debonded shear key interface on the girders top flange is to provide horizontal shear resistance through the shear keys while allowing easier future removal of the bridge deck. During deck demolition, a debonded interface may prevent damage to the concrete girders top flanges because the deck would theoretically be lifted off the girder as the connectors are removed. The primary resistance mechanisms of the shear key are bearing and friction between the top flange and bottom of the deck, including the tensile and shear strength of the steel connectors crossing the interface.

Methods of constructing the shear keys by rolling, stamping, and forming were investigated. In the first method, a roller was passed several times perpendicular to the axis of the girder while the concrete was still plastic. The second method consists of pressing the shear key stamp into the top flange while the concrete was still plastic. Forming the shear keys by providing steel or wood forms was the third option considered, and this method was used in the production of



(a) Top surface



(b) Bottom surface

Figure 37. Cracks at ultimate stage.

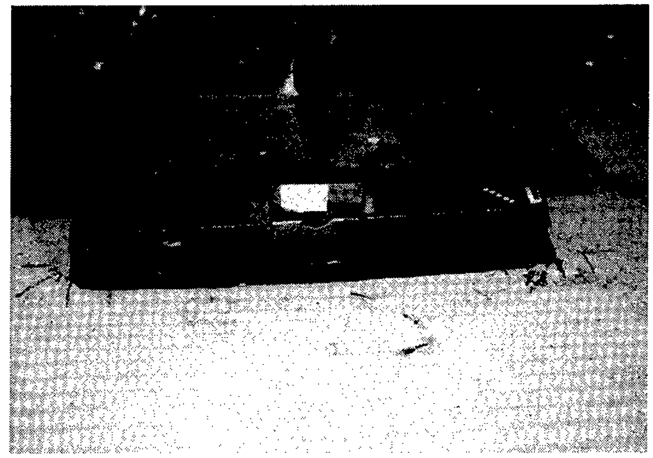


Figure 38. Punching shear failure.

Series 10. For all the methods used for providing shear keys, a debonding agent was applied to the hardened concrete using a brush or hand-held sprayer.

The results of the 73 push-off specimens for all the series are provided in Appendix I. The clamping stress, $\rho_v f_y$, and concrete strengths for each specimen are given. Also included are the maximum horizontal shear stress of the interface and the shear stress at 0.005-in. (0.13-mm) and 0.02-in. (0.51-mm) slip, respectively. The value μ is the apparent coefficient of friction determined at ultimate strength. This ultimate strength could be determined at two values of relative slip: 0.005 in. (0.13 mm) as recommended by Hanson (*26*) and 0.02 in. (0.51 mm) as recommended by Loov and Patnaik (*34*). This coefficient is the horizontal shear stress divided by the clamping stress. Observations of the test results show that the ultimate stress is comparable to the horizontal shear stress at 0.02-in. (0.51-mm) slip. This comparison shows that a slip value of 0.02 in. (0.51 mm), as recommended by Loov and Patnaik, should be a slip limit to the strength of the connection.

Full-Scale Tests

Two full-scale tests were performed to compare the performance of the unbonded shear key system with that of a conventional system. The first girder was a 78-ft (23.4-m) long NU-1100 section, which was designed using the conventional method with a roughened interface and vertical stirrups extending from the girder web into the concrete deck. The second girder was a 71-ft (21.7-m) long NU-1100 section, which was designed by using the unbonded shear key system. A 4-ft (1.22-m) wide concrete deck with 8-in. (20-mm) thickness was placed on the top of the girder.

Both girders were fabricated by Wilson Concrete Company. In the production of the second girder, 10-ft (3.05-m) long shear key forms were attached to the top of the NU-1100 girder forms. The steel connectors then were inserted through the shear key form openings and connected to the top strands of the girder. Concrete was then placed through the opening of the shear key forms. Once the concrete hardened, Stifel V, a debonding agent made by Nox Crete, was sprayed on the girder top flange. The concrete was then placed and consolidated around steel connectors using a small spud vibrator.

A load of 75 and 87 percent of the flexural load capacity of the bonded composite member (first girder) and the debonded composite member (second girder), respectively, was applied at center span of each girder by means of two hydraulic jacks with maximum capacity of 400 kips (1,780 kN). The load was applied in increments of 30 kips (134 kN), at which slippage was measured at midspan and at 4 ft away from the end support. Strain distributions and deflections of the girder at midspan were measured at each load increment. Demec points were placed across the full depth of the girder deck assembly at midspan of the girder to measure the stress distributions. Linear variable displacement transducers (LVDT)

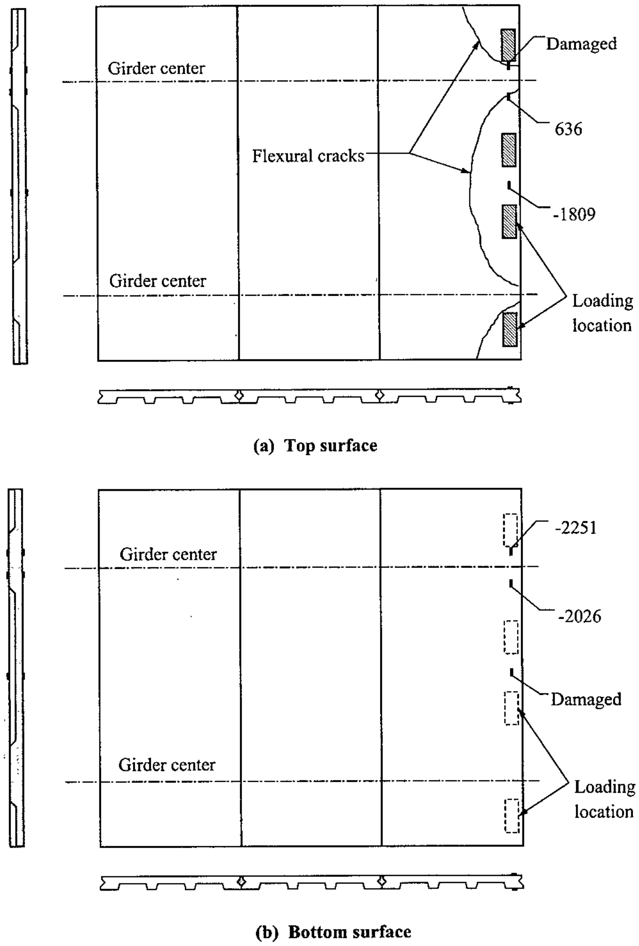


Figure 39. Cracks and stresses under service load (in psi).

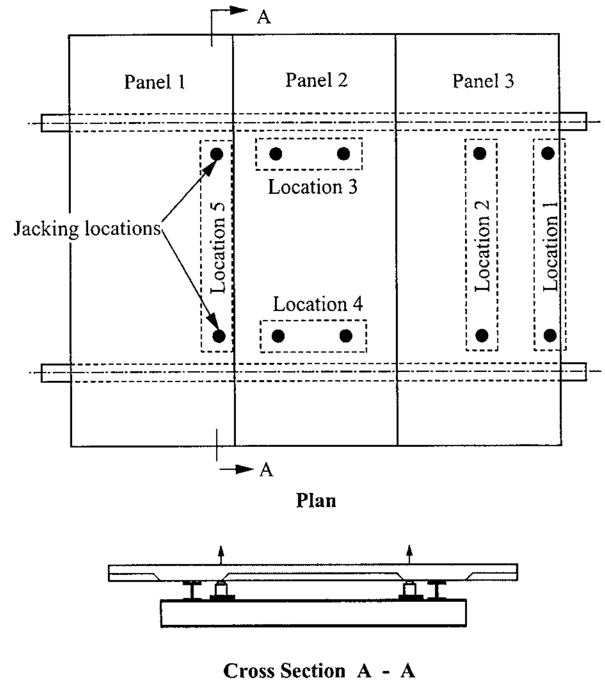


Figure 40. Schematic of deck removal.

were located at end and midspan to measure slippage, and a potentiometer was placed at midspan to measure deflection. The maximum loads applied to the first and second girders were 167 kips (743 kN) and 276 kip (1228 kN), respectively. The concrete strengths at the time of testing were 8.2 ksi (56.5 MPa) and 7.2 ksi (49.64 MPa) for the first girder and its deck, respectively, and 10.1 ksi (69.6 MPa) and 5.1 ksi (35.16 MPa) for the second girder and its deck, respectively. Design calculations for the second girder are given in Appendix M.

Slippage. Slippage measured at different stages of loading is shown in Figure 43 for the debonded composite member. These data show that slippage was less than 0.001 in. (0.025 mm) for loads less than 240 kips (1,068 kN), then increased to 0.006 in. (0.152 mm) when the maximum load of 276 kips (1,228 kN) was applied. According to the shear friction theory, such a slip should occur in order for the steel connectors to resist the horizontal shear forces. No cracks had been noticed at the interface between the girder and the deck during the test, and there was no indication of composite action failure due to the maximum applied load.

Strain and Stress Distributions. Strain and stress distributions obtained from the tests are shown in Figure 44. The strain distribution for the debonded composite member at a moment of 2,040 kip·ft (29,779 kN·m) was similar to that of the bonded composite member at a moment of 1,950 kip·ft (28,465 kN·m), indicating a similar behavior for both systems when they are subjected to approximately the same levels of stresses. The strain diagram of the debonded composite member when subjected to a higher level of stress [moment of 4,080 kip·ft (59,558 kN·m)] shows smooth linear strain

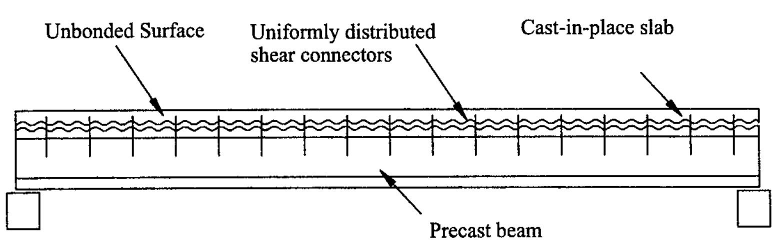


Figure 41. Overview of the new composite system.

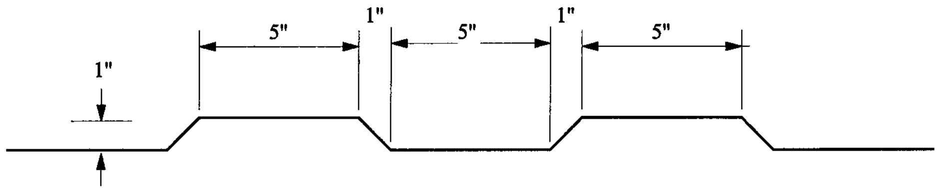


Figure 42. Formed shear key concrete interface detail.

distribution across the full depth of the concrete deck and the girder, indicating an excellent composite action between the concrete girder top flange and the concrete deck.

Load Versus Deflection. Figures 45 and 46 show the load-deflection relationships for the bonded and debonded composite members, respectively, determined from test data and theoretical calculations. Because of the complexity of calculating the theoretical deflection of cracked prestressed members, the theoretical deflection was calculated for loads up to the cracking load. For both systems, test results compared favorably to the theoretical values.

Deck Removal. A 60-lb (27.2-kg) jackhammer was used to remove a 10-ft (3.05-m) section of the concrete deck from both the bonded and debonded composite members. The time required for removal of the concrete deck was recorded, and the damage to both the top flange and the steel connectors was noted. Removing concrete deck from debonded composite member resulted in saving 50 percent in time over removing concrete deck from the bonded composite member at sections away from the steel connectors and saving 20 percent in time removing concrete from around the steel connectors. No damage to the debonded shear keys and the steel connectors in the second girder was noticed as a result of the deck removal processes.

The girder top flange serves a number of important functions, such as lateral girder stability during erection, connection to the deck, and support of the deck forming system. However, it contributes very little to member capacity in the positive moment zone after the deck has become a part of the composite member. Research has demonstrated that concrete girders with up to 50 percent of the flange width damaged at the maximum positive moment location caused no noticeable structural deficiency. It may, therefore, be wise to allow for a certain degree of top flange damage if this would speed up deck removal.

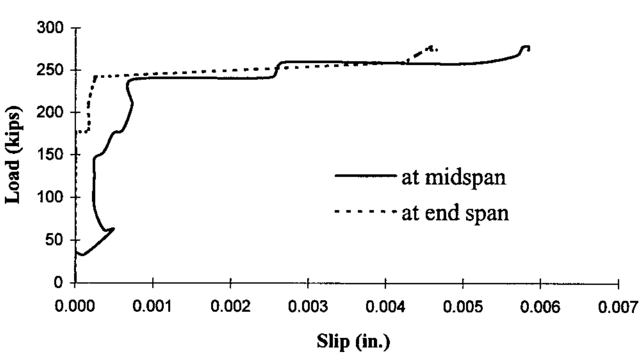


Figure 43. Load versus slip of the debonded composite member.

Design Criteria

The basic linear shear friction equation could be used for the design of horizontal shear:

$$v_n = \rho_v f_y \mu \tag{8}$$

where

ρ_v = the amount of reinforcement crossing the surface per unit area;
f_y = the yield stress of the steel reinforcement; and
μ = the friction coefficient = 1.0.

The proposed design Equation 8 for the debonded shear key system assumes that the shear keys have enough strength to resist the forces and allow sliding of the two concrete layers until failure occurs in the connector but not in the shear key. Failure modes in the shear key are bearing failure at the side of the shear keys and shear failure at the base plane of the shear key.

The shear key is subjected to a bearing force P:

$$P = v_{nh} b_v S_{sk} \tag{9}$$

where

v_{nh} = total horizontal shear;
b_v = girder top flange width; and
S_{sk} = spacing of the shear keys.

Total horizontal shear is given by

$$v_{nh} = V_u / \phi b_v d \tag{10}$$

where

V_u = ultimate shear force;
d = effective shear depth; and
ϕ = reduction factor for shear = 0.9.

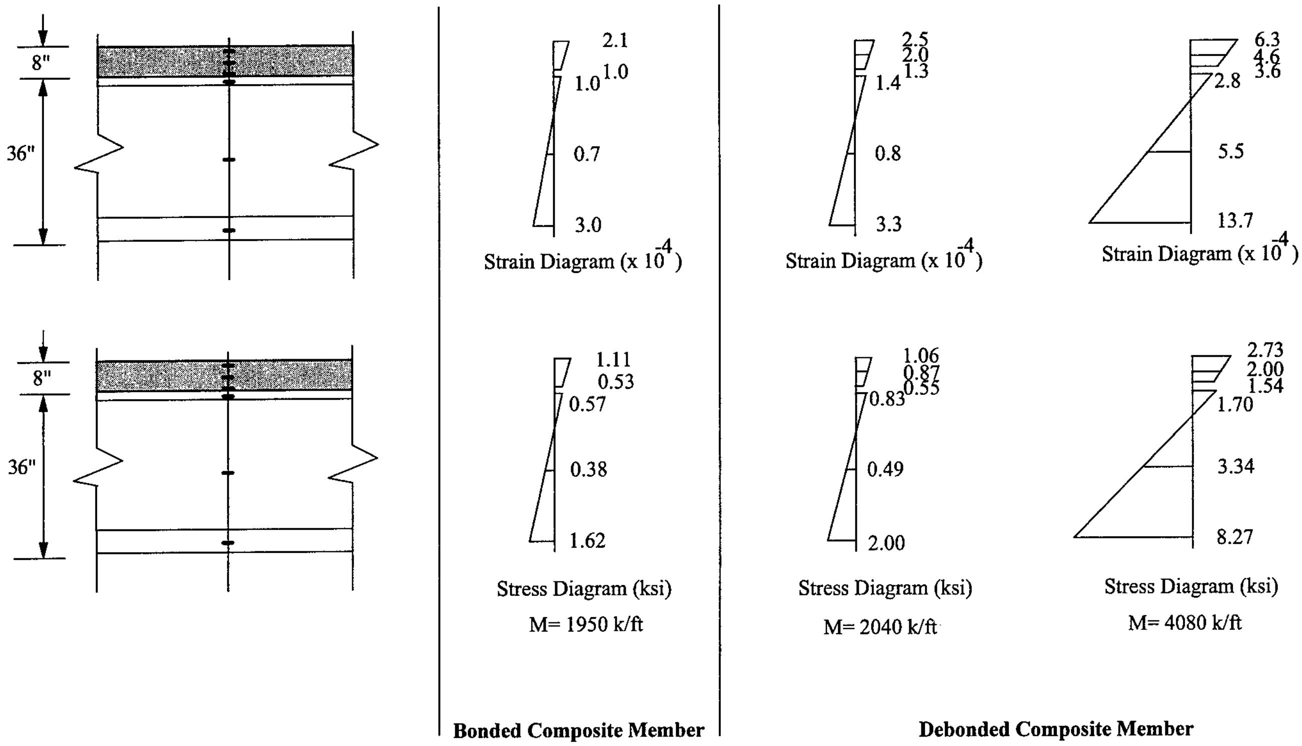


Figure 44. Midspan strain and stress distributions.

$$\text{Maximum allowable bearing stress} = \phi(0.85f_c') = 0.7(0.85)f_c'0.6f_c' \quad (11)$$

where ϕ = reduction factor for bearing = 0.7.

$$\text{Maximum allowable bearing force on the shear key} = 0.6f_c'(t)(b_s) \quad (12)$$

where t and b_s are the height and the width of the shear key, respectively.

The minimum height of the shear key then can be found from Equations 10 and 12 as follows:

$$v_{nh}(b_v)(S_{sk}) = 0.6f_c'(t)(b_s) \quad (13)$$

$$\text{Minimum height (t)} \quad \frac{1.67v_{nh}S_{sk}b_v}{f_c'b_s} \quad (14)$$

The basic shear friction equation with the cohesion term can be applied to determine the minimum area of the shear key needed at the base to resist the shear stresses due to force P (Equation 9):

$$P = v_{nh}(b_vS_{sk}) = (c + \mu f_{cl})A_{sk} \quad (15)$$

$$P = v_{nh}(b_vS_{sk}) = \left(c + \frac{\mu A_{vf}f_y}{b_s2S_{sk}}\right)A_{sk} \quad (16)$$

with μ = 1.4 and c = 0.15 ksi (concrete cast monolithically) as required by ASSHTO LRFD (*7*), then

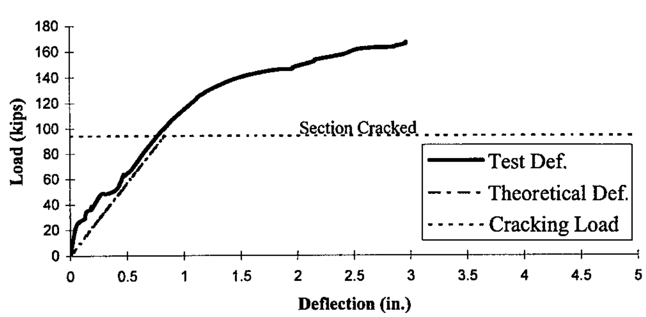


Figure 45. Load versus deflection for bonded composite member.

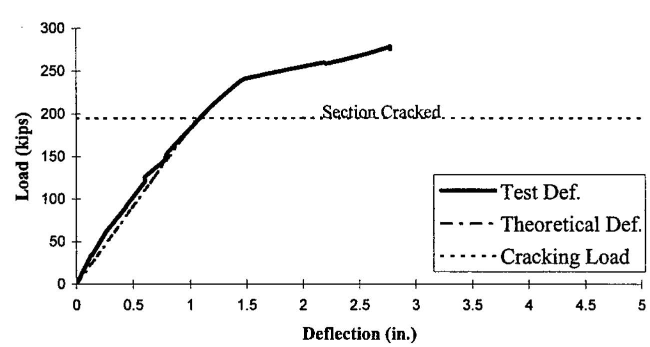


Figure 46. Load versus deflection for debonded composite member.

$$A_{sk} = \frac{v_{nh} b_v S_{sk}}{\left(0.15 + 1.4 \frac{A_{vf} f_y}{b_s 2S_{sk}}\right.} \tag{17}$$

where:

A_{sk} = area of shear key at base (in.^{2});
c = cohesion stress (ksi);
μ = coefficient of friction;
A_{vf} = area of interface shear reinforcement (in.^{2});
b_v = girder top flange width (in.);
S_{sk} = spacing of shear keys (in.);
b_s = width of shear keys (in.);
λ = 1.0 for normal concrete weight;
b_s = width of the shear key;
f_y = yield strength of steel connectors; and
f_c' = concrete strength at 28 days.

Equations 15 and 16 assume that connectors are provided for every other shear key; therefore, the clamping stresses are provided per $2S_{sk}$.

Steel Girder-to-Deck Connections

The main objective of this portion of the research was to develop connection systems for composite action between steel girders and concrete decks, allowing for rapid replacement of deteriorated bridge decks. The most common type of shear connector used for developing composite action between steel girders and concrete decks is a welded shear stud. The most common diameters of steel shear studs used in composite bridge construction are ¾ in. (19 mm) and ⅞ in. (22 mm), with a height of generally 5 in. (127 mm).

A test program was implemented to evaluate the feasibility of using 1¼-in. (32-mm) diameter, 5-in. (127-mm) long studs. Because the load capacity of these studs is about twice that of the ⅞-in. (22-mm) studs, fewer studs will be required along the length of the steel girder. This would decrease the effort required for deck removal from around these studs and decrease the probability of damage to these large studs and to the girder top flange. The test program consisted of 28 push-off tests divided into five groups and a full-scale test. Test results confirmed that 1¼-in. (32-mm) studs could be efficiently welded in the shop or in the field, conveniently inspected for quality of weld, and designed using standard procedures.

Development of Larger Diameter Stud and Welding Equipment

Although the use of 1¼-in. (32-mm) studs is not entirely new, no convenient method of stud gun welding of these studs exists, and methods for testing the weld's quality in a manner consistent with testing ¾-in. (19-mm) and ⅞-in. (22-mm) studs are not available. This research was intended to determine the optimum stud material for large studs; develop welding equipment or modifications of existing equipment; develop a device for testing the studs for quality of weld; and evaluate structural performance.

Studies in cooperation with a stud manufacturer have revealed that standard SAE (Society of Automotive Engineers) 1018 steel could be used for 1¼-in. (32-mm) studs. The estimated yield strength for the cold drawn SAE 1018 steel studs by SAE specifications is 52,000 psi (358 MPa) and the ultimate tensile strength is 64,000 psi (441 MPa). For commercial production, a higher ductility material, such as SAE 1008, may be more beneficial in terms of weldability. Details of the studs produced for the testing program are shown in Figure 47. The steep chamfer of these studs was found to greatly facilitate the welding process. The amount of flux material used was twice that needed for conventional ⅞-in. (22-mm) studs.

The gun used to weld ⅞-in. (22-mm) studs had to be modified to the configuration shown in Figure 48. These modifications included inserting a specially sized chuck for gripping the head of the stud and attaching a plate with three adjustable legs and a one-way bubble level to the stud gun to keep the stud and stud gun plumb prior to and during welding. Welding 1¼-in. (32-mm) studs requires a 3,000-amp power source with an appropriate power supply cord to avoid overheating of the equipment. Such a power source is now available from commercial vendors. With these modifications, stud welding can be appropriately performed.

Inspection of Welded Studs

Testing of studs for quality assurance is required by bridge owners. The Nebraska Department of Roads Specifications (*44*), Section 7.2, requires an inspection of steel studs welded to steel bridge girders. Several of these routine inspections were performed on the welded 1¼-in. (32-mm) diameter shear studs to determine if these large diameter studs could be sufficiently tested in the field.

Field Bending of 1¼-in. (33-mm) Diameter Studs. Section 7.12.04-8 of the Nebraska Department of Roads Specification, states the following:

> The first two studs welded on each beam or girder, after being allowed to cool, shall be bent 45 degrees by striking the stud with a hammer. If failure occurs in the weld of the stud, the procedure shall be corrected and two successive studs successfully welded and tested before any more studs are welded to the beam or girder. The engineer shall be promptly informed of any changes in the welding procedure at any time during construction.

Several 1¼-in. (32-mm) studs were welded to a steel W-section. A sledgehammer was used to strike the studs. The large diameter prevented the stud from being bent, and the weld exhibited no signs of damage. For inspection purposes, a simple method of testing the quality of weld was sought.

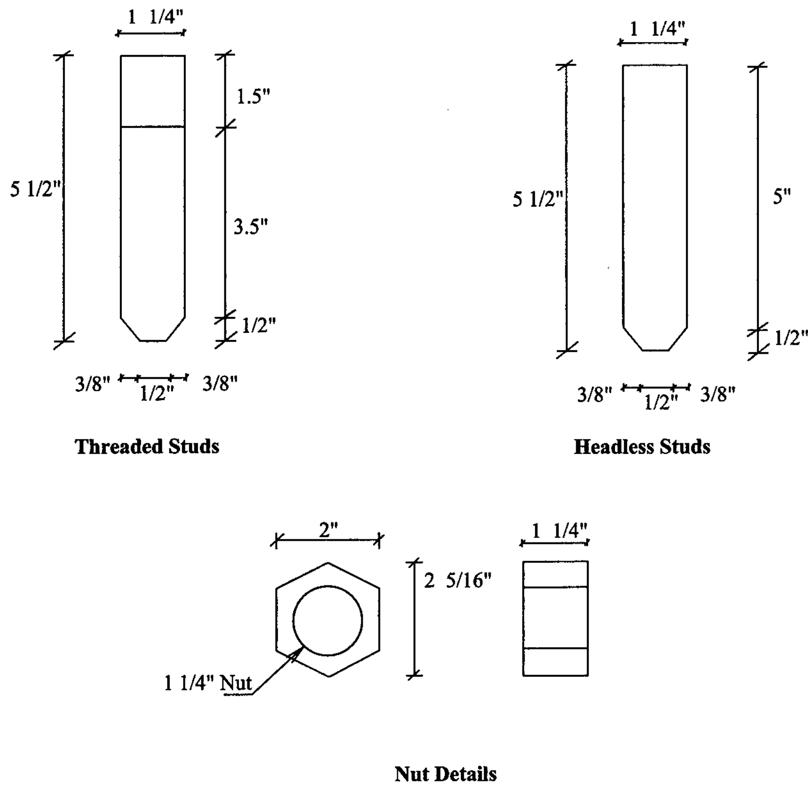


Figure 47. Details of the proposed 1¼-in. stud.

Proposed Field Inspection for 1¼ in. (32-mm) Diameter Studs. Most state DOT specifications require that studs be bent at a 45-deg angle without breakage. Obviously this was not possible for these larger studs. For this reason, the research team developed a portable hydraulic jacking system that could be used in the shop or the field for testing pairs of studs. The device, shown in Figure 49, was successfully used to test studs welded to steel plates and to a full-scale steel girder.

The device consisted of two collars placed around adjoining 1¼-in. (32-mm) diameter studs. The base of the collars was chamfered to accommodate the welds at the base of the studs. A small hydraulic jack was placed between the collars to apply a lateral shear force on the studs. Results of the shear tests indicated that the stud welds exceeded the required capacity of the studs.

Full-Scale Tests

The main objective of the full-scale test was to evaluate the performance of 1¼-in. (32-mm) diameter shear studs. A 40-ft (12.2-m) long W36 × 160 hot rolled section, with alternating headed and headless 1¼-in. (32-mm) shear studs spaced at 6 in. (150 mm), was used in the test. A 4-ft (1.22-m) wide concrete deck with 8-in. (20-mm) thickness was placed on the top of the girder. The slippage between the concrete deck and the steel beam and stress distributions due to live load were recorded before and after applying the repeated loads. The difference in slippage before and after the fatigue ranged from 0.00013 to 0.0005 in. (0.0035 to 0.0125 mm). The stress distribution diagrams showed no change of the neutral axis location before or after the fatigue test. No major cracks, concrete crushing, or stud failure were observed during testing.

Fabrication of test specimens, the experimental program, and test procedures are described in Appendix N.

Slippage. Table 3 shows the results of measured slippage at different stages. The test results show a small slippage due to the monotonic load at 4 ft from the end support before the fatigue load was applied. Some increase of slippage at the span end and midspan section were recorded at 4.8 million cycles of fatigue loading. Loading the girder at midspan monotonically to 100 kips after 4.8 million cycles of loading also resulted in small slippage. Testing showed no loss of composite action between the steel girder and the concrete deck because the slippage was below the failure limits. Only small cracks were noticed at the loading point location.

Stress Distributions. Results of measured strains and stresses at different stages of loading are shown in Figures 50 and 51 for a location 4 ft away from the end support and midspan, respectively. These diagrams show that the neutral axis remained at the same location, indicating a composite action over the entire beam length.

Linear strain distribution along the composite cross section is expected, assuming complete interaction and elastic

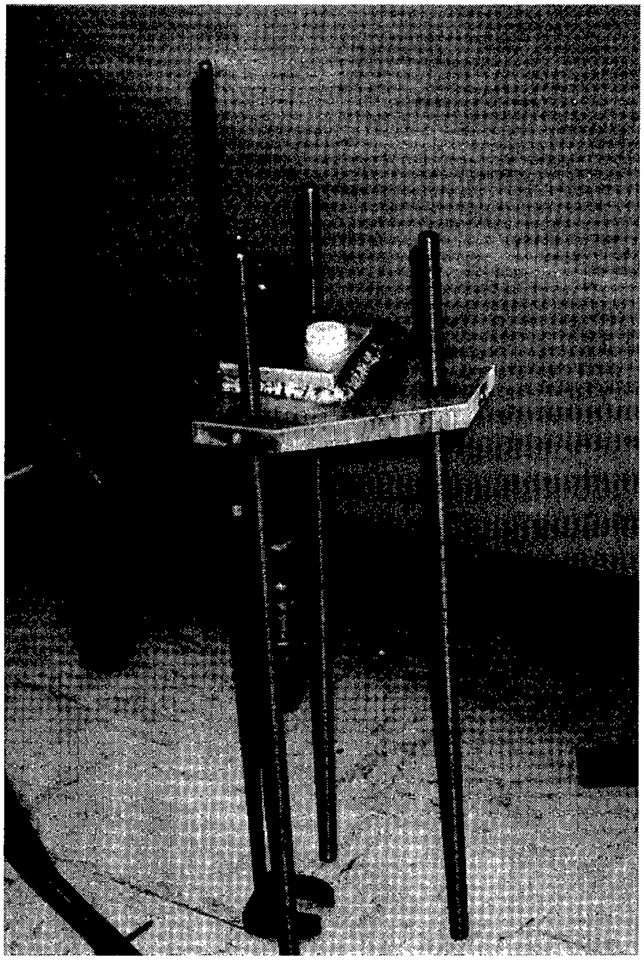


Figure 48. Modified stud gun for 1¼-in. studs.

behavior under working loads. Figures 50 and 51 indicate that there was no complete interaction between the steel beam and the concrete slab. This behavior is expected because a perfect connection requires a connector with infinite shear, bending, and axial stiffness capacities, and there is no mechanical connector capable of providing that. This distribution is similar to that obtained from tests conducted on full-scale beams by Yam and Chapman (*45*). The effect of the incomplete interaction has proven to increase the maximum stresses at the section under the point load by 5 to 10 percent.

Deck Removal. The deck was sawed along one side of the stud line. Other cuts were made in the transverse directions at 8-ft (2.44-m) spacings, as shown in Figure 52. Two methods were then followed to complete the deck removal. In one method, Section 1a of the slab was removed by a crane. Then a 60-lb (27.2-kg) electric jackhammer was used to break concrete from around the studs in Section 1b of the slab before it was removed using a crane. An average of 8 minutes was needed to break the concrete from around each stud. In the second method, another saw cut was made along the other side of the stud line, and then the remaining sections of slab were then removed by the crane. Next, a 10-lb (4.5-kg) sledgehammer was used to break the 2.5-in. (63.5-mm) wide concrete strip between the studs. This method was faster than the first one but was found to be more expensive because of the additional saw cutting cost.

Upon complete removal of the deck, no damage was noticed to the top of the girder due to either saw cutting or jackhammering. Figure 53 shows the girder after deck removal. Photos of the deck removal processes are provided in Appendix N.

Design Criteria

*Fatigue Design.*On the basis of the test data, the following tentative formula was developed for the allowable range of load:

$$Z_r = \alpha d_s^2 \tag{18}$$

where:

- Z_r = allowable range of shear force per stud in lbs;
- d_s = diameter of the stud in in.; and
- α = 16,900 for 100,000 cycles; 15,350 for 500,000 cycles; and 14,150 for 2,000,000 cycles.

It is necessary to provide a sufficient number of connectors to ensure the sufficient strength under repeated applications of working load. The horizontal shear stress caused by applied live loads can reasonably be computed using the elastic theory. Thus,

$$S_r = \frac{v_r Q}{I} \tag{19}$$

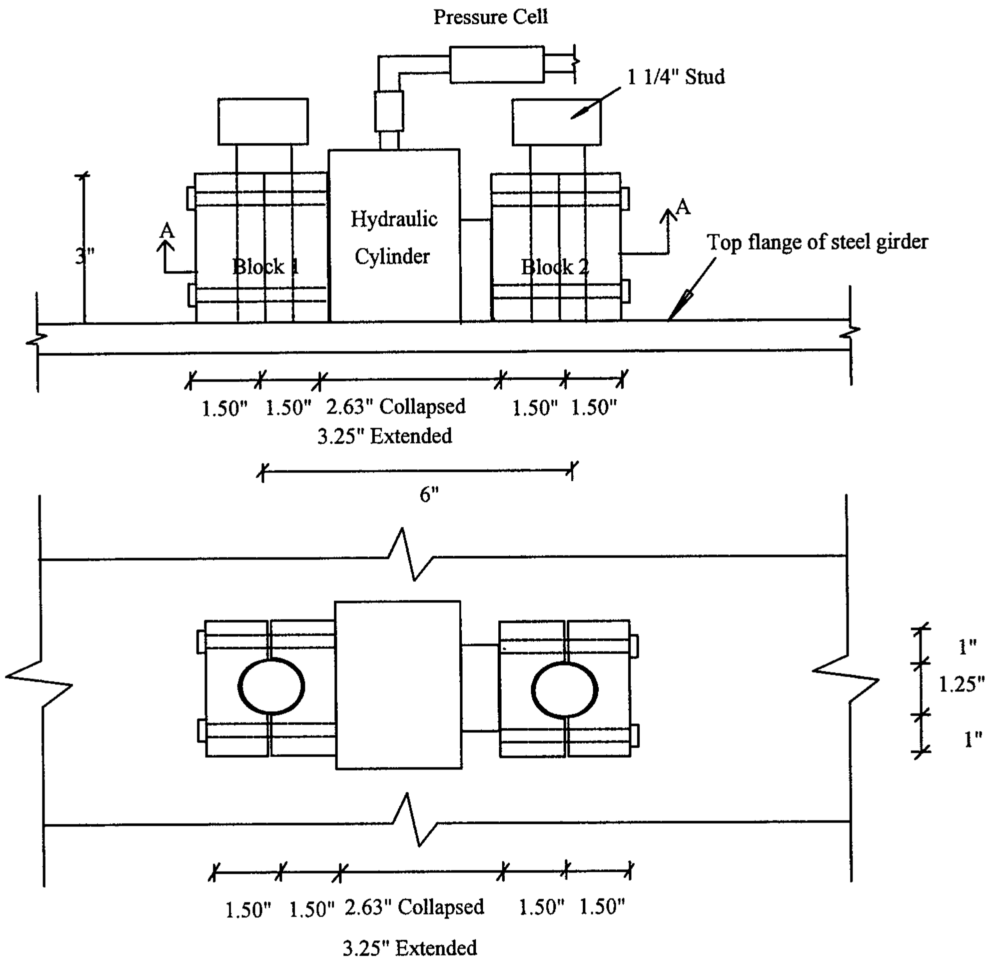
where:

- S_r = range of horizontal shear (kip/in.);
- v_r = range of shear due to live loads and impact (truck or lane loading) in kips;
- Q = statical moment about the neutral axis of the composite section of the transformed compressive concrete area, or the area of reinforcement embedded in the concrete for negative moment (in.^{3}); and
- I = moment of inertia of the composite section (in.^{4}).

The spacing of the connectors can be then found as

$$\text{Spacing} = \frac{Z_r(\text{number of studs per ro}}{S_r} \tag{20}$$

Parametric study of bridges with a simple span, ranging from 60 to 120 ft (18 to 36 m), has shown that the horizontal shear stress due to live loads ranges from maximum at the end of the span to an average of 87 percent at midspan. Table 4



Section A-A

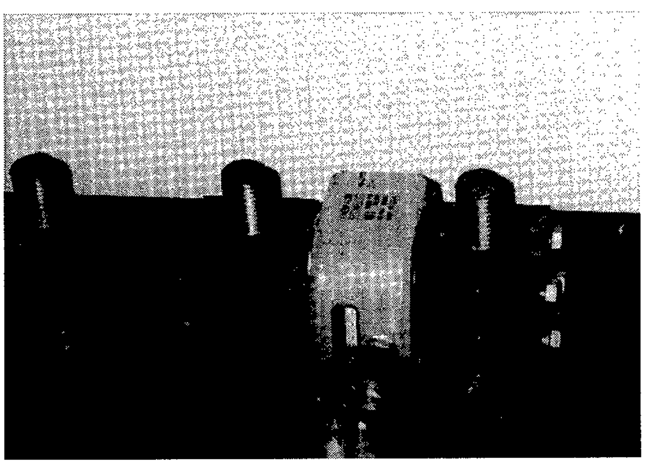


Figure 49. Quality control inspection of 1¼-in. studs.

gives a summary of the design of different bridges that varies from 30 to 120 ft (9 to 36 m) in length and from 6 to 12 ft (2 to 4 m) in girder spacing. The variation of the horizontal shear stress along the span decreases as the span length increases. A more conservative and simpler approach for the connector spacing would be to consider the shear at end span and uniform spacing of the connectors along the span based on that shear value. This approach is not applicable for multispan bridges because the range of horizontal shear stress varies in greater manner as compared to simple span bridges.

Strength Design. The number of connectors provided for fatigue shall be checked to ensure that adequate connectors are provided for ultimate strength. The ultimate strength of

TABLE 3 Slippage measurements

Load position		Slippage at 4 ft from support (in.)	Slippage at mid span section (in.)
Load at 4 ft from end support	before fatigue	0.0015	0.0006
	after 1.76 million cycles	0.0015	0.0006
	after 4.80 million cycles	0.0019	0.0007
Load at mid span	after 4.80 million cycles	0.0015	0.0008

the shear studs, determined according to equation 10-66 of AASHTO Standard Specifications (*6*), is given by

$$S_u = 0.4d^2\sqrt{f_c'E_c} \le A_{sc}f_u \qquad (21)$$

where:

- S_u = ultimate strength of the shear connector (kips);
- d = stud diameter (in.^{2});
- f_c' = compressive strength of the concrete at 28 days;
- E_c = modulus of elasticity of the concrete;
- A_{sc} = cross-sectional area of the stud shear connector (in.^{2}); and
- f_u = specified minimum tensile strength of a stud shear connector.

Equation 22 limits the capacity of the stud to $(A_{sc})(f_u)$. AASHTO LRFD design equation 6.10.7.4.4c-1 can be used to determine the capacity of the 1¼-in. (32-mm) studs without any modifications.

As indicated in Equation 21, the shear connector strength depends on the concrete compressive strength and the tensile strength of the shear stud. Ollgaard et al. (*46*) indicated that tests with calculated strengths governed by concrete strength have exhibited a combination of concrete and steel failures. This inconsistency between predication capacity and the experimental failure modes could be related to the shear stud ductile behavior as indicated by Viest et al. (*47*). Viest also showed that the ductility is the result of high local stresses: the concrete is undergoing inelastic permanent deformations or crushing locally around the lower part of the stud, creating

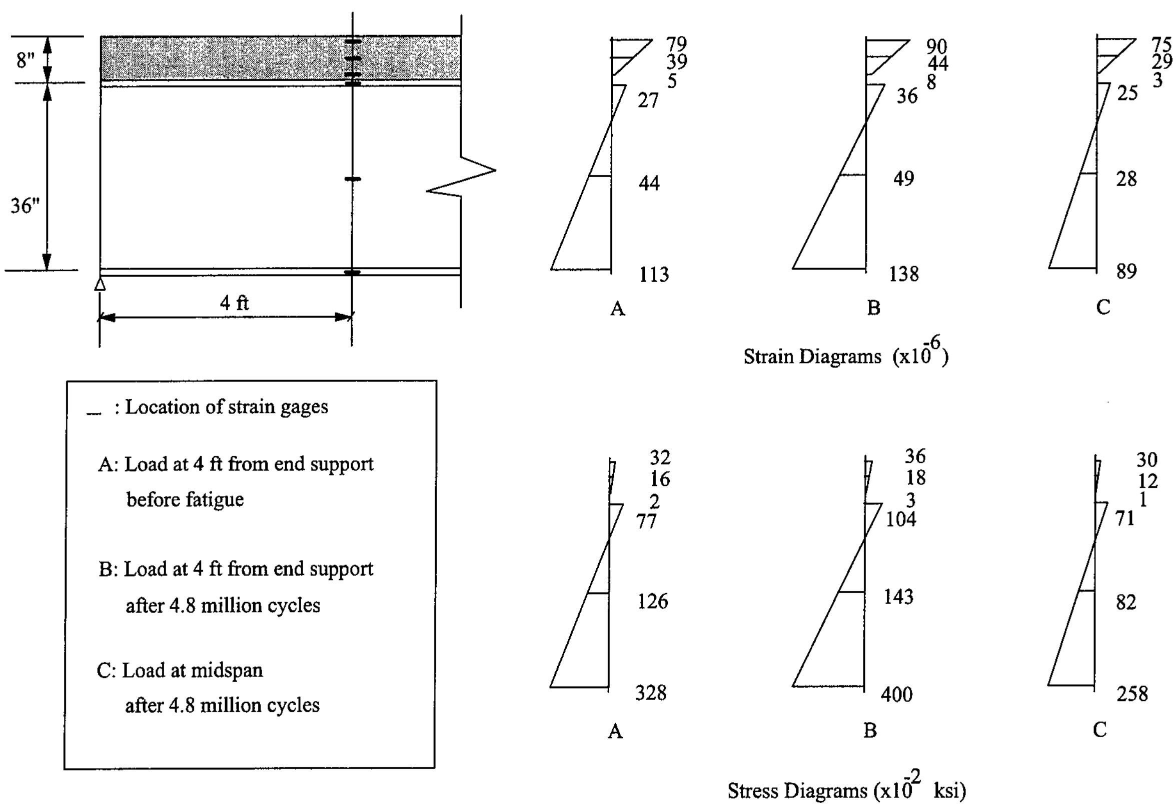


Figure 50. Stress distribution at 4 ft from end support.

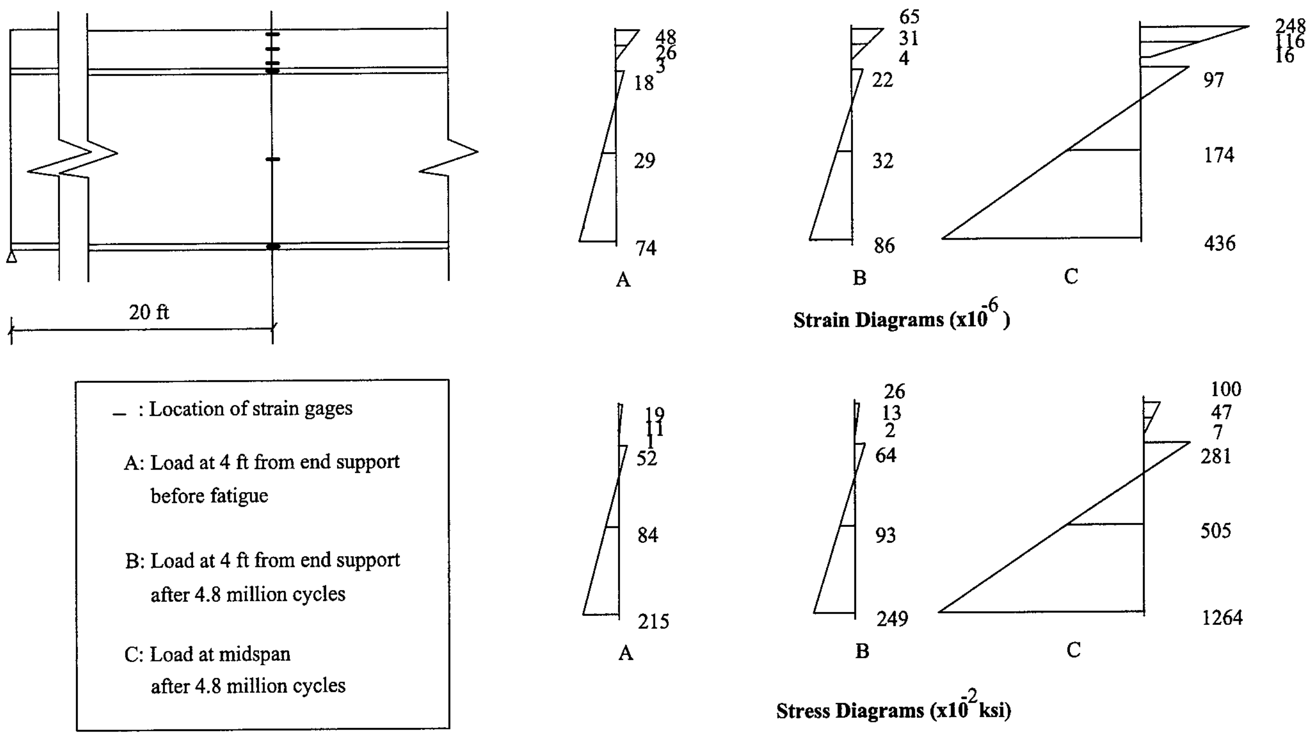


Figure 51. Stress distribution at midspan.

a void that permits the stud to deform (Figure 54). Thus, even if the predicted strength appears to be based on a concrete failure, the overall shear connector behavior is still ductile because of the deformations occurring in the stud.

The STLBRIDGE (design of continuous steel bridge girders) and AISI (design of short span bridges) programs were used to design the bridge girders for the parametric study. The designed sections were in the elastic stages under applied factored loads. Therefore, the horizontal shear stresses at the steel girder top flange interface can be determined according to the elastic theory.

Slutter and Driscoll (*41*) proved that the elastic theory can still be used to determine the number of studs needed for resisting the horizontal shear stresses at the interface of the steel girder top flange at ultimate loads.

It was found in the parametric study that the number of connectors required by the elastic theory is less than that required by the specifications. The specifications require a sufficient number of shear connectors to resist the compressive force in the concrete slab when the fully plastic stress distribution for the ultimate flexural strength is reached. The parametric study, as discussed before, had shown that the stresses in the slab for noncompact sections were in the elastic stages. Therefore, designing the shear connectors by the elastic theory is more appropriate, as long as the stresses in the concrete and the steel remain elastic, which is the case for the noncompact sections.

Goble (*48*) indicated that when the stud diameter exceeded 2.5 times the plate (or the girder top flange) thickness, the stud shear connector fails prematurely by tearing out the plate. It was, therefore, recommended that this ratio should not be exceeded unless the stud was placed over the web of the supporting steel beam.

Stud Spacing. The parametric study of simple span bridges for horizontal shear indicated that one line of 1¼-in. (32-mm) shear studs was adequate for all design cases. Figure 55 shows the range of the stud spacings. The lower bound of spacing is higher than the minimum spacing allowed by the specifications, which is four times the stud's diameter, 5 in. (127 mm). The upper bound is 24 in. (600 mm) because connectors perform the necessary function of holding the concrete slab in contact with the steel beam. However, Viest et al. (*47*) reported that the maximum stud spacing should not exceed the lesser of eight times the slab thickness or 36 in. (91 cm) and the minimum spacing of six stud diameters center to center along the length of the beam for installations without a steel deck.

PROPOSED SPECIAL PROVISIONS FOR REMOVAL OF EXISTING BRIDGE DECK

Because common specifications for deck removal are not readily available, the research team has developed sample special provisions, given in Appendix P, for deck removal.

The philosophy throughout these provisions is to give the contractor as much freedom as possible to use the latest equipment and the contractor's unique abilities to remove the deck as fast as possible while preserving the structural and environmental qualities of the project.

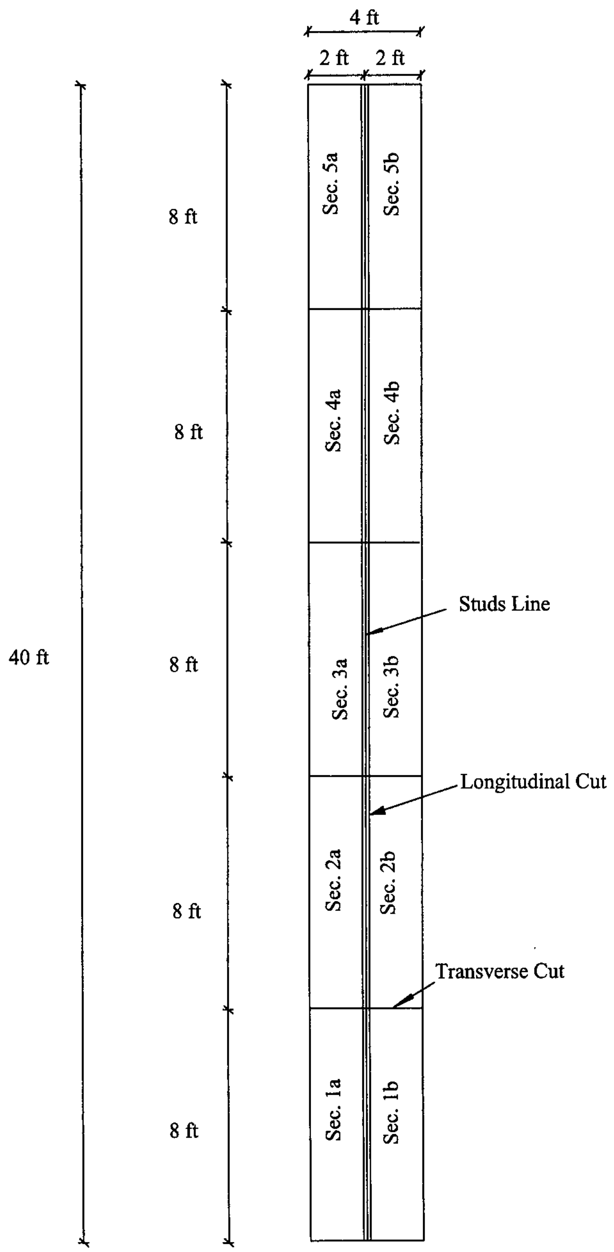


Figure 52. Saw cutting lines of the slab.

CONSTRUCTION TIMELINES

Timelines were developed for the different bridge systems tested in this project. For comparison, a typical situation for bridge construction provided a baseline timeline. The impact of varying the length of the work period per day (16 hour/day and 24 hour/day) and the effects of complete closure, partial closure, and nighttime/weekend closure on construction time were considered. The timelines, provided in Appendix Q, compare the speed of construction for various deck replacement systems, to show potential time-saving benefits. However, cost and other factors would also need to be considered when evaluating the benefits of accelerated construction methods.

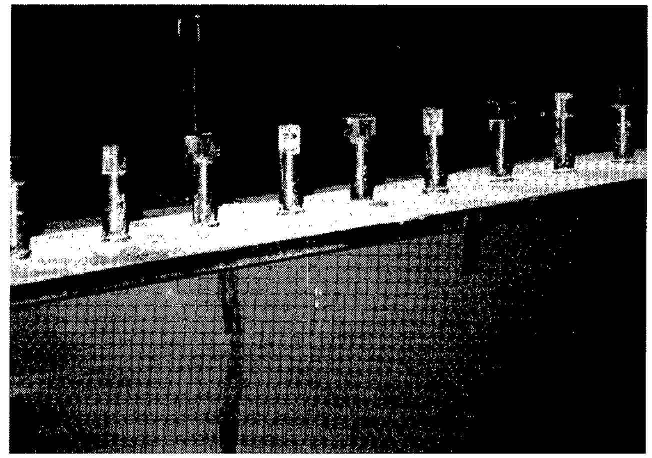


Figure 53. Studs after deck removal.

To compare the speed of deck replacement options, all bridges were assumed to be approximately 44 ft (13.41 m) wide and 250 ft (76.2 m) long. Also, a normal strength gain of CIP concrete was assumed, although high early strength concrete would likely provide added time savings. The use of precast barrier systems and other means to accelerate construction were not considered.

The following assumptions were made in developing the timelines:

- The work shift is always 8 hours, but the number of work hours per day and work days per week vary:
  –8 hours/day, 5 days per week (40-hour work week),
  –16 hours/day, 6 days per week (96-hour work week), and
  –24 hours/day, 7 days per week (168-hour work week).
- Because of inefficiencies and night work, increasing the number of work hours per week from 40 hours to 96 or 168 hours per week would increase production by 100 or 250 percent, respectively.
- When traffic is maintained on a bridge and work proceeds on approximately one-half of the bridge at a time, production efficiency is assumed to be reduced to only 80 percent of normal because of restrictions imposed by traffic.
- For CIP systems, an adequate strength for carrying traffic is attained in the deck at 14 days, and adequate strength for barriers and expansion joint pours at 5 days. Also, light construction activity (tying rail reinforcing and setting expansion joints) will be allowed after at least 3 days, and form stripping can be started 7 days after deck casting.
- For full-depth precast systems and systems using SIP precast forms, rapid setting grouts will achieve required compressive strength in 3 to 4 hours.
- Bridge sites provide for favorable access for construction equipment, and delays for weather or equipment failure are not considered.
- One timeline was developed that required only nighttime and weekend closure of the bridge. It was felt that

TABLE 4 Parametric study of stud shear connector spacing

Girder Size	Rolled Section: W24X55						Rolled Section: W24X94					
Span Length (ft)	30						30					
Girder Spacing (ft)	6						12					
Location	1.0	1.1	1.2	1.3	1.4	1.5	1.0	1.1	1.2	1.3	1.4	1.5
Spacing (fatigue requirement)	13.3	14.2	15.4	16.7	18.2	19.4	6.6	7.1	7.8	8.3	9.0	9.7
Spacing (elastic theory)	14.5	16.6	19.9	24.9	30.6	44.1	7.3	8.4	10.0	12.2	15.5	22.0
Spacing (specifications)	12.0	12.0	12.0	12.0	12.0	12.0	8.0	8.0	8.0	8.0	8.0	8.0
Girder Size	Rolled Section: W30X124						Rolled Section: W40X211					
Span Length (ft)	60						60					
Girder Spacing (ft)	6						12					
Location	1.0	1.1	1.2	1.3	1.4	1.5	1.0	1.1	1.2	1.3	1.4	1.5
Spacing (fatigue requirement)	14.2	15.4	16.4	17.1	17.1	17.9	9.3	9.8	10.7	11.0	11.3	11.3
Spacing (elastic theory)	15.0	17.5	20.2	23.9	29.8	40.7	9.9	11.3	13.2	15.6	19.6	25.6
Spacing (specifications)	11.0	11.0	11.0	11.0	11.0	11.0	9.0	9.0	9.0	9.0	9.0	9.0
Girder Size	Rolled Section: W40X211						Rolled Section: W36X393					
Span Length (ft)	90						90					
Girder Spacing (ft)	6						12					
Location	1.0	1.1	1.2	1.3	1.4	1.5	1.0	1.1	1.2	1.3	1.4	1.5
Spacing (fatigue requirement)	19.5	20.6	21.4	21.9	21.9	21.9	10.3	11.0	11.5	11.6	11.6	11.6
Spacing (elastic theory)	19.9	22.5	26.1	31.0	38.1	49.6	10.8	12.2	14.1	16.7	20.4	26.3
Spacing (specifications)	16.0	16.0	16.0	16.0	16.0	16.0	13.0	13.0	13.0	13.0	13.0	13.0
Girder Size	Rolled Section: W30X124						Plate Section: web 68"x0.75", top flange 16"x1", bottom flange 18"x2"					
Span Length (ft)	120						120					
Girder Spacing (ft)	6						12					
Location	1.0	1.1	1.2	1.3	1.4	1.5	1.0	1.1	1.2	1.3	1.4	1.5
Spacing (fatigue requirement)	22.6	23.9	24.8	24.8	24.8	24.8	17.3	18.5	18.9	18.9	18.9	18.9
Spacing (elastic theory)	22.4	25.3	29.4	35.2	42.9	56.3	17.7	19.2	20.0	20.4	20.9	21.4
Spacing (specifications)	16.0	16.0	16.0	16.0	16.0	16.0	17.5	17.5	17.5	17.5	17.5	17.5
Slab Stress	0.0	0.7	1.3	1.6	1.9	1.9	0.0	0.7	1.2	1.5	1.7	1.7

Deck thickness = 8 inches 28-day concrete strength of the deck = 5.0 ksi Yield strength of the steel girder = 50 ksi
Design based on composite loads: Superimposed Dead Load = 25 psf, HS20 AASHTO Truck Load with impact

the only viable system that would accommodate these time constraints would be a full-depth precast system. (CIP systems, even employing very high early strength concrete, would normally not be suitable for total deck replacement under these constraints.) For this system, a temporary barrier system could be used to constrict traffic during construction and allow construction of CIP barriers essentially off the critical path.

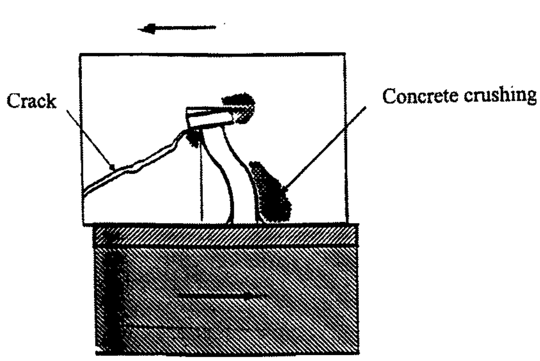


Figure 54. Distress locations resulting from stud deformation.

The timelines produced for 22 cases are summarized in Table 5. In 12 cases, the road is closed for traffic and the bridge is demolished and replaced at one time. These included three cases for each of the conventional CIP decks, standard precast SIP panels with CIP concrete topping, SIP panels with integral cantilevers and CIP, and full-depth precast deck systems. In another nine cases, the traffic is maintained on one-half of the bridge while the other half is constructed. These included three

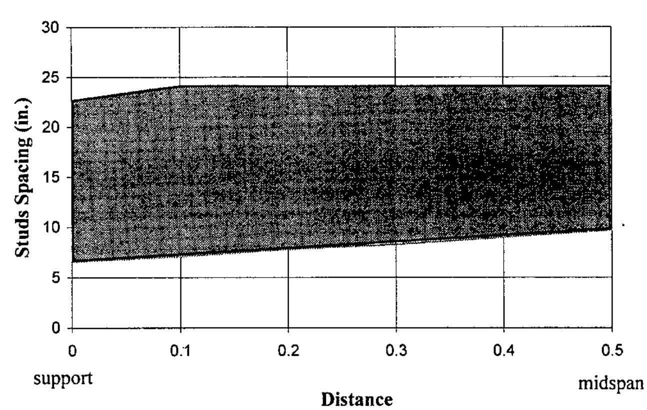
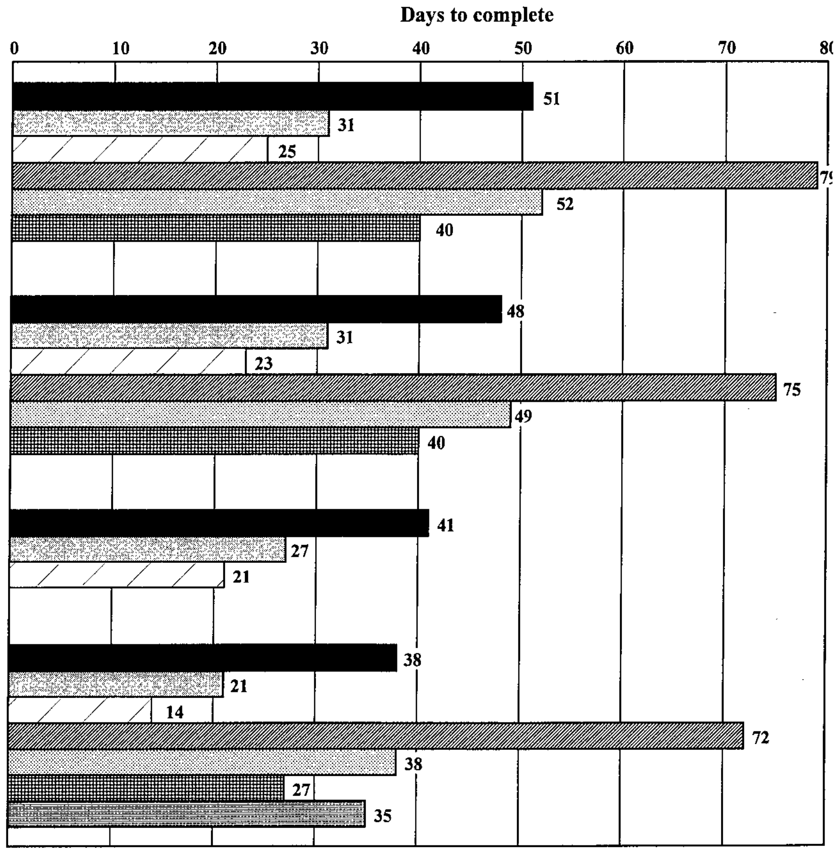


Figure 55. Lower and upper bounds of stud spacing.

TABLE 5 Timelines for different systems

Deck Type	Hours per work day	Days per work week	# of stages
Full-depth cast-in-place deck system	8	5	1
	16	6	1
	24	7	1
	8	5	2
	16	6	2
	24	7	2
Conventional stay-in-place deck system	8	5	1
	16	6	1
	24	7	1
	8	5	2
	16	6	2
	24	7	2
Continuous stay-in-place deck system	8	5	1
	16	6	1
	24	7	1
Full-depth precast deck system	8	5	1
	16	6	1
	24	7	1
	8	5	2
	16	6	2
	24	7	2
	Night time and weekend closure		



cases each of conventional CIP, standard precast SIP panels with CIP concrete topping, and full-depth precast deck systems. In one case, for a full-depth precast system, the bridge is closed in the nighttime and weekends only.

COST ANALYSIS OF BRIDGE DECK SYSTEMS INVESTIGATED

Schedule, construction cost, field productivity, single versus multiple shift construction, and the out-of-service duration or road use cost were evaluated for the following bridge-deck replacement systems:

- Full-depth CIP bridge deck [9-in. (229-mm) thick with conventional epoxy-coated reinforcement];
- Precast prestressed panel with a 6-in. (152-mm) CIP concrete deck, using epoxy-coated welded wire fabric reinforcement;
- Precast prestressed continuous panel with a 4.5-in. (114-mm) CIP concrete deck, using epoxy-coated welded wire fabric reinforcement; and
- Full-depth precast prestressed deck system with longitudinal post-tensioning.

The evaluation for each system is based on the removal and replacement of an existing 250- × 44-ft (76.20- × 13.41-m) bridge deck and traffic barrier or railing. The total area of this bridge deck is 11,000 sq ft (1,022 m^2). Wages and material costs were based on those for Kansas City, Missouri, in June 1996. Maintenance of traffic; structural steel repairs; bearing repair or replacement; approach slab removal or replacement; and drainage, grading, or roadway approach work were not included in the evaluations.

All systems were evaluated for three conditions of work schedule. These are 8 hours/day, single shift, 5 days/week (40-hour work week); 16 hours/day, double shift, 6 days/week (96-hour work week); and 24 hours/day, triple shift, 7 days/week (168-hour work week).

The out-of-service duration is limited to the number of calendar days the bridge will be out of service for the actual deck removal and replacement. The research team has incorporated the road user cost (currently in use by many agencies) to help evaluate the total cost of the project by assigning a dollar value to the time component of the construction project.

Base Bid (A) + Road User Cost (B) = Award Basis (A + B)

TABLE 6 Cost estimates

AWARD BASIS (A + B) = BASE BID (A) + USER COST COMPARISON (B)									
				ROAD USER COST = $0/CAL DAY		ROAD USER COST = $20,000/CAL DAY		ROAD USER COST = $40,000/CAL DAY	
		A		B	A + B	B	A + B	B	A + B
Deck type	WORK HOURS / DAYS	Base Bid	Out of Serv Dur	Road User Cost Days x 0	Award Basis	Road User Cost Days x 20,000	Award Basis	Road User Cost Days x 40,000	Award Basis
Full depth w/ conventional reinforcement	5 DAYS/WK x 8 HR /DAY	$337,281	52	$0	$337,281	$1,040,000	$1,377,281	$2,080,000	$2,417,281
Conventional precast SIP panel system		$315,351	47	$0	$315,351	$940,000	$1,255,351	$1,880,000	$2,195,351
Continuous precast SIP panel system		$345,583	40	$0	$345,583	$800,000	$1,145,583	$1,600,000	$1,945,583
Full depth precast deck system		$359,322	37	$0	$359,322	$740,000	$1,099,322	$1,480,000	$1,839,322
Full depth w/ conventional reinforcement	6 DAYS/WK x 16 HR /DAY	$409,196	30	$0	$409,196	$600,000	$1,009,196	$1,200,000	$1,609,196
Conventional precast SIP panel system		$377,600	30	$0	$377,600	$600,000	$977,600	$1,200,000	$1,577,600
Continuous precast SIP panel system		$403,982	26	$0	$403,982	$520,000	$923,982	$1,040,000	$1,443,982
Full depth precast deck system		$409,876	20	$0	$409,876	$400,000	$809,876	$800,000	$1,209,876
Full depth w/ conventional reinforcement	7 DAYS/WK x 24 HR /DAY	$521,718	24	$0	$521,718	$480,000	$1,001,718	$960,000	$1,481,718
Conventional precast SIP panel system		$473,505	22	$0	$473,505	$440,000	$913,505	$880,000	$1,353,505
Continuous precast SIP panel system		$497,398	20	$0	$497,398	$400,000	$897,398	$800,000	$1,297,398
Full depth precast deck system		$497,032	13	$0	$497,032	$260,000	$757,032	$520,000	$1,017,032

The base bid (A) is the conventional unit price or lump sum bid. The road user cost (B) is typically the construction duration in calendar days multiplied by the road user cost (determined by the owner). The award basis of the contract is the lowest A + B from a responsive, responsible bidder. This type of contract will stipulate the construction duration in calendar days. The owner will provide the maximum number of days the contractor is allowed to complete the project; and the contractor's bid days used in the road user cost (B) become the contract completion duration, and incentive/disincentives will be based on the contractor's bid days. An incentive/disincentive clause usually defines the road user cost multiplier and may provide a maximum cap on the incentive. The incentive is an integral component for this type of contract. Without the incentive, the "risks" of the contract may not outweigh the "rewards" for the contractor.

Table 6 shows the cost estimates for different deck systems and work schedules. These estimates indicate that, for the assumptions made, the full-depth precast prestressed deck system with longitudinal post-tensioning would yield the largest cost savings.

CHAPTER 4

CONCLUSIONS AND SUGGESTED RESEARCH

CONCLUSIONS

With increasing highway traffic, motorists are becoming more intolerant of delays during rehabilitation of bridge decks. Their impatience also increases the risk of accidents. Methods of bridge-deck replacement that allow repair work to be completed at night or during other periods of low traffic and methods that reduce the total time for reconstruction help to improve public acceptance, reduce accident risk, and yield economic and environmental benefits. In recent years new methods have been developed to incorporate rapid replacement of bridge decks. The research reported here attempts to evaluate existing rapid bridge-deck replacement methods and develop better procedures and superstructure designs for future rapid deck replacement by improving both conventional and new materials, designs, and construction techniques. The three main areas where modifications could be made to make deck systems more suitable for rapid replacement are the demolition process and equipment, the bridge deck systems, and the bridge girder-to-deck connections.

Deck demolition often controls the rate of progress because concrete removal is tedious and labor intensive. Removing concrete from around steel connectors, without damaging the connectors or the girders, is particularly expensive. The cost of removal has a major influence on the selection of the method of rehabilitation, and the equipment used for removal may be different for different projects. Each method of removal has its advantages and disadvantages. For example, hand-held jackhammers minimize the potential for connector and girder damage, but they are extremely slow and noisy. Concrete crushers are perhaps the fastest equipment, but the resulting waste needs to be handled in an environmentally acceptable manner. In recent years, powerful circular saws that are capable of cutting an 8-in. (200-mm) reinforced concrete deck at rates exceeding 10 ft/hr (3.0 m/hr) have become available. However, one possible disadvantage is the potential of girder flange damage if the cut is directly over it, although this problem was not experienced during the course of this research or during recent construction projects. The time allotted for the job to be completed, as well as environmental concerns, controls the decision regarding the equipment and method used for deck removal. Because every job has unique location, traffic, and time constraints, the method and equipment used for deck removal should be selected on a job-by-job basis. A set of special provisions is introduced in Chapter 3 to aid owners in the task of deck removal. These provisions suggest that owners should allow contractors the freedom to choose methods and equipment for deck removal, without compromising structural and environmental concerns as equipment becomes more advanced, and contractors have unique abilities.

The girder top flange serves a number of other functions, such as lateral girder stability during erection, connection to the deck, and support of the deck forming system. However, it contributes very little to member capacity in the positive moment zone after the deck has become a part of the composite member. Therefore, significant localized loss of the top flange during deck removal would have practically no impact on the structural capacity of the girder-to-deck system as long as full composite connection capacity is preserved. Thus, accidental transverse cuts in the top flange of steel girders in the positive moment area should be tolerated.

Because the top flanges of steel girders in negative moment areas are the primary tension elements, no damage due to deck removal should be permitted in these areas. Also, because the need for composite action in these areas is minimal, a noncomposite design in the negative moment areas over the piers is acceptable. With the absence of connectors in these areas, deck removal is significantly facilitated and the risk of damaging the top flanges is minimized.

Experimental studies have demonstrated that concrete girders with up to 50 percent of the top flange width damaged at the maximum positive moment location caused no noticeable structural deficiency. Theoretically, full loss of the top flange has no measurable impact on the structural capacity of the girder-to-deck composite system. A similar conclusion was found for the positive moment areas of steel girder bridges. Thus accidental damage to the top girder flange should be permitted if it would expedite deck removal. The extent of damage should be limited to that corresponding to the maximum horizontal shear stress on the concrete interface, limited according to AASHTO LRFD specifications by the lesser of 0.20 f_c' and 800 psi (5.5 MPa).

The second area of study to improve the speed of deck replacement deals with the deck system itself. About 70 percent of U.S. bridges have CIP-reinforced concrete decks over steel or concrete girders. Typically, deck design is performed in accordance with the provisions of the AASHTO standard specifications or the AASHTO LRFD specifications. The empirical method in the AASHTO LRFD specifications yields a significantly reduced amount of reinforcement in the deck. This method has been validated by research and field experience and should be used whenever applicable. Because this method results in the least amount of reinforcement, deck removal is made simpler.

The main advantage of a CIP concrete deck is its ability to easily accept field adjustments to achieve the required bridge profile. However, this type of deck requires a long construction time, forming and placing of the reinforcement, extensive field quality control, and high likelihood of shrinkage cracking. The use of welded wire fabric (WWF) as a replacement for conventional reinforcing bars can considerably reduce the construction time. Two bridge deck systems were tested, one system with conventional reinforcement and one with WWF. The laboratory time log shows that the deck with WWF took 30 percent less time than the conventionally reinforced deck to place the reinforcement. Therefore, to reduce construction time, epoxy-coated WWF should be considered wherever applicable to replace conventional reinforcing bars. However, the difference in material cost and strength should also be considered.

SIP bridge deck systems include metal deck panels and precast pretensioned concrete panels. They act as a permanent form in CIP construction and save considerable forming time. The precast concrete SIP panels become part of the deck thickness and eliminate the need for positive moment reinforcement, but result in reflective deck cracking. With conventional SIP panels, the overhangs must be formed with conventional formwork.

The research team has developed an innovative continuous precast prestressed SIP panel system. This system consists of full-width 4.5-in. (114-mm) thick precast prestressed panels with leveling devices, a grout stop, and a 4.5-in. (114-mm) CIP concrete topping. The portion over the girder line is kept open to accommodate shear studs. In this system, the overhang form is a part of the SIP system. The SIP system is continuous both in the transverse and the longitudinal directions to help eliminate reflective cracks. The construction time for this system is 20 percent less than a conventional SIP system and 60 percent less than a conventional CIP system. This system uses readily available materials, and its cost is comparable to that of the CIP deck system. Therefore, it has the potential of being a common construction method, not just limited to cases where construction speed is of primary importance.

Full-depth precast deck systems with reinforced or prestressed concrete panels can also eliminate a considerable amount of deck construction time. However, because full-depth precast deck systems do not give as good a riding surface as CIP systems, an overlay or surface grinding of up to 0.5 in. (12 mm) is often required to provide a smooth riding surface. Blockouts are needed for shear connectors to make the system composite.

A full-depth precast system was developed as part of this research. Each panel can span over several girder lines up to the full width of a bridge. The panels are transversely pretensioned and longitudinally post-tensioned. The system is about 10 percent thinner and 20 percent lighter than conventional CIP or solid precast reinforced concrete decks. To provide a smooth riding surface, it is recommended that an additional 0.5-in. (12-mm) thickness be provided over the panels to allow for grinding and achieving a smooth surface. The full-depth precast panel system was found to provide the fastest construction of all systems studied, and the two-way prestressing results in controlled cracking. Cost estimates show that the proposed system compares very favorably with other full-depth systems because of its non-proprietary nature, but it is more expensive than the CIP and SIP deck systems.

The third area of study to improve the speed of deck replacement examined the connection system between concrete decks and concrete or steel girders. Demolition of bridge decks that are compositely connected to either structural steel or precast concrete I-girders is one of the major time-consuming tasks in deck replacement, especially for precast concrete I-girders with wide, thin top flanges. The demolition time can be reduced by constructing bridges with connections that provide composite action but also allow for easier deck removal. Two new connection systems were developed, one for concrete girder-to-concrete deck connections and another for steel girder-to-concrete deck connections. For concrete girders, a debonded shear key system was developed. In addition to extensive laboratory tests on push-off specimens, full-scale tests were also performed. This system provided excellent results both for composite action as well as deck removal.

For steel girder-to-concrete deck connections, a new 1¼-in. (32-mm) diameter shear stud was developed to replace the popular ¾-in. (19-mm) and ⅞-in. (22-mm) shear studs. The ¾- and ⅞-in. (19- and 22-mm) shear studs are often provided in two or three rows spaced at 5 in. (125 mm) to 18 in. (450 mm). A large number of small studs results in increased concrete removal time and higher probability of damage to the girder top flange or the studs themselves. The new 1¼-in. (32-mm) stud, which provides approximately twice the capacity of a ⅞-in. (22-mm) stud, would allow positioning in a single row over the girder web. This arrangement would greatly reduce the amount of jackhammering because saw cutting the old concrete could be done very close to the girder centerline. Also, it was found that alternating headed and headless studs provided adequate anchorage to the concrete deck and facilitated deck removal.

This report provides details of stud materials, stud geometry, stud welding, and quality control procedures. Full-scale girder testing demonstrated that welding of 1¼-in. (32-mm) studs can be done at approximately the same rate as for ⅞-in. (22-mm) studs, i.e., two studs per minute. Thus, the total time required to weld the larger studs is about 50 percent of that required to weld the smaller studs. The total material cost is comparable.

Timelines were produced for different bridge-deck replacement systems to compare the speed of construction and provide a means for evaluating the potential time-saving benefits. Finally, a limited cost analysis was done based on the methods depicted in the timelines to determine the cost-effectiveness of systems investigated in this project.

SUGGESTED RESEARCH

In addition to several full-scale tests, extensive tests were performed in the laboratory on various deck and girder-to-deck connection systems. Test results confirmed the expected theoretical behavior and the systems' suitability for implementation. The research team suggests that a program be initiated in cooperation with the Federal Highway Administration and state DOTs for evaluating the various systems in field installations.

Limited testing was performed to determine the fatigue capacity of the 1¼-in. (32-mm) shear studs. The tests show that these studs have higher fatigue resistance than the conventional ⅞-in. (22-mm) studs. Additional tests are needed to provide sufficient data to support changes to the fatigue resistance provisions in the AASHTO specifications.

Use of WWF in bridge decks has shown good promise in the construction speed and quality control areas. However, there is a need for additional evaluations to establish fatigue requirements of bridge deck systems incorporating WWF.

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APPENDIXES A THROUGH Q
UNPUBLISHED MATERIAL

Appendixes A through Q contained in the research agency's final report are not published herein. For a limited time, copies of that report, entitled "Rapid Replacement of Bridge Decks," that contains these appendixes will be available on a loan basis or for purchase ($26.00) on request to NCHRP, Transportation Research Board, Box 289, Washington, D.C., 20055.

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