F. W. Klaiber, T. J. Wipf

Alternative Solutions to Meet the Service Needs of Low Volume Bridges in Iowa

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Sponsored by the
Iowa Department of Transportation
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Iowa DOT Project TR - 452

Final

REPORT

IOWA STATE UNIVERSITY
OF SCIENCE AND TECHNOLOGY

Department of Civil, Construction and Environmental Engineering

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Abstract

There is a nationwide need for a safe, efficient and cost effective transportation system. An essential component of this system is the bridges. Local agencies perhaps have an even greater task than federal and state agencies in maintaining the low volume road (LVR) bridge system due to lack of sufficient resources and funding. The primary focus of this study was to review the various aspects of off-system bridge design, rehabilitation, and replacement. Specifically, a reference report was developed to address common problems in LVR bridges. The source of information included both Iowa and national agencies. This report is intended to be a “user manual” or “tool box” of information, procedures and choices for county engineers to employ in the management of their bridge inventory plus identify areas and problems that need to be researched

To obtain pertinent published information, past Iowa Highway Research Board (HRB) projects were identified and reviewed. These reports were briefly summarized and cross-referenced to the various final reports. In addition, literature reviews were performed to identify pertinent information related to LVR bridge design, rehabilitation/strengthening and replacement. Relatively detailed summaries of rehabilitation/strengthening methods are presented.

A questionnaire was sent to all Iowa county engineers to determine the various problems that are encountered on LVR and their solutions to these problems. Fifty-two Iowa counties responded to the survey. A large percentage of the respondents indicated that they use in-house crews for bridge replacement or rehabilitation. A large part of the in-house work uses steel stringers and wood decks. Approximately one-half of the respondents indicated that they have experience with strengthening superstructure and substructure bridge elements, although adding piling to the substructure was the most common response.

A questionnaire was also sent to other states to obtain similar information. The questionnaire was sent to State DOT’s, County and Local bridge owners and consultants involved with off-system bridge design and rehabilitation. The assistance of the National Association of County Engineers (NACE) was employed to disseminate the survey to all potentially interested parties. In all, several hundred surveys were distributed electronically via email. The response to the questionnaire included a total of 20 states and 70 local agencies nationally. One significant finding is that more appropriate decisions are required in all areas of bridge maintenance, rehabilitation, and replacement. “Data based” decisions through asset/bridge management as well as construction techniques, maintenance procedures, materials, etc. to promote extended life are required. New high performance materials as well as fiber reinforced polymer (FRP) products are currently being researched. Several of these materials show promise for use in off-system bridges since they have excellent durability, require minimal maintenance, and appear to have long life.

A list of research needs was developed, based on the evaluation of the information obtained from this study [i.e. comparing current state-of-the-art with existing problems], input from a research needs forum meeting held last year, and conversations with several county engineers. The research needs list will form the basis of a work plan for developing solutions to current LVR bridge problems.

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1. INTRODUCTION

1.1 Background

In the United States, as well as in countless other countries, problems related to the lack of resources to address bridge deficiencies, replacement and maintenance problems have been well documented. More and more existing bridges are becoming structurally inadequate while funds to repair or replace these bridges are limited. The bridge problem is especially critical in Iowa because there are approximately 26,000 bridges, the majority of which (approximately 85%) are on secondary roads and thus the responsibility of the counties. The number of bridges in Iowa ranks it 6th in the nation (Texas is 1st with close to 42,000 bridges and Ohio is 2nd with over 28,000 bridges). On the other hand, Iowa ranks 25th in population which limits its tax base. Based on these two facts (i.e., a large number of deficient bridges and a limited tax base), not many states have more severe bridge problems than Iowa.

Although in the past 10 years there have been several Iowa Highway Research Board (HRB) sponsored projects that address different low volume road (LVR) bridge problems, none of these projects have provided a summary of the various rehabilitation, replacement or strengthening (RRS) procedures available. Also during the past few years, due to retirements, there are many new county engineers in the state that are unaware of the previous research. As county budgets are limited, it is very important that county engineers are made aware of the various RRS procedures that are available. In addition, more than likely there are numerous existing bridge problems that are not widely known and thus have not been addressed to date.

For these reasons, a compilation of various alternative RRS procedures that are particularly applicable to LVR bridges and current bridge problems that haven't been addressed is needed.

1.2 Objectives

The overall objective of this phase (Phase I) of this project was to develop a reference to document the state of practice in the area of maintenance/rehabilitation/strengthening and to address common problems encountered on the county bridge system. Details on how this reference (i.e. this final report) was developed are presented in the following sections. Another objective was to develop a work plan to address those areas where inadequate information is available, so that appropriate research (Phase II) can be undertaken to develop viable solutions to some of Iowa's LVR bridge problems.

1.3 Research Tasks

To develop the desired reference and workplan, a series of tasks – all of which are associated with the collection of information – were completed.

Through the years there have been numerous bridge related projects sponsored by the Iowa DOT and Iowa DOT HRB. These projects are summarized and categorized in Chapter 2. The projects have been categorized into 10 topics: Abutments, Bridge Alternatives for Low Volume Roads, Bridge Rehabilitation/Strengthening, Concrete Decks, Culverts, Load Rating, Low Water Stream Crossings, Miscellaneous, Prestressed and Reinforced Concrete Beams and Bridges, and Scour. Projects prior to 1980 are presented in Appendix C and are categorized by the same topic areas. As noted in Chapter 2, if additional information is desired on a particular project, the projects' final report may be obtained from the Iowa DOT Material Research website or by contacting the Iowa DOT Research Engineer, Mark Dunn.

To obtain other published literature, several literature searches, including the ISU Library, Internet, Iowa DOT Library, and the Transportation Research Information Service (TRIS) were made. The information located on bridge maintenance, bridge rehabilitation, and bridge strengthening have been summarized and presented in Chapter 4. The considerable information located on bridge replacement alternatives has been summarized and presented in Chapter 5.

To obtain unpublished LVR bridge information, two questionnaires were used. One was sent to all Iowa County Engineers (henceforth referred to as the Iowa Questionnaire) to obtain information on unique solutions to various bridge problems they have encountered plus information on LVR bridge problems they have encountered. The response rate to this questionnaire was 55%.

Pertinent results from a second questionnaire that was part of a national study performed by the authors of this report are also included in this final report. This questionnaire (henceforth referred to as the National Questionnaire) was sent to the membership of the National Association of County Engineers (NACE) plus other bridge owners on the national level. Results from both these questionnaires are presented in Chapter 3.

By reviewing and comparing the information obtained in the literature review, the review of Iowa DOT research, and the results from the two questionnaires with problems identified in the Iowa Questionnaire, a work plan was developed (see Chapter 6). Two other sources of information were used to complete the work plan. A structures research needs focus group meeting (SRNFG) and informal meetings and conversations between the research team and numerous county engineers. In the interactions with county engineers new input on solutions as well as problems were obtained. The bridge problems identified at last year's SRNFG meeting that involved LVR bridges have also been included in the work plan for Phase II.

2. PREVIOUS IOWA DOT BRIDGE RELATED RESEARCH

In this chapter, essentially all bridge-related research sponsored by the Iowa DOT and the Iowa Highway Research Board has been categorized in reverse chronological order. Some of the final reports on these research projects are available on the Iowa DOT Materials Research site [www.dot.state.ia.us/materials/research/research_home.html]. If the final report of interest is not on the web site, a copy may be obtained from the Iowa DOT Research Engineer, Mark Dunn, whose E-mail address is on this web site. Only projects that have been completed are included in this chapter. Information on projects that are in progress may be found also on this website in the section labeled annual research report.

As previously noted, in the following sections abstracts of Iowa DOT sponsored research are categorized. Only the research completed since 1980 is included. Older research has been similarly categorized and included in Appendix C. For cross-reference, Iowa DOT research projects in the following sections and in Appendix C have been listed by number and title in Table 2.1.

2.1 Abutments

HR-292 Validation of Design Recommendations for Integral Abutment Piles

Since integral abutment bridges decrease the initial and maintenance costs of bridges, they provide an attractive alternative for bridge designers. The objective of this project was to develop rational and experimentally verified design recommendations for these bridges.

Field testing consisted of instrumenting two bridges in Iowa to monitor air and bridge temperatures, bridge displacements, and pile strains. Core samples were also collected to determine coefficients of thermal expansion for the two bridges. Design values for the coefficient of thermal expansion of concrete are recommended, as well as revised temperature ranges for the deck and girders of steel and concrete bridges.

A girder extension model was developed to predict the longitudinal bridge displacements caused by changing bridge temperatures. The model is subdivided into segments that have uniform temperatures, coefficients of expansion, and moduli of elasticity. Weak axis pile strains were predicted using a fixed-head model. The pile was idealized as an equivalent cantilever with a length determined by the surrounding soil conditions and pile properties. Both the girder extension model and the fixed-head model are conservative for design purposes.

A longitudinal frame model was developed to account for abutment rotations. The frame model better predicts both the longitudinal displacements and weak axis pile strains than do the simpler models. A lateral frame model is presented to predict the lateral motion of skewed bridges and the associated strong axis pile strains. Full passive soil pressure is assumed on the abutment face.

Two alternatives for the pile design are presented. Alternative One is the more conservative and includes thermally induced stresses. Alternative Two neglects thermally induced stresses but allows for the partial formation of plastic hinges (inelastic redistribution of forces). Ductility criteria are presented for this alternative. Both alternatives are illustrated in a design example.

HR-273 Pile Design and Tests for Integral Abutment Bridges

Expansion joints increase both the initial and maintenance costs of bridges. Integral abutment bridges provide an attractive design alternative because expansion joints are eliminated from the bridge itself. However, the piles in these bridges are subjected to horizontal movement as the bridge expands and contracts during temperature changes. The objective of this research was to develop a method of designing piles for these conditions.

Separate field tests simulating a pile and a bridge girder were conducted for three loading cases: (1) vertical load only, (2) horizontal displacement of pile head only, and (3) combined horizontal displacement of pile head with subsequent vertical load. Both tests (1) and (3) reached the same ultimate vertical load; that is, the horizontal displacement had no effect on the vertical load capacity. Several model tests with a scale factor of about 1:10 were conducted in sand. Experimental results from both the field and model tests were used to develop the vertical and horizontal load-displacement properties of the soil. These properties were used in the finite element computer program Integral Abutment Bridge Two-Dimensional (IAB2D), which was developed under a previous research contract. Experimental and analytical results compared well for the test cases.

Two alternative design methods, both based upon the AASHTO Specifications, were developed. Alternative One is quite conservative relative to IAB2D results and does not permit plastic redistribution of forces. Alternative Two is also conservative when compared to IAB2D, but plastic redistribution is permitted. To use Alternative Two, the pile cross section must have sufficient inelastic rotation capacity before local buckling occurs. Both alternatives are illustrated in design examples for a friction pile and an end-bearing pile.

HR-252 Design of Integral Abutment Bridges

More and more, integral abutment bridges are being used in place of the more traditional bridge designs with expansion releases. In this study, states which use integral abutment bridges were surveyed to determine their current practice in the design of these structures.

To study piles in integral abutment bridges, a finite element program for the soil pile system was developed (1) with materially and geometrically nonlinear, two and three dimensional beam elements and (2) with a nonlinear, Winkler soil model with vertical, horizontal, and pile tip springs. The model was verified by comparison to several analytical and experimental examples.

A simplified design model for analyzing piles in integral abutment bridges is also presented. This model was based on results from previous analytical models and observations of pile behavior. The design model correctly describes the essential behavioral characteristics of the pile and conservatively predicts the vertical load carrying capacity.

2.2 Bridge Alternatives for Low Volume Roads

TR-444 Demonstration Project Using Railroad Flatcars for Low-Volume Bridge

The feasibility of using Railroad Flatcars (RRFCs) as the superstructure on low-volume county bridges has been investigated in research project TR-421. In order to illustrate the constructability, adequacy, and economy of this type of bridge, two RRFC demonstration bridges were designed, constructed, and tested: one in Buchanan County and the other in Winnebago County.

The Buchanan County Bridge was constructed as a single span with 56-ft-long flatcars supported at their ends by new, concrete abutments. The use of concrete in the substructure allowed for an integral abutment at one end of the bridge and an expansion joint at the other end. Reinforced concrete beams serving as longitudinal connections between the three adjacent flatcars were installed to distribute live loads more effectively among the RRFCs. Guardrails and an asphalt milling driving surface completed the bridge.

The Winnebago County Bridge was constructed from 89-ft-long flatcars. Preliminary calculations determined that they were not adequate to span 89 ft as a simple span. Therefore, the flatcars were supported by new, steel-capped piers and abutments at the RRFCs' bolsters and ends, resulting in a 66-ft main span and two 10-ft end spans. Due to the RRFC geometry, the longitudinal flatcar connections between adjacent RRFCs were inadequate to support significant loads, and therefore, transverse, recycled timber planks were utilized to effectively distribute live loads to all three RRFCs. A gravel driving surface was placed on top of the timber planks, and a guardrail system installed to complete the bridge.

Bridge behavior predicted by grillage models for each bridge was validated by strain and deflection data from field tests; it was found that the engineered RRFC bridges have live load stresses significantly below the yield strength of the steel and deflections well below the AASHTO Bridge Design Specification limits. To assist in future RRFC bridge projects, RRFC selection criteria were established for visual inspection and selection of structurally adequate RRFCs. In addition, design recommendations have been developed to simplify live load distribution calculations for design of the bridges. Based on the results of this research, it has been determined that through proper RRFC selection, construction, and engineering, RRFC bridges are a viable, economic replacement system for low-volume road bridges.

TR-421 Use of Railroad Flat Cars for Low Volume Bridges

In an attempt to solve the bridge problem faced by many county engineers, this investigation focused on a low cost bridge alternative that consists of using railroad flatcars (RRFC) as the bridge superstructure. The intent of this study is to determine whether these types of bridges are structurally adequate and potentially feasible for use on low volume roads.

A questionnaire was sent to the Bridge Committee members of the American Association of State Highway and Transportation Officials (AASHTO) to determine their use of RRFC bridges and to assess the pros and cons of these bridges based on others' experiences. It was found that these types of bridges are widely used in many states with large rural populations and they are reported to be a viable bridge alternative due to their low cost, quick and easy installation, and low maintenance.

A main focus of this investigation was to study an existing RRFC bridge that is located in Tama County, IA. This bridge was analyzed using computer modeling and field load testing. The analytical results were compared with those obtained in the field tests, which involved instrumenting the bridge and loading it with a fully loaded rear tandem axle truck. Both sets of data (experimental and theoretical) show that the Tama County Bridge (TCB) experienced very low strains and deflections when loaded and the RRFCs appeared to be structurally adequate to serve as a bridge superstructure. A calculated load rating of the TCB agrees with this conclusion.

Because many different types of flatcars exist, other flatcars were modeled and analyzed. It was very difficult to obtain the structural plans of RRFCs; thus, only two additional flatcars were analyzed. The results of these analyses also yielded very low strains and displacements.

Taking into account the experiences of other states, the inspection of several RRFC bridges in Oklahoma, the field test and computer analysis of the TCB, and the computer analysis of two additional flatcars, RRFC bridges appear to provide a safe and feasible bridge alternative for low volume roads.

TR-410 Investigation of Two Bridge Alternatives for Low Volume Roads – Phase II Vol. 1 & 2

This project continues the research sponsored by the Project Development Division of the Iowa DOT and the Iowa Highway Research Board which addressed numerous bridge problems on the Iowa secondary road system. It is a continuation (Phase 2) of Project HR-382 in which two replacement alternatives, Concept 1 – Steel Beam Precast Units and Concept 2 – Modification of the Benton County Beam-in-Slab Bridge (BISB), were investigated.

Work continued on both of the replacement alternatives in this study, the results of which are presented in two volumes. Results of Concept 1 – Steel Beam Precast Units are presented in Volume 1, while the continued work on Concept 2 – Modification of the BISB is presented in Volume 2.

In previous research (HR-382), a precast unit bridge was developed through laboratory testing. The steel-beam precast unit bridge requires the fabrication of precast double-tee (PCDT) units, each consisting of two steel beams connected by a reinforced concrete deck. The weight of each PCDT unit is minimized by limiting the deck thickness to 4 in. which permits the units to be constructed off-site and then transported to the bridge site. The number of units required is a function of the width of bridge desired. Once the PCDT units are connected, a cast-in-place (CIP) reinforced concrete deck is cast over the PCDT units and the bridge railing attached. Since the steel beam PCDT unit bridge design is intended primarily for use on low-volume roads, used steel beams can be utilized for a significant cost savings.

This project involved three major tasks during the design/fabrication/construction and testing of the replacement bridge. The first task involved documenting the fabrication of the PCDT units through photographs, slides and a video. As part of this effort, a design methodology was developed that includes the development of standard plan sheets from computer templates. The second task involved transporting the completed units to the bridge site where final construction was completed by an independent contractor. The final task involved the service load testing of the bridge at different stages in the construction process and after completion of the construction. This process was also documented through slides and video.

Based upon the construction and service load testing, the steel-beam precast unit bridge was successfully shown to be a viable low volume road bridge alternative. The construction process utilized standard methods resulting in a simple system that can be completed with a limited staff. Results from the service load tests indicated adequate strength for all legal loads. An inspection of the bridge one year after its' construction revealed no change in the bridge's performance.

Also in HR-382 an alternate shear connector (ASC) was developed and subjected to static loading. In this investigation, the ASC was subjected to cyclic loading in both push-out specimens and composite beam tests. Based on these tests, the fatigue strength of the ASC was determined to be significantly greater than that required in typical low volume road single span bridges.

The ASC was also used in the full-scale composite beam specimens tested to determine their service load behavior, ultimate strength and fatigue strength. Two of the specimens had inverted T-beams and one was constructed with an I-beam. Two full-scale two-beam specimens – representing possible bridge systems – were constructed and tested to determine their strength and behavior. These specimens also used the ASC. One of the specimens was very similar to the Canadian steel free deck system, the other – a concrete arch system – was essentially the BISB with concrete removed from the tension side and composite action added.

In all of these tests, the ASC was effective in creating full composite action during the service load tests. None of the specimens experienced a bond failure when loaded to failure. Both the steel-free deck system and concrete arch system – with the

ASC for composite action – were determined to meet AASHTO strength and serviceability requirements and thus are viable low volume road bridge systems.

Each of the systems previously described are relatively easy to construct. Use of the ASC rather than welded studs significantly simplifies the work, equipment, and materials required to develop composite action between the steel beams and the concrete deck.

HR-382 Investigation of Two Bridge Alternatives for Low Volume Roads

In a recent investigation, HR-365 "Evaluation of Bridge Replacement Alternatives for the County Bridge System," several types of replacement bridges that are currently being used on low volume roads were identified. After reviewing the results from HR-365, the research team developed one "new" bridge replacement concept and a modification of a replacement system currently being used.

Both of these bridge replacement alternatives were investigated in this study, the results of which are presented in two volumes. Concept 2 – Modification of the Beam-in-Slab Bridge, was presented in Volume 2 of the final report, while Concept I – Steel Beam Precast Units was presented in Volume 1. Concept 2 involved various laboratory tests of the Beam-in-Slab bridge (BISB) currently being used by Benton County and several other Iowa counties. In this investigation, the behavior and strength of the BISB were determined; a new method, the alternate shear connector (ASC), of obtaining composite action between the steel beams and concrete was also tested. Since the Concept 2 bridge is primarily intended for use on low-volume roads, the system can be constructed with new or used beams.

In addition to the experimental laboratory tests in this investigation, there was a field test in which an existing BISB was service load tested. An equation was also developed for predicting the strength of the ASC.

In this investigation, the existing BISB (L = 50 ft) was determined to be extremely stiff in both the longitudinal and transverse directions, deflecting approximately 1/4 in. when subjected to approximately 100 kips of truck loading.

HR-365 Evaluation of Bridge Replacement Alternatives for County Bridges

The objective of this investigation was to identify, review and evaluate replacement bridges currently being used by various counties in Iowa and surrounding states. Iowa county engineers, county engineers in neighboring states as well as private manufacturers of bridge components, and regional precast/prestressed concrete manufacturers were contacted to determine the most common replacement bridge types being used.

A questionnaire was developed and sent to county engineers in Iowa and several counties in surrounding states. The results of the questionnaire showed that the most common replacement bridges in Iowa are the continuous concrete slab and prestressed concrete bridges; the primary reason these types are used is because of the availability of standard designs and because of their ease of maintenance. Counties seldom construct these types of bridges using their own labor forces, but instead contract the work. However, county forces are used to construct steel stringer, precast reinforced concrete and timber bridges. In general, 69 percent of the counties indicate an ability and willingness to use their own forces to design and construct relatively short span bridges (i.e., 40 ft or less) provided the construction procedures are relatively simple.

Several unique replacement bridge types used in Iowa that are constructed by county forces are documented and presented in this report. Sufficient details are provided to allow county engineers to determine if some of these bridges could be used to resolve some of their own replacement bridge problems.

Based on the results of this study, one new bridge replacement concept and one modification of a current Iowa county bridge replacement concept were developed.

2.3 Bridge Rehabilitation/Strengthening

TR-436 Retrofit Methods for Distortion Cracking Problems in Plate Girder Bridges

In this report, four individual investigations related to similar web gap fatigue problems are presented. Multiple steel girder bridges commonly exhibit fatigue cracking due to out-of-plane displacement of the web near the diaphragm connections. This fatigue-prone web gap area is typically located in negative moment regions of the girders where the diaphragm stiffener is not attached to the top flange. In the past, the Iowa DOT has attempted to stop fatigue crack propagation in these steel girder bridges by drilling holes at the crack tips. Other nondestructive retrofits have been tried; in a particular case on a two-girder bridge with floor beams, angles were bolted between the stiffener and top flange. The bolted angle retrofit has failed in the past and may not be a viable solution for diaphragm bridges. The drilled hole retrofit is often only a temporary solution, so a more permanent and effective retrofit is required. A new field retrofit has been developed that involves loosening the bolts in the connection between the diaphragm and the girders. Research on the retrofit has been initiated; however, no long-term studies on the effects of bolt loosening have been performed.

The intent of this research is to investigate the short-term effects of the bolt loosening retrofit on I-beam and channel diaphragm bridges. The research also addressed the development of a continuous remote monitoring system to investigate the bolt loosening retrofit on an X-type diaphragm bridge over a number of months, ensuring that the measured strain and displacement reductions are not affected by time and continuous traffic loading on the bridge.

The testing for the first three investigations is based on instrumentation of web gaps in a negative moment region on Iowa DOT bridges with I-beam, channel, and X-type diaphragms. One bridge of each type was instrumented with strain gages and deflection transducers. Field tests, using loaded trucks of known weight and configuration, were conducted on the bridges with the bolts in the tight condition and after implementing the bolt loosening retrofit to measure the effects of loosening the diaphragm bolts. Long-term data were also collected on the X-diaphragm bridge by a data acquisition system that collected the data continuously under ambient truck loading. The accuracy and ruggedness of this system for remote bridge monitoring make it a viable system for future bridge monitoring projects in Iowa.

Results indicate that loosening the diaphragm bolts reduces strain and out-of-plane displacement in the web gap, and that the reduction is not affected over time by traffic or environmental loading on the bridge. Reducing the strain in the web gap allows the bridge to support more cycles of loading before experiencing fatigue, thus increase the service life of the bridge.

Two-girder floor beam bridges, may also exhibit fatigue cracking in girder webs. The fourth investigation describes a bridge that was retrofitted with bolted angles at the connection between the top flange and the web stiffener. The retrofit failed and was repaired. A short-term load test was completed to determine the behavior and effectiveness of the repaired retrofit. Testing indicated large displacements, and data suggest the retrofit was ineffective. The study concluded that the bridge should be inspected frequently for signs of failure in the retrofit and cracking in the web.

TR-429 Evaluation of Appropriate Maintenance, Repair and Rehabilitation Methods for Iowa Bridges

Most states, including Iowa, have a significant number of substandard bridges. This number will increase significantly unless some type of preventative maintenance is employed. Both the Iowa DOT and numerous Iowa counties have successfully employed numerous maintenance, repair and rehabilitation (MR&R) strategies for correcting various types of deficiencies. However, successfully employed MR&R procedures are often not systematically documented or defined for others involved in bridge maintenance. This study addressed the need for a standard bridge (MR&R) manual for Iowa with emphasis on secondary road applications. As part of the study, bridge MR&R activities that are relevant to the state of Iowa have been systematically categorized into a manual, in a standardized format. Where pertinent, design guidelines have been presented.

Material presented in this manual is divided into two major categories: 1) Repair and Rehabilitation of Bridge Superstructure Components, and 2) Repair and Rehabilitation of Bridge Superstructure Components. There are multiple subcategories within both major categories that provide detailed information. Some of the detailed information includes step-by-step procedures for accomplishing MR&R activities, material specifications and detailed drawings. The source of information contained in the manual came from technical literature and from information provided by several Iowa county engineers. A questionnaire was sent to all 99 counties in Iowa to solicit information; as a follow up to the questionnaire, the research team personally solicited input from many Iowa counties.

HR-397 Field/Laboratory Testing of Damaged Prestressed Concrete Girder Bridges

Due to frequent accidental damage to prestressed concrete (P/C) bridges caused by impact from overheight vehicles, a project was initiated to evaluate the strength and load distribution characteristics of damaged P/C bridges. Through a comprehensive literature review, it was determined that only a few references pertain to the assessment and repair of damaged P/C beams. No reference was found on the testing of a damaged bridge(s) as well as the damaged beams following their removal.

Structural testing of two bridges was conducted in the field. The first bridge tested, damaged by accidental impact, was the westbound (WB) I-680 bridge in Beebeetown, Iowa. This bridge had significant damage to the first and second beams consisting of extensive loss of section and the exposure of numerous strands. The second bridge, the adjacent eastbound (EB) structure, was used as a baseline of the behavior of an undamaged bridge. Load testing concluded that a redistribution of load away from the damaged beams of the WB bridge was occurring. Subsequent to these tests, the damaged beams in the WB bridge were replaced and the bridge retested. The repaired WB bridge behaved, for the most part, like the undamaged EB bridge indicating that the beam replacement restored the original live load distribution.

A large-scale bridge model constructed for a previous project was tested to determine the changes in behavior due to incrementally applied damage consisting initially of only concrete removal and then concrete removal and strand damage. A total of 180 tests were conducted with the general conclusion that for exterior beam damage, the bridge load distribution characteristics were relatively unchanged until significant portions of the bottom flange were removed along with several strands. A large amount of the total applied moment to the exterior beam was redistributed to the interior beam of the model.

Four isolated P/C beams were tested, the two removed from the Beebeetown bridge and two from the aforementioned bridge model. Beam 1W from the Beebeetown bridge was tested in an “as removed” condition to obtain the baseline characteristics of a damaged beam. Beam 2W was retrofit with carbon fiber reinforced polymer (CFRP) longitudinal plates and transverse stirrups to strengthen the section. The strengthened Beam was 12% stronger than Beam 1W. Beams 1 and 2 from the bridge model were also tested. Beam 1 was not damaged and served as the baseline behavior of a “new” beam while Beam 2 was damaged and repaired again using CFRP plates. Prior to debonding of the plates from the beam, the behavior of both Beams 1 and 2 was similar. The retrofit beam attained a capacity greater than an undamaged beam prior to plate debonding.

Analytical models were created for the undamaged and damaged center spans of the WB bridge; stiffened plate and refined grillage models were used. Both models accurately predicted the deflections in the tested bridge and should be similarly accurate in modeling other P/C bridges. The moment fractions per beam were computed using both models for the undamaged and damaged bridges. The damaged model indicated a significant decrease in moment in the damaged beams and a redistribution of load to the adjacent curb and rail as well as to the undamaged beam lines.

HR-393 Preventing Cracking at Diaphragm/Plate Girder Connections in Steel Bridges

Some of the Iowa DOT continuous, steel, welded plate girder bridges have developed web cracking in the negative moment regions at the diaphragm connection plates. The cracks are due to out-of-plane bending of the web near the top flange of the girder. The out-of-plane bending occurs in the “web-gap”, which is the portion of the girder web between (1) the top of the fillet welds attaching the diaphragm connection plate to the web and (2) the fillet welds attaching the flange to the web. A literature search indicated that four retrofit techniques have been suggested by other researchers to prevent or control this type of cracking: (1) drilling holes at crack tip locations, (2) increasing the web gap length, (3) providing rigid attachment between the connection plate and the tension flange, and (4) removing the diaphragms.

To eliminate the problem in new bridges, current AASHTO Specifications require a positive attachment between the connection plate and the top (tension) flange. Applying this requirement to existing bridges is expensive and difficult. The Iowa DOT has relied primarily on the hole-drilling technique to prevent crack extension once cracking has occurred; however, the literature indicates that hole-drilling alone may not be entirely effective in preventing crack extension.

The objective of this research was to investigate experimentally a method proposed by the Iowa DOT to prevent cracking at the diaphragm/plate girder connection in steel bridges with X-type or K-type diaphragms. The method consists of loosening the bolts at some connections between the diaphragm diagonals and the connection plates.

The experimental investigation of the method included selecting and testing five bridges: three with X-type diaphragms and two with K-type diaphragms. During 1996 and 1997, these bridges were instrumented to obtain the response at various locations (web gaps, diaphragms, and girder flanges and webs) before and after implementing the method. Bridges were subjected to loaded test trucks traveling in different lanes with speeds varying from crawl speed to 65 mph to determine the effectiveness of the proposed method.

The results of the study show that the effective of out-of-plane loading was confined to widths of approximately 4 in. on either side of the connection plates. They also demonstrate that the stresses in gaps with drilled holes were higher than those in gaps without cracks, implying that the hole drilling technique is not sufficient to prevent crack extension. The behavior of the web gaps in X-type diaphragm bridges was greatly enhanced by the proposed method as the stress range and out-of-plane distortion were reduced by at least 42% at the exterior girders. For bridges with K-type diaphragms, a similar trend was obtained. However, the stress range increased in one of the web gaps after implementing the proposed method. Other design aspects (wind, stability of compression flange, and lateral distribution of loads) must be considered when deciding whether to adopt the proposed method. Considering the results of this investigation, the proposed method can be implemented for X-type diaphragm bridges. Further research is recommended for K-type diaphragm bridges.

HR-333 Design Methodology for Post-Tension Strengthening of Continuous Span Bridges

This manual presents two methods for strengthening continuous span composite bridges: post-tensioning of the positive moment regions of the bridge stringers and the addition of superimposed trusses at the piers. The use of these two systems is an efficient method of reducing flexural overstresses in undercapacity bridges. However, before strengthening a given bridge, other deficiencies (inadequate shear connection, fatigue problems, extensive corrosion) should be addressed.

Since continuous span composite bridges are indeterminate structures, there is longitudinal and transverse distribution of the strengthening axial forces and moments.

This manual basically provides the engineer with a procedure for determining the distribution of strengthening forces and moments throughout the bridge. As a result of the longitudinal and transverse force distribution, the design methodology presented in this manual for continuous span composite bridges is extremely complex. To simplify the procedure, a spreadsheet has been developed for use by practicing engineers. The force and moment distribution fraction formulas developed in this manual are primarily for the Iowa DOT V12 and V14 three-span, four-stringer bridges. The formulas developed may be used on other bridges if they are within the limits stated in this manual.

HR-323 Development of Evaluation, Rehabilitation, and Strengthening Concepts For Low Volume Bridges

This report contains an evaluation and design manual for strengthening and replacing low volume steel stringer and timber stringer bridges which have the greatest need for cost-effective strengthening methods. Procedures for strengthening these two types of structures have been developed. Various types of replacement bridges have also been included so that the most cost effective solution for a deficient bridge may be obtained.

The key result of this study is an extensive compilation, of the most effective techniques for strengthening deficient existing bridges. The replacement bridge types included have been used in numerous low volume applications in surrounding states, as well as in Iowa. An economic analysis for determining the cost-effectiveness of the various strengthening methods and replacement bridges is also included in the manual.

HR-308 Strengthening of Existing Continuous Span Steel Beam Concrete Deck Bridges by Post Tensioning

The need to upgrade a large number of understrength and obsolete bridges in the United States has been well documented in the literature. Through several Iowa DOT projects, the concept of strengthening simple span bridges by post-tensioning has been developed. The purpose of the project described in this report was to investigate the use of post-tensioning for strengthening continuous composite bridges. In a previous, successfully completed investigation, (HR-287), the feasibility of strengthening continuous, composite bridges by post-tensioning was demonstrated on a laboratory 1/3-scale-model bridge.

The bridge selected for strengthening was in Pocahontas County near Fonda, Iowa, on County Road N28. Based on analyses it was determined that post-tensioning of the positive moment regions of both the interior and exterior beams was required. During the summer of 1988, the strengthening system was installed along with instrumentation to determine the bridge’s response and behavior. Before and after post-tensioning, the bridge was subjected to truck loading (one or two trucks at various predetermined critical locations) to determine the effectiveness of the strengthening system.

Approximately one year after the initial strengthening, in the summer of 1989, the bridge was retested to identify any changes in its behavior. Post-tensioning forces were removed to reveal any losses over the one-year period. Post-tensioning was reapplied to the bridge, and the bridge was tested using the loading program previously described.

Although considerably more involved than the post-tension strengthening of single-span bridges, the post-tensioning of continuous-span bridges was determined to be a feasible, practical, strengthening technique.

HR-302 Alternate Method of Bridge Strengthening

The need for upgrading a large number of understrength and obsolete bridges in the United States has been well documented in the literature. The purpose of this project was to investigate two additional strengthening alternatives that may be more efficient than other commonly used procedures.

In Part I of this report, the strengthening of existing steel stringers in composite steel-beam concrete deck bridges by providing partial end restraint was shown to be feasible. Various degrees of end restraint were investigated on a full-scale bridge stringer as well as on an existing 1/3 scale bridge model. By varying the amount of restraint, different amounts of strain reduction can be obtained.

Part 2 of this report summarizes the research that was undertaken to strengthen the negative moment regions of continuous, composite bridges. Two schemes were investigated: post-compression of stringers and superimposed trusses within the stringers. Both schemes were designed to apply positive moment to the negative moment regions of continuous stringers and thus reduce the stresses resulting from service loads. Both schemes were effective in reducing bottom flange stresses; however, the post-compression scheme slightly increased the top flange stresses because of the tension applied to the section. The superimposed truss was very effective in reducing both the top and bottom flange stresses as it applied only positive moment to the mockup.

HR-287 Strengthening Existing Continuous Composite Bridges

The need for upgrading a large number of understrength and obsolete bridges in the United States is well known; unfortunately Iowa has many of the bridges that require upgrading. Iowa began designing and constructing continuous span, composite bridges earlier than other states, and consequently, there are many such bridges in Iowa. Because of changes in bridge design standards and increases in legal truck loads, a considerable number of the continuous, composite bridges in Iowa require posting for reduced loads.

The posted bridges, if in otherwise good condition, often can be strengthened at a cost considerably less than replacement cost. Because strengthening should be based on adequate testing and design information, the research described in this report was conducted to investigate the potential of strengthening continuous bridges by post tensioning.

The research program consisted of the following: a literature review, selection and rating of a prototype continuous composite bridge, tests of a one-third-scale continuous composite bridge model (3 spans: 41 ft-11 in. long by 8 ft-8 in. wide), finite element analysis of the bridge model, and the testing of a full scale composite beam mockup of a negative moment region.

Results from the investigation indicated that the strengthening of continuous, composite bridges is feasible. The primary objective in applying the post tensioning should be to provide moments opposite to those produced by live and dead loads.

Longitudinal distribution of post tensioning must always be considered if only exterior or only interior beams are post tensioned. Changes in tension in tendons may be either beneficial or detrimental when live loads are applied to a strengthened bridge and thus must be carefully considered in design.

HR-238 Strengthening Existing Single Span Steel Beam Concrete Deck Bridges

A considerable number of single span, composite concrete deck and steel beam bridges in Iowa, as well as in most states, presently cannot be rated to carry today's design loads. This problem was initially addressed in the research project, HR-214, "Feasibility Study of Strengthening Existing Single Span Steel Beam Concrete Deck Bridges", henceforth referred to as Phase I. The research of Phase I verified that post-tensioning can be used to provide strengthening to the composite bridge in question. This was determined analytically, using a modification of the orthotropic plate theory, and experimentally, through testing of various post-tensioning schemes on a half-scale model bridge.

Because of the importance of the strengthening problems and the wide range of variables, a second research study was undertaken which involved two phases. The primary emphasis of Phase II involved the strengthening of two full-scale prototype bridges. One of the bridges was a prototype of the model bridge tested during Phase I; the other bridge was longer and skewed. In addition to this field work, Phase II also involved a considerable amount of laboratory work to obtain data on the angle-plus-bar shear connectors.

Phase III of the investigation involved the inspection of the two strengthened bridges approximately every three months for a period of two years. Both bridges are tested under service loads to determine if there are any behavioral changes from the initial service load tests.

In line with the overall objective of Phase II of this study, the following secondary objectives were established:

- Determine load distribution before and after post tensioning in actual bridges.
- Determine the vertical load and post tension force distribution in skewed bridges.
- Determine the strength and behavior of angle-plus-bar shear connectors and compare it with the strength and behavior of other shear connectors, such as studs and channels.
- Develop a simple method of adding shear connectors to existing bridges and evaluate their strength and effectiveness.

HR-214 Feasibility Study of Strengthening Existing Single Span Steel Beam Concrete Deck Bridges

This was the initial project in the use of post-tensioning to strengthen composite, steel stringer bridges. As a result of changes in the AASHTO specifications, increases in Iowa design loads, and deterioration, many Iowa bridges either must be posted at reduced load limits or must be strengthened. In this investigation, an one-half scale, single-span, composite steel stringer bridge was constructed and subjected to various post-tensioning schemes. Based on the experimental results from the model testing and theoretical analysis, it was determined that post-tensioning could be used to strengthen single-span, composite steel stringer bridges.

2.4 Concrete Decks

HR-310 Composite Precast Prestressed Concrete Bridge Slabs

Precast prestressed concrete panels have been used as subdecks in bridge construction in Iowa and other states. To investigate the performance of these types of composite slabs at locations adjacent to abutment and pier diaphragms in skewed bridges, a research project which involved surveys of design agencies and precast producers, field inspections of existing bridges, analytical studies, and experimental testing was conducted.

The survey results from the design agencies and panel producers showed that standardization of precast panel construction would be desirable, that additional inspections at the precast plant and at the bridge site would be beneficial, and that some form of economical study should be undertaken to determine actual cost savings associated with composite slab construction.

Three bridges in Hardin County, Iowa were inspected to observe general geometric relationships, construction details, and to note visually the condition of the bridges. Hairline cracks were observed beneath several of the prestressing strands in many of the precast panels, and a slight discoloration of the concrete was seen beneath most of the strands. Also, some rust staining was visible at isolated locations on several panels. Based on the findings of these inspections, future inspections are recommended to monitor the condition of these and other bridges constructed with precast panel subdecks.

Five full-scale composite slab specimens were constructed in the ISU Structural Engineering Laboratory. One specimen modeled bridge deck conditions which are not adjacent to abutment or pier diaphragms, while the other four specimens represented the geometric conditions which occur for skewed diaphragms of 0, 15, 30, and 40 degrees. The specimens were subjected to service wheel loads and factored service wheel loads at several locations on the slab surface and to concentrated loads which failed the composite slab. To analytically evaluate the nominal strength for a composite slab specimen, yield-line and punching shear theories were applied.

The development lengths for the prestressing strands in the rectangular and trapezoidal shaped panels was qualitatively investigated by monitoring strand slippage at the ends of selected prestressing strands. The initial strand transfer length was established experimentally by monitoring concrete strains during strand detensioning; this length was verified analytically by a finite element analysis. Composite behavior of the slab specimens was evaluated by monitoring slippage between a panel and the topping slab and by computation of the difference in the flexural strains between the top of the precast panel and the underside of the topping slab.

The static load strength performance of the composite slab specimens significantly exceeded the design load requirements. At service and factored level loads, the joint between precast panels did not appear to influence the load distribution along the length of the specimens. Based on the static load strength of the composite slab specimens, the continued use of precast panels as subdecks in bridge deck construction was recommended.

HR-192 Evaluation of Dense Bridge Floor Concrete Using High Range Water Reducer

Much effort is being expended by various state, federal, and private organizations to protect and preserve concrete bridge floors. The generally recognized culprit is the chloride ion from the deicing salt reaching the reinforcing steel, and along with water and oxygen, causing corrosion. The corrosion process exerts pressure which eventually causes cracks and spalls in the bridge floor. To prevent corrosion, the reinforcing steel has been coated; various types of "waterproof" membranes have been placed on the deck surface; decks have been surfaced with dense and modified concrete; decks have been electrically protected; and attempts to internally seal the concrete have been made. No one method has been proven and accepted by the various government agencies as being the "best" when considering the initial cost, application effort, length and effectiveness of protection.

This research is an effort to prevent bridge deck deterioration by using a high range water reducing admixture (HRWR) to obtain a dense concrete that is workable during construction to prevent chloride intrusion.

The objectives of this research project were:

- To determine the feasibility of proportioning, mixing, placing and finishing a dense Portland cement concrete in a bridge floor using conventional mixing, placing and finishing equipment,
- To determine the economics, longevity, maintenance performance and protective qualities of a dense Portland cement concrete bridge floor when using a HRWR.

The purpose of a HRWR is to produce a dense, high quality concrete at a low water-cement ratio with adequate workability. A low water-cement ratio contributes greatly to increased strength. The normal 7-day strength of untreated concrete would be expected in 3 days using a superplasticizer. A dense concrete also has the desirable properties of excellent durability and reduced permeability.

The addition of a HRWR will result in a higher quality, denser, higher strength Portland cement concrete that can be placed using conventional equipment. Such a dense concrete, with a water/cement ratio of approximately 0.3 to 0.35, would be expected to be much less permeable and thus retard the intrusion of chloride. With care and attention given to obtaining the design cover over steel (2 1/2 in. clear), it is hoped that protection for the design life of the structure will be obtained.

2.5 Culverts

HR-373 Investigation of Plastic Pipe for Highway Applications, Phases I & II

Phase I: In the past, culvert pipes were made only of corrugated metal or reinforced concrete. However, in recent years, several manufacturers have made pipe of lightweight plastic – for example, high density polyethylene (HDPE). It appears that there are several highway applications in which HDPE pipe would be an economically favorable alternative. The objective of this study was to review and evaluate the use of HDPE pipe in roadway applications. Structural performance, soil-structure interaction, and the sensitivity of the pipe to installation were investigated.

A questionnaire was developed and sent to all Iowa county engineers to learn of their use of HDPE pipe. Responses indicated that the majority of county engineers were aware of the product but were not confident in its ability to perform as well as conventional materials. Counties currently using HDPE pipe in general only use it in driveway crossings.

In an effort to develop more confidence in the pipe's performance parameters, this research included laboratory tests to determine the ring and flexural stiffness of HDPE pipe provided by various manufacturers. Flexural testing revealed that pipe profile had a significant effect on the longitudinal stiffness and that strength could not be accurately predicted on the basis of diameter alone.

Realizing that the soil around a buried HDPE pipe contributes to the pipe stiffness, the research team completed a limited series of tests on buried 3 ft-diameter HDPE pipe. The tests simulated the effects of truck wheel loads above the pipe that had 2 ft of cover. These tests indicated that the type and quality of backfill significantly influences the performance of HDPE pipe, however, after a certain point, no additional strength is realized by increasing the quality of the backfill.

Phase II: It is generally accepted that HDPE performs well under live loads with shallow cover, provided the backfill is well compacted. Although industry standards required carefully compacted backfill, poor inspection and/or faulty construction may result in soils that provide inadequate restraint at the springlines of the pipes thereby causing failure. The objectives of this study were: (1) to experimentally define a lower limit of compaction under which the pipes perform satisfactorily, (2) to quantify the increase in soil support as compaction effort increases, (3) to evaluate pipe response for loads applied near the ends of the buried pipes, (4) to determine minimum depths of cover for a variety of pipes and soil conditions by analytically expanding the experimental results through the use of the finite element program CANDE.

The test procedures used here are conservative especially for low-density fills loaded to high contact stresses. The failures observed in these tests were the combined effect of soil bearing capacity at the soil surface and localized wall bending of the pipes. Under a pavement system, the pipes' performance would be expected to be considerably better. With those caveats, the following conclusions are drawn from this study:

- Glacial till compacted to 50% and 80% provides insufficient support; pipe failure occurs at surface contact stresses lower than those induced by highway trucks.
- At moderate tire pressures; i.e. contact stresses, deflections are reduced significantly when backfill density is increased from about 50 pcf to 90 pcf. Above that unit weight, little improvement in the soil-pipe system is observed.
- Although pipe stiffness may vary as much as 16%, analyses show that backfill density is more important than pipe stiffness in controlling both deflections at low pipe stresses and at the ultimate capacity of the soil-pipe system. The rate of increase in ultimate strength of the system increases nearly linearly with increasing backfill density.

- For flowable fill backfill, the ultimate capacity of the pipes is nearly doubled and at the upper limit of highway truck tire pressures, deflections are negligible.
- The minimum soil cover depths, determined from the CANDE analysis, are controlled by the 5% deflection criterion. The minimum soil cover height is 12 in. Pipes with the poor silt and clay backfills with less than 85% compaction require a minimum soil cover height of 24 in.

HR-370 Pipe Rehabilitation With Polyethylene Pipe Liners

Corroded, deteriorated, misaligned, and distorted drainage pipes can cause a serious threat to a roadway. Normal practice is to remove and replace the damaged drainage structure. An alternative method of rehabilitating these structures is to slip line them with a polyethylene liner.

Twelve drainage structures were slip lined with polyethylene liners during 1994 in Iowa. Two types of liners installed were “Culvert Renew” and “Snap-Tite”. It was found that the liners could be easily installed by most highway, county, and city maintenance departments. The liners restore the flow and increase the service life of the original drainage structure. The liners were found to be cost competitive with the removal and replacement of the existing drainage structure. Slip lining has the largest economic benefit when the roadway is paved, the culvert is under a deep fill, or traffic volumes are high. The annular space between the original pipe and the liner was filled with flowable mortar. Care should be taken to properly brace and grout the annular space between the liner and the culvert.

HR-362 Design Methodology for Corrugated Metal Pipe Tiedowns

This investigation is the final phase of a three-part study whose overall objectives were to determine if a restraining force is required to prevent inlet uplift failures in corrugated metal pipe (CMP) installations, and to develop a procedure for calculating the required force when restraint is required.

In the initial phase of the study (HR-306), the extent of the uplift problem in Iowa was determined and the forces acting on a CMP were quantified. In the second phase of the study (HR-332), laboratory and field tests were conducted. Laboratory tests measured the longitudinal stiffness of CMP and a full scale field test on a 10 ft diameter CMP with 2 ft of cover determined the soil-structure interaction in response to uplift forces.

In this phase, a buried 8 ft CMP was tested with and without end-restraint and with various configurations of soil at the inlet end of the pipe. A total of four different soil configurations were tested; in all tests, the soil cover was 2 ft. Data from these tests were used to verify the finite element analysis model that was developed in this phase of the research. Both experiments and analyses indicate that the primary soil contribution to uplift resistance occurs in the foreslope and that depth of soil cover does not affect the required tiedown force.

Using experimental and theoretical results, design charts were developed which engineers can use to determine for a given situation if a restraint force is required to prevent an uplift failure. If an engineer determines restraint is needed, the design charts provide the magnitude of the required force. The design charts are applicable to six gages of CMP, four flow conditions, and two types of soil.

HR-354 An Engineering Study to Design Triple Box Culvert Standards

Standard plans are very cost effective for counties since they eliminate duplication of effort in performing detailed design and drafting work. Counties in the past have made extensive use of the culvert standards developed through a Highway Research Advisory Board project. To develop box culvert designs, most counties request computer generated design information from the Iowa DOT Bridge Design Office and perform the required detailing and estimating. This procedure is time consuming for both the Office of Bridge Design and the counties. New box culvert standards would eliminate much of the time and expense counties are expending on culvert plans. The objective of this study was to develop triple reinforced concrete box culvert standards, headwalls, barrel sections, and bell joints for use by the Iowa counties.

The study involved developing details and quantities for 336 different box culverts. Seven different triple box culverts were designed for sizes from 101 x 81 to 121 x 121, with 12 different fills and 4 different skewed headwalls (0°, 15°, 30° and 45°). These standards included details for bell joints and will be similar in layout. The Office of Bridge Design provided computer generated design information needed to develop the barrel cross-sections for the triple box culverts to the engineering firm. The final report is a set of reinforced concrete triple box culvert standards which include barrel sections, headwalls and bell joints.

HR-332 Design Methodology for Corrugated Metal Pipe Tiedowns

Questionnaires were sent to transportation agencies in all 50 states in the U.S., to Puerto Rico, and all provinces in Canada asking about their experiences with uplift problems of corrugated metal pipe (CMP). Responses were received from 52 agencies who reported 9 failures within the last 5 years. Some agencies also provided design standards for tiedowns to resist uplift. These responses verified the earlier conclusion based on responses from Iowa county engineers that a potential uplift danger exists when end restraint is not provided for CMP and that existing designs have an unclear theoretical or experimental basis.

In an effort to develop more rational design standards, the longitudinal stiffness of three CMP ranging from 4 to 8 ft in diameter were measured in the laboratory and a theoretical model was developed to conservatively evaluate the stiffness of pipes with a variety of gages and corrugation geometries.

Recognizing that soil over and around CMPs will contribute to their stiffnesses, one field test was conducted on a 10 ft diameter pipe. The test was conducted with 2 ft of soil cover and a foreslope of 2:1. This test indicated that the soil cover significantly increased the stiffness of the pipe.

HR-314 Air-formed Arch Construction – Crawford County
HR-313 Air Formed Arch Culvert Construction – Washington County

Iowa's secondary roads contain nearly 15,000 bridges which are less than 40 ft in length. Many of these bridges were built several decades ago and need to be replaced. Box culvert construction has proven to be an adequate bridge replacement techniques. Recently a new bridge replacement alternative, called the Air-Form method, has emerged which has several potential advantages over box culvert construction. This new technique uses inflated balloons as the interior form in the construction of an arch culvert. Concrete is then shotcreted onto the balloon form.

The objective of these research projects was to construct two air formed arch culverts – one in Crawford County and one in Washington County, to determine the applicability of the Air-Form technique as a county bridge replacement alternative. The semi-circular air-formed arch in Crawford County had a 9 ft radius while the one in Washington County had a 12 ft radius; both air-formed arches were 52 ft in length.

The projects had the following results:

- The Air-form method can be used to construct a structurally sound arch culvert.
- The method must become more economical if it is to compete with box culverts.

Continued monitoring should be conducted to evaluate the long-term performance of the Air-Form method.

HR-306 Investigation of Uplift Failures in Flexible Pipe Culverts

This study was precipitated by several failures of corrugated metal pipe culverts apparently due to inlet flotation. In a survey, Iowa County Engineers reported 31 culvert failures on pipes greater than 72 in. in diameter in eight Iowa counties within the past five years. While no special hydrologic, topographical or geotechnical environments appeared to be particularly susceptible to failure, most failures seemed to be on inlet control pipes. Geographically, most of the failures were in the southern and western sections of Iowa.

In this investigation, the forces acting on a culvert pipe were quantified. A worst case scenario, where the pipe is completely plugged, was evaluated to determine the magnitude of force that must be provided by a tie-down system or headwall to prevent flotation failures. Concrete headwalls or slope collars are recommended for most pipes over 4 ft in diameter.

HR-219 Settlement at Culverts

Past construction methods have resulted in the need for leveling wedges of asphaltic cement concrete or mud jacking at locations where a reinforced concrete box culvert was replaced with a pipe culvert. With the restraint of limited funds, more reconstruction, restoration, repair and resurfacing projects will be constructed. This will result in using existing pavements with trench replacement of small box culverts. The installation of culverts in trenches only permits the use of hand tampers or small vibratory plate compactors for compaction. To increase the size of the trench and increase the size of the full depth pavement repair is expensive.

The objective of this research is to develop and evaluate various methods of backfilling adjacent to culverts to reduce the need for future leveling or mud jacking to maintain a smooth pavement surface.

Of all five methods of backfill, the Class "C" Bedding with Moisture Control is the only method which would have required any maintenance leveling. Of all methods of backfill, the backfill with the excavated soil was the most expensive method used. The Class "C" Stone Backfill had the least settlement of the backfill methods. When considering the cost of material and time used, the total cost of the Class "C" Stone Backfill was nearly the same as the soil backfill. The most cost effective method with a minor amount of total settlement was the flowable mortar backfill. In fact, the contractor who installed these culverts indicated that he would suggest flowable mortar backfill in similar situations on future projects.

More continuous Iowa DOT inspection was provided on this research project than is normally available for other culvert pipe installations. This forced the contractor to follow specifications and plans more rigidly than if an inspector was only periodically checking the operation.

2.6 Load Rating

TR-445 Development of Bridge Load Testing Process for Load Evaluation

Recent reports indicate that of the over 25,000 bridges in Iowa, slightly over 7,000 (29%) are either structurally deficient or functionally obsolete. While many of these bridges may be strengthened or rehabilitated, some simply need to be replaced. Before implementing one of these options, one should consider performing a diagnostic load test on the structure to more accurately assess it's load carrying capacity. Frequently, diagnostic load tests reveal strength and serviceability characteristics that exceed the predicted codified parameters. Usually, codified parameters are very conservative in predicting lateral load distribution characteristics and the influence of other structural attributes. As a result, the predicted rating factors are typically conservative. In cases where theoretical calculations show a structural deficiency, it may be very beneficial to apply a "tool" that utilizes a more accurate theoretical model which incorporates field-test data. At a minimum, this approach results in more accurate load ratings and many times results in increased rating factors. Bridge Diagnostics, Inc., (BDI) developed hardware and software that is specially designed for performing bridge ratings based on data obtained from physical testing.

To evaluate the BDI system, the research team performed diagnostic load tests on seven "typical" bridge structures: three steel-girder bridges with concrete decks, two concrete slab bridges, and two steel-girder bridges with timber decks. In addition, a steel-girder bridge with a concrete deck previously tested and modeled by BDI was investigated for model verification purposes. The tests were performed by attaching BDI strain transducers on the bridge at critical locations to measure strains resulting from truck loading positioned at various locations on the bridge.

The field test results were used to develop and validate analytical rating models. Based on the experimental and analytical results, it was determined that bridge tests could be conducted relatively easy, that accurate models could be generated with the BDI software, and that the load ratings, in general, were greater than the ratings obtained using the codified LFD Method (according to AASHTO Standard Specifications for Highway Bridges).

HR-390 Testing of Old Reinforced Concrete Bridges

According to data obtained from the National Bridge Inventory (NBI), there are over 12,000 reinforced concrete bridges within the state of Iowa on the county road system. Of these 12,000 bridges, over 1,900 are considered structurally deficient based on traditional analytical rating methods. Current rating practices are based on the procedures outlined in the “Manual for Maintenance Inspection of Bridges I” which typically underestimate the load carrying capacity of existing bridges. Since the cost of replacing all these bridges is prohibitive, a procedure is needed to give a more accurate assessment of a bridge’s actual safe load carrying capacity. The objective of this research project was to service load test a representative sample of old reinforced concrete bridges with the results being used to create a database so the performance of similar bridges could be predicted.

The types of bridges tested included two reinforced concrete open spandrel arches, two reinforced concrete filled spandrel arches, one reinforced concrete slab bridge, and one two span reinforced concrete stringer bridge. The testing of each bridge consisted of applying a static load at various locations on the bridges and monitoring strains and deflections in critical members. The load was applied by a tandem axle dump truck with varying magnitudes of load. At each load increment, the truck was stopped at predetermined transverse and longitudinal locations and strain and deflection data were obtained.

The response of a majority of the bridges tested was considerably lower than that predicted by analysis. Thus, the safe load carrying capacities of the bridges were greater than that predicted by the analytical models, and in a few cases, the load carrying capacities were found to be three or four times greater than calculated values. However, the test results of one bridge were lower than that predicted by analysis and thus resulted in the analytical rating being reduced. The results of the testing verified that traditional analytical methods, in most instances, are conservative and that the safe load carrying capacities of a majority of the reinforced concrete bridges are considerably greater than what one would determine on the basis of analytical analysis alone. In extrapolating the results obtained from diagnostic load tests to levels greater than those placed on the bridge during the load test, care must be taken to ensure safe bridge performance at the higher load levels.

HR-239 Load Ratings for Secondary Bridges

In this investigation, twenty-two Iowa DOT bridge standards for three types of bridges (Precast Beam (H Series), Reinforcing Concrete Slab (J Series), and Steel Beam (V Series)) were rated for the AASHTO HS 20-44 and three different Iowa legal vehicles. The Inventory and Operating Ratings are based on the standard AASHTO HS 20-44 loading while the legal load ratings were based on previously noted three Iowa legal vehicles using allowable Operating stresses. All bridges were rated for two lanes of traffic. Rating applies only to those bridges that were built according to the applicable bridge standards, have no structural deterioration or damage, and have an added wearing surface of 1/2 in. or less.

2.7 Low Water Stream Crossings

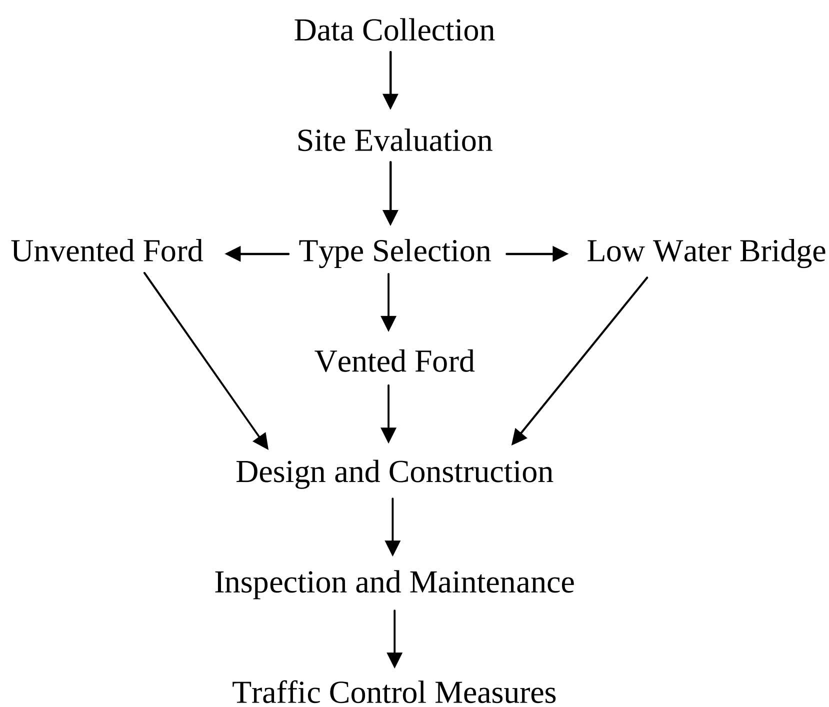
HR-453M Low Water Stream Crossings: Design and Construction Recommendations and Design Guide (2003)

Most Iowa counties maintain low volume roads with at least one bridge or culvert that is structurally deficient or obsolete. In some counties, the percentage of deficient drainage structures may be as high as 62%. Replacement with structures of similar size would require large capital expenditures that many counties cannot afford. Low water stream crossings (LWSCs) may be an acceptable low-cost alternative in some cases.

LWSCs are particularly suitable for low volume roads across streams where the normal volume of flow is relatively low. There are three common types of LWSCs: unvented fords, vented fords, and low water bridges.

LWSC sites, types, and designs should be carefully selected since low water stream crossings will be flooded periodically, requiring the road to be temporarily closed to traffic.

This guide provides a simplified approach to LWSC selection and design. After weighing public opinion and considering potential liability, jurisdictions interested in low water stream crossings should follow these steps:



TR-453 Low Water Stream Crossings: Design and Construction Recommendations

Most counties have some bridges that are no longer adequate, and are faced with large capital expenditure for replacement structures of the same size. In this regard, low water stream crossings (LWSCs) can provide an acceptable, low cost alternative to bridges and culverts on low volume and reduced maintenance level roads. In addition to providing a low cost option for stream crossings, LWSCs have been designed to have the additional benefit of streambed stabilization.

Considerable information on the current status of LWSCs in Iowa, along with insight of needs for design assistance, was gained from a survey of county engineers that was conducted as part of this research.

This document provides guidelines for the design of LWSCs. There are three common types of LWSCs: unvented ford, vented ford with pipes, and low water bridges. The selection of one of these depends on stream geometry, discharge, importance of road, and budget availability. To minimize exposure to tort liability, local agencies using low water stream crossings should consider adopting reasonable selection and design criteria and certainly provide adequate warning of these structures to road users.

HR-247 Design Manual for Low Water Stream Crossings

A low water stream crossing (LWSC) is a stream crossing that periodically will be flooded and closed to traffic. This project was undertaken, to develop a manual for use in designing such structures. The resulting manual provides design guidelines for LWSCs. Rigid criteria for determining the applicability of a LWSC to a given site are not established since each site is unique in terms of physical, social, economic, and political factors. Because conditions vary from county to county, this manual does not provide a “cook-book” design procedure. Rather, engineering judgment must be applied to the guidelines presented in the manual.

2.8 Miscellaneous

HR-378 Metric Short Courses for the Office of Bridges and Structures

This metric short course was developed in response to a request from the Office of Bridges and Structures to assist in the training of engineers in the use of metric units of measure which will be required in all highway designs and construction after Sept. 30, 1996 (CFR Presidential Executive Order No. 12770).

The course notes which are contained in this report, were developed for an one-half day course. The course contains a brief review of metrication in the U.S., metric units, prefixes, symbols, basic conversions, etc. The unique part of the course is that it presents several typical bridge calculations (such as capacity of reinforced concrete compression members, strength of pilecaps, etc.) worked two ways: inch-pound units throughout with end conversion to metric and initial hard conversion to metric with metric units throughout. Comparisons of partial results and final results (obtained by working the problems the two ways) are made for each of the example problems.

HR-51 Use of Aluminum in Highway Bridges

Aluminum bridge structures are unique structures. They have been used as a viable alternative when fabricated structural steel was difficult to obtain. In recent years, there has been increased interest in new bridge materials, including aluminum. Its lightweight and corrosion resistance provides opportunities for its use in special situations. Research that addresses the behavior of full-scale aluminum members needs to be conducted to provide behavioral characteristics that can be incorporated into additional design recommendations for aluminum bridge structures and components.

In 1957, the Iowa State Highway Commission, with financial assistance from the aluminum industry, constructed a 220-ft long, four-span continuous, aluminum girder bridge to carry traffic on Clive Road (86th Street) over Interstate 80 near Des Moines, Iowa. The bridge, which was one of only nine existing aluminum girder bridges in the continental United States, was constructed with four, all-welded, aluminum girders. A composite, reinforced concrete deck served as the roadway surface. The bridge, which had performed successfully for about 35 years of service, was removed in the fall of 1993 to make way for an interchange at the same location.

A review of the inspection history of the bridge is included and shows that except for the need of a possible deck resurfacing, the bridge was in very good condition. Load tests of the bridge were conducted by driving an overloaded truck to preselected locations on the bridge deck and then monitoring the induced strains in the girder flanges and diaphragm webs of the bridge. Deflections were also measured in the northern end span. Fatigue testing of the aluminum girders that were removed from the end spans were conducted by applying constant-amplitude, cyclic loads. These tests established the fatigue strength of an existing, welded, flange-splice detail and added, welded, flange cover plates and web-stiffener plate details. The results from the experimental tests of this research will provide additional information regarding behavioral characteristics of full-scale, aluminum members and confirm that aluminum has the strength properties needed for highway bridge girders.

A comparison of the experimental girder strain and deflection test results and those results obtained from a finite element analysis of the bridge showed that the theoretical model accurately predicted the bridge response to applied wheel loads. The results of the load tests and theoretical analyses provided basic information on load distribution and confirmed that the new AASHTO LRFD Bridge Design Specifications provide load distribution criteria that were applicable to the original Clive Road aluminum girder – concrete deck bridge. Even though these specifications currently identify only precast concrete and steel girder bridges, the load distribution criteria appears also to be applicable for I-shaped aluminum girder bridges.

The nominal strength SN-curve obtained by this research essentially matched the SN-curve for Category E aluminum weldments given in the AASHTO-LRFD specifications. All of the Category E fatigue fractures that developed in the girder test specimens satisfied the allowable SN-relationship specified by the fatigue provisions of the Aluminum Association. The results from the experimental tests of this research have provided additional information regarding behavioral characteristics of full-size, aluminum members and have confirmed that aluminum has the strength properties needed for highway bridge girders.

2.9 Prestressed and Reinforced Concrete Beams and Bridges

TR-440 Field and Laboratory Evaluation of Precast Concrete Bridges

The objective of this project was to determine the load capacity of a particular type of deteriorating bridge – the precast concrete deck bridge – which is commonly found on Iowa's secondary roads. The number of these precast concrete structures requiring load postings and/or replacement can be significantly reduced if the deteriorated structures are found to have adequate load capacity or can be reliably evaluated.

Approximately 600 precast concrete deck bridges (PCDBs) exist in Iowa. A typical PCDB span is 19 to 36 ft long and consists of eight to ten simply supported precast panels. Bolts and either a pipe shear key or a grouted shear key are used to join adjacent panels. The panels resemble a steel channel in cross-section; the web is orientated horizontally and forms the roadway deck and the legs act as shallow beams. The primary longitudinal reinforcing steel bundled in each of the legs frequently corrodes and causes longitudinal cracks and spalling in the concrete.

The research team performed service load tests on four deteriorated PCDBs: two with shear keys in place and two without. Conventional strain gages were used to measure strains in both the steel and concrete, and transducers were used to measure vertical deflections. Based on the field results, it was determined that these bridges have sufficient lateral load distribution and adequate strength when shear keys are properly installed between adjacent panels. The measured lateral load distribution factors are larger than AASHTO values when shear keys were not installed. Since some of the reinforcement had hooks, deterioration of the reinforcement has a minimal affect on the service level performance of the bridges when there is minimal loss of cross-sectional area.

Laboratory tests were performed on the PCDB panels obtained from three bridge replacement projects. Twelve deteriorated panels were loaded to failure in a four point bending arrangement. Although the panels had significant deflections prior to failure, the experimental capacity of eleven panels exceeded the theoretical capacity. Experimental capacity of the twelfth panel, and extremely distressed panel, was only slightly below the theoretical capacity. Service tests and an ultimate strength test were performed on a laboratory bridge model consisting of four joined panels to determine the effect of various shear connection configurations. These data were used to validate a PCDB finite element model that can provide more accurate live load distribution factors for use in rating calculations. Finally, a strengthening system was developed and tested for use in situations where one or more panels of an existing PCDB need strengthening.

HR-353 Epoxy-Coated Strands in Composite Precast Prestressed Concrete Panels

Phase 1 research on epoxy-coated, prestressing strands in precast prestressed concrete (PC) panels has been published in two volumes: Volume 1 covers the literature review, survey results, descriptions of the test specimens, experimental tests, analytical models, discussions of the analytical and experimental results, the summary, conclusions, and recommendations; Volume 2 contains additional information in the form of summarized responses to the questionnaires, strands forces, geometry of the specimens, and concrete crack patterns that formed in the strands transfer length.

PC subdeck panels that act compositely with a cast-in-place reinforced concrete topping slab have been used for years in Iowa PC girder bridges. The durability of this alternate form of bridge deck construction has been questioned because the prestressing strands and welded wire fabric (WWF) that reinforce the panels are not epoxy coated. The primary objective of the research was to determine the feasibility of using grit-impregnated, epoxy-coated strands and epoxy-coated WWF in thin PC panels. Since larger bond stresses occur between a grit-impregnated, epoxy-coated strand and the surrounding concrete when compared to uncoated strands, a minimum thickness for a PC panel needed to be established so that concrete cracking would not occur when the panels were prestressed. Other objectives of the research were to determine the transfer and development length for 3/8-in. diameter, seven-wire, 270-ksi, low-relaxation, grit-impregnated, epoxy-coated (coated) and bare (uncoated) prestressing strands.

In the extensive laboratory study, 115 PC specimens were tested. The survey responses showed that the use of epoxy-coated strands in bridge components has been minimal. Only one design agency has used coated strands in PC subdeck panels. The amount of concrete side cover provided in the test specimens affected the uncoated-strands transfer and development lengths but apparently did not affect the coated-strands development length. The AASHTO Specification expression for strand development length significantly overestimated the measured strand development length for coated strands, substantially underestimated this length for uncoated strands with small amounts of concrete side cover, and provided a good prediction for this length for uncoated strands with adequate side cover and spacing.

HR-319 Lateral Road Resistance of Diaphragms in Prestressed Concrete Girder Bridges

Each year several prestressed concrete girder bridges in Iowa and other states are struck and damaged by vehicles with loads too high to pass under the bridge. Whether or not intermediate diaphragms play a significant role in reducing the effect of these unusual loading conditions has often been a topic of discussion. A study of the effects of the type and location of intermediate diaphragms in prestressed concrete girder bridges when the bridge girder flanges were subjected to various levels of vertical and horizontal loading was undertaken. The purpose of the research was to determine whether steel diaphragms of any conventional configuration can provide adequate protection to minimize the damage to prestressed concrete girders caused by lateral load, similar to the protection provided by the reinforced concrete intermediate diaphragms presently being used by the Iowa DOT.

The research program conducted and described in this report included the following: A comprehensive literature search and survey questionnaire were undertaken to define the state-of-the-art in the use of intermediate diaphragms in prestressed concrete girder bridges. A full scale, simple span, prestressed concrete girder bridge model, containing three beams was constructed and tested with several types of intermediate diaphragms located at the one-third points of the span or at the mid-span. Analytical studies involving a three-dimensional finite element analysis model were used to provide additional information on the behavior of the experimental bridge.

The performance of the bridge with no intermediate diaphragms was quite different than that with intermediate diaphragms in place. All intermediate diaphragms tested had some effect in distributing the loads to the slab and other girders, although some diaphragm types performed better than others. The research conducted has indicated that the replacement of the reinforced concrete intermediate diaphragms currently being used in Iowa with structural steel diaphragms may be possible.

2.10 Scour

HR-385 Stream Stabilization in Western Iowa: Structure Evaluation and Design Manual

Stream degradation is the action of deepening the streambed and widening the banks due to the increasing velocity of water flow. Degradation is pervasive in channeled streams found within the deep to moderately deep loess regions of the central United States. Of all the streams, however, the most severe and widespread entrenchment occurs in western Iowa streams that are tributaries to the Missouri River.

In September 1995, the Iowa DOT funded a study, HR-385 "Stream Stabilization in Western Iowa: Structure Evaluation and Design Manual," to provide an assessment of the effectiveness and costs of various stabilization structures in controlling erosion on channeled streams. A review of literature, a survey of professionals, field observations and an analysis of the data recorded on fifty-two selected structures led to the conclusions presented in the project's publication, *Design Manual, Streambed Degradation and Streambank Widening in Western Iowa*. Technical standards and specifications for the design and construction of stream channel stabilization structures are included in the manual.

HR-344 Potential Scour Assessments and Estimates of Maximum Scour at Selected Bridges in Iowa

The results of potential scour assessments at 130 bridges and estimates of maximum scour at 10 bridges in Iowa are presented. All of the bridges evaluated in the study are constructed bridges (not culverts) that are sites of active or discontinued streamflow gaging stations and peak stage measurement sites. The period of the study was from October 1991 to September 1994.

The potential scour assessments were made using a potential scour index developed by the U.S. Geological Survey for a study in western Tennessee. Higher values of the index suggest a greater likelihood of scour related problems occurring at a bridge. For the Iowa assessments,, the maximum value of the index was 24.5, the minimum value was 3, and the median value was 11.5. The two components of the potential scour index that affected the indices the most in this study were the bed material component, and bank erosion at the bridge. Because the potential-scour index represents conditions at a single moment in time, the usefulness of potential scour assessments is dependent upon regular assessments if the index is used to monitor potential scour conditions; however, few of the components of the index considered in this study are likely to change between assessments.

The estimates of maximum scour were made using scour equations recommended by the Federal Highway Administration. In this study, the long-term aggradation or degradation that occurred during the period of streamflow data collection at each site was evaluated. The stream-bed appeared to be stable at 6 of the 10 sites, was degrading at 3 sites, and was aggrading at 1 site. The estimates of maximum scour were made at most of the bridges using 100-year and 500-year flood discharges.

No pier-scour measurements were obtained in the study except for about 4 ft of local pier scour that was measured at the bridge over the Iowa River at Wapello, Iowa. However, the streambed was below the base of the pier footing, which is supported by piling, at the time the measurement was made.

Discharge measurement cross sections collected at two other bridges, which are not supported by piling, show the streambed between the piers to be lower than the bases of the piers. Additional investigation may be warranted at these sites to determine whether the streambed has been scoured below the bases at the upstream edges of the piers.

Although the abutment scour equation predicted deep scour holes at many of the sites, the only significant abutment scour that was measured was erosion of the embankment in the vicinity of one bridge abutment after a flood.

HR-307 Sediment Control in Bridge Water Ways

The objective of this study was to develop guidelines for use of the Iowa Vanes technique for sediment control in bridge waterways. Iowa Vanes are small flow training structures (foils) designed to modify the near-bed flow pattern and redistribute flow and sediment transport within the channel cross section. The structures are installed at an angle of attack of 15 - 25° with the flow, and their initial height is 0.2 – 0.5 times water depth at design stage. The vanes function by generating secondary circulation in the flow. The circulation alters magnitude and direction of the bed shear stress and causes a reduction in velocity and sediment transport in the vane controlled area. As a result, the river bed aggrades in the vane controlled area and degrades outside. This report summarizes the basic theory, describes results of laboratory and field tests, and presents the resulting design procedure.

Design graphs have been developed based on the theory. The graphs are entered with basic flow variables and desired bed topography. The output is vane layout and design. The procedure is illustrated with two numerical examples prepared with data that are typical for many rivers in Iowa and the Midwest.

HR-237 Shelby County Evaluation of Control Structures for Stabilizing Degrading Stream Channels

This investigation was undertaken to study the deepening and/or degrading problems in Western Iowa streams. The objectives were to document the effectiveness of existing control structures and to develop new control methods that were low in cost and adaptable for use in various county road situations. One of the initial designs – a soil cement structure – had to be abandoned since there were no bids received on the project. This structure was replaced with a proposal for a two row, concrete capped, sheet pile weir with a rip-rap lined stilling basin. The stilling basin was unique in that it had an engineering fabric and rip-rap lining with energy dissipating blocks.

HR-236 Evaluation of Control Structures for Stabilizing Degrading Stream Channels

Stream degradation due to steep stream gradients and large deposits of loess soil is a serious problem in western Iowa. One solution to this problem is to construct grade stabilization structures at critical points along the length of the stream. This project was initiated to study the effectiveness of such structures in preventing stream degradation. The construction and four-year performance of a gabion drop structure constructed along Keg Creek during the winter of 1982-83 is described in the final report to this investigation.

HR-208 Alternative Method of Stabilizing the Degrading Stream Channels in Western Iowa

Since the turn of the century, tributaries to the Missouri River in western Iowa have entrenched their channels to as much as six times their original depth. This channel degradation is accompanied by widening as the channel side slopes become unstable and landslides occur. The deepening and widening of these streams have endangered about 25% of the highway bridges in 13 counties.

Grade stabilization structures have been recommended as the most effective remedial measure for stream degradation. In western Iowa, within the last seven years, reinforced concrete grade stabilization structures have cost between $300,000 and $1,200,000. Recognizing that the high cost of these structures may be prohibitive in many situations, the Iowa DOT sponsored a study at ISU to find low-cost alternative structures. Analytical and laboratory work led to the conclusion that alternative construction materials such as gabions and soil-cement might result in more economical structures (Phase I). The ISU study also recommended that six experimental structures be built and their performance evaluated. Phase II involved the design of the demonstration structure, and Phase III included monitoring and evaluating their performance.

The gabion grade stabilization structure has shown satisfactory structural performance throughout the two-year observation period, with minimal differential settling and no evidence of side slope instability since construction was finished. It should be recognized that the maximum flow to date has been less than 15% of the design flow.

The major amount of sedimentation occurred during construction and is likely to extend to at least 5,500 ft upstream of the structure. A more optimistic estimate is that the depositional wedge will extend 6,500 ft upstream. In any event, the sedimentation effects of the structure will not submerge the knickpoint that exists upstream, so continued upstream erosion problems are likely upstream of the sedimentation area.

Gabions are deformable and may collapse into any scour hole that forms, thereby becoming somewhat self protecting. This downstream erosion is the result of inefficient energy dissipation by the stilling basin. An analysis of the cost of the gabion structure as compared with costs of four concrete structures included the size, drainage area, and design flow of each of the structures. This analysis suggests that the gabion structure cost about 20% of what an equivalent concrete structure would have cost.

3. SUMMARY OF QUESTIONNAIRES

Two separate questionnaires were sent to bridge owners as part of this investigation. One questionnaire (Iowa County Questionnaire) was developed specifically as part of this study and was sent to all Iowa County Engineers and is presented in Appendix A of this report. The second questionnaire (National Questionnaire) was developed as part of a national study performed by the authors of this report; that questionnaire is shown in Appendix B. Responses to both questionnaires are also summarized in Appendix A and B. That survey was sent to bridge owners at the national level and included a much more comprehensive questionnaire than needed for the study presented here. Therefore, only pertinent questions and responses from that national questionnaire are presented in this report.

3.1 Iowa County Questionnaire

As part of this study, a survey was developed and sent (with the assistance of the Iowa DOT) to all of the Iowa County Engineers. General discussions of specific survey responses are presented in the following paragraphs. As previously noted, a copy of the survey and summary of responses is presented in Appendix A.

Fifty-two Iowa counties responded to the survey. All but twelve (23%) counties responded that they have experience with bridge rehabilitation. Thirty-four (65%) said that they do use in-house crews for bridge replacement or rehabilitation. Those who use in-house crews to construct bridges construct different types from low water crossings and wood stringers with wood decks to RRFC and BISB decks. The most common response was steel stringers with wood decks.

Nineteen (37%) of the responding counties have developed replacement superstructures, or have plans for such structures. These plans included wood or steel stringers with wood decks and wearing surfaces. Keokuk County has plans for a county-designed bridge using steel stringers, steel abutments and a wood deck. Winnebago County is replacing bridges with culverts. Shelby and Audubon Counties bid private contractors to install prefabricated bridges for spans over 60 feet. Union County is planning to use the beam-in-slab design, while Adair, Marion and Tama Counties currently use BISB design.

When asked if they have experience in strengthening bridge elements, twenty-one responded that they have strengthened superstructures, and another twenty-one noted they have strengthened substructures (40%). The most common response was that they added members and piling to the structure. Other responses included replacement of stringers and abutments, rebuilding pile caps and strengthening existing members. Dallas County is adding cable supports to truss members. Washington County reported a technique for replacing rotted piles with concrete-filled PVC pipes.

The type of bridge system found to be most in need of strengthening procedures was the steel stringer type (FHWA No. 302). The steel girder plus floor beam system type (FHWA No. 303) was ranked least in need of strengthening. The question with the greatest response was “What problems are frequently encountered on low volume bridges?” The most frequent answer to the question was rotting of elements, whether deck, stringers or piling.

For superstructure elements, breaking of wood deck elements and stringers due to overload was a frequent response. Rotting of deck elements and corrosion of exterior I-beams was noted. Nine counties (17%) reported problems with steel stringers, either being too small, light, or too far apart. Narrow bridges were also reported as a problem by sixteen counties (31%).

In the substructure, backwall failures due to lateral pressure, rotted planks and sometimes stream erosion was indicated. Stream action was another common reply, causing erosion behind backwalls, around wings, and smashing logs or chunks into piling. Rotting piling was listed as a problem by every county that responded.

3.2 National Questionnaire

As part of a national study by the authors of this report, a survey was developed and circulated to various potential respondents including State DOT's, County and Local bridge owners and consultants involved with off-system bridge design and rehabilitation. The assistance of the National Association of County Engineers (NACE) was employed to disseminate the survey to all potentially interested parties. In all, several hundred surveys were distributed electronically via email.

Due to the scope of this national study (i.e. a general study of off-system bridge issues), the questionnaire was broader based and intended to acquire more general information than the questionnaire developed for dissemination to Iowa counties. The response to the national survey was very low; only 20 states and 70 local agencies from various areas of the nation responded.

State DOT's responding to the survey included:

- Arizona, Colorado, Wyoming, Montana, North Dakota, Minnesota, Texas, Arkansas, Mississippi, Louisiana, Tennessee, West Virginia, Pennsylvania, New York, New Jersey, Connecticut, Vermont, Maine, Hawaii.

Local agency responses were received from the following states:

- Washington, Oregon, North Dakota, Kansas, Iowa, Illinois, Alabama, Ohio, Maryland and New York.

For the local agency response, some of the respondents are from states with large off-system bridge populations; Illinois, Iowa, Kansas and Ohio are in the top quintile of locally owned bridges by count and by percentage. Considering the significance of these states, the local agency responses are from the states that have significant concerns with local bridge management issues.

General discussions of specific survey responses are presented in the following sections. As noted previously, a copy of the survey and summary of responses is presented in Appendix B of this report.

3.2.1 Superstructure Options for Low Volume Road Bridges

In several Bridge Type (BT) survey questions (questions BT-1 and BT-2), agency preferences regarding structure type were determined. The agencies were asked whether they would choose to or would be able to build any of the various bridge types and the reasons for their expressed preference. Examining the local agency construction capabilities, bridges of simple construction requiring minimal fieldwork and small equipment are the structures likely to be constructed using local agency personnel.

The most favored type of structure by both state and local agencies responding to the survey is a concrete box culvert. Other than culverts, State agencies prefer materials more common on higher volume roads as well, namely structural steel and prestressed concrete structures. Local agencies, on the other hand, have a greater preference for reinforced concrete and timber bridges and steel pipe arches and culverts.

Concerning the reasons for the expressed preference in structure types (question BT-6), initial cost, ease of construction, life cycle costs and durability were evenly ranked as the primary reasons for choosing a particular type of structure. Following closely behind these four reasons were material availability, familiarity and ease of design. A small percentage of owners indicated their choices reflect a lack of competing options.

Remoteness, ability to transport large pieces and proximity of steel or prestressed concrete fabricators were also frequently mentioned as important considerations. Several responses indicate a preference for prestressed concrete structures due to the inability of local steel fabricators to compete economically. It was also mentioned that the local availability of heavy lift equipment allows for the use of heavier concrete products limiting some of the advantages of other materials relative to weight and handling.

In general, the responses illustrate that local agencies are aware of the choices available and those that traditionally perform best in their areas. In some cases, lack of competitive options dictates their choice but it appears that lack of options alone is not a problem. Geography, geology and local agency and contractor experience generally dictate the choice of structure regardless of the merits of some other possible solutions.

Specific questions (questions BT-7 and BT-8) were asked in the survey to determine bridge deck and railing preferences. A discussion of the responses is presented in the following sections.

3.2.1.1 Bridge Decks

Because bridge decks are significant problem, the survey desired to determine the deck type preferences (question BT-7). The preferences indicate that CIP concrete decks are still the most preferred deck type by state and local agencies. After CIP concrete, the order of preference is full depth precast and partial depth precast with CIP toppings. The only difference in responses from state and local agencies is in their order of preference for steel grid or timber decks.

The most common bridge deck types continue to be those constructed of reinforced concrete but the survey response also indicates that concrete bridge deck maintenance is one of the most pervasive bridge maintenance problems. A large number of deck problems are associated with older concrete bridge decks that have, among other problems, insufficient cover, unprotected reinforcing steel, or both. Most of the deck deterioration problems stem from cracking that allows for the intrusion of moisture and salts which accelerates the corrosion process.

Some of the responses mention use of shrinkage compensating concrete in concrete deck construction. The Ohio Turnpike Commission (OTC) has extensive experience with the use of Type K Shrinkage Compensating Concrete (SCC) having used it in 520 bridge decks. Typically used on steel bridges, either composite or non-composite, decks have been constructed using SCC since 1984; the OTC is pleased with the performance of bridge decks constructed using SCC. It should be noted that it is not recommended to construct SCC bridge decks on bridges with concrete stringers due to the significant restraint provided by concrete stringers to shrinkage and thermal expansion and contraction.

Effective use of SCC in bridge decks requires some specific procedures for placing the concrete and curing. Highlights of differences in construction are that SCC typically requires placement by pumping, has a shorter working time and must be wet cured with moist burlap for seven continuous days.

SCC bridge decks may be an effective solution to bridge deck deterioration problems that plague essentially all bridge decks. They may be considered for use in new bridge decks for LVR bridges provided that the more involved construction procedures can be accommodated. The complexities in obtaining the appropriate concrete and accurately constructing and curing these decks may be a challenge for some agencies; the use of SCC decks should be considered with these limitations in mind.

3.2.1.2 Bridge Railings

Concerning the type of railings used by state and local agencies, the survey requested input regarding the percent of agencies that use traditional concrete barrier rails (i.e. Jersey barriers), timber railings, steel railings or no rails at all on off-system bridges (question BT-8). Of the States responding, all indicated mandatory compliance with the NCHRP 350 requirements. For State respondents, 89% indicated that concrete Jersey type rails were used, with a similar percentage of respondents, 83%, using post and beam steel rails. Approximately half of the States indicated that they have used timber railing systems.

For the county respondents, the most prevalent railing is the post and beam steel rail with approximately 3/4 of the responses indicating its use. Timber railings are used by approximately 42% of responding agencies.

It would appear from the railing survey responses that the concrete railing is much more common on state-owned off-system bridges than on those bridges under local control. There is a clear implication that local agencies consider the Jersey barrier (including rails such as the F-shape) either too expensive or simply “too much railing” for LVR bridges. This disparity is likely a reflection of traditional construction practices and state standard rails that are used system-wide regardless of traffic volume. Only half of the local agencies indicated that such railings are used at all and no information was collected as to how frequently they are used. One of the concerns expressed by a local agency respondent relative to tall/solid railing systems is their tendency to trap snow on bridge decks. This was considered a source of future maintenance problems. Other rail systems (non-solid) are not as prone to the debris/snow trapping problem.

A survey of State DOT web sites was conducted to determine to the extent possible the types of railings in use by the various agencies and if there were any special railings in use for LVR bridges. The state of New York has a significant number of approved railings and a variety of choices for LVR bridges; non-National Highway System (non-NHS) structures is their criteria for alternate railings. Examples of railings approved for use on non-NHS structures include thrie beam railings, double box beam curbless railings, timber railings for longitudinally laminated timber decks, timber rails on concrete decks, thrie beam transitions to timber rails and standards for upgrading numerous existing bridge railings.

The majority of information in the literature regarding bridge railings for LVR bridges comes from research sponsored by the USDA Forest Products Laboratory (FPL). There is minimal information in the literature outside of that sponsored by the FPL programs.

3.2.2 Prefabricated Bridge Systems

One of the areas of special interest in this questionnaire was the experience and opinions of bridge owners relative to the use of prefabricated and pre-engineered bridge systems or bridge components (questions BT-3 and BT-4). Due to the simplicity of construction of some of the prefabricated systems, the availability of “off-the-shelf” engineered bridges, and the lack of engineering and construction staffs in small agencies, it was anticipated that the use of prefabricated and pre-engineered bridge products would be looked upon favorably by local bridge owners.

Both state and local agencies indicate essentially the same ranking of reasons for the use of prefabricated and pre-engineered products (question BT-4). It is interesting that the reasons for a particular selection are essentially the same as those given for site-built bridges. Cost, ease of construction, traffic considerations and durability are the primary reasons for selecting manufactured products with the lack of staff or other options ranked last as for site-built bridges. It was anticipated that the lack of engineering staffs would be a greater “selling point” for these types of systems and that the savings garnered by not having to procure engineering services would have a positive economic benefit. The survey responses did not indicate such a perceived problem/benefit relative to engineering. There is also the possibility that such systems are simply underutilized and that the potential benefits are greater than currently realized.

Regarding the propensity of state and local agencies to consider the use of pre-engineered (and usually prefabricated) bridge components, they were queried in the Maintenance and Rehabilitation (MR) survey question MR-2. With regard to the consideration of pre-engineered decks, 39% of state and 43% of local agencies indicated that such systems are considered. A larger difference exists relative to pre-engineered bridge replacements. For such systems, 72% of state and 55% of local agencies consider the use of such products. No reasons are given for the responses to this survey question.

One could speculate that the reasons include lack of familiarity, lack of perceived need for rapid replacement, size or weight issues for installation, cost, or others.

Concerning the actual products used, a large number of responses were provided and various systems were discussed (question BT-3). A number of responses indicate the use of products produced by local maintenance crews while most others describe the use of locally produced commercial products. A short synopsis by material type is presented in the following sections.

3.2.2.1 Precast Concrete Products

Almost all of the survey responses indicate the use of a precast or precast / prestressed concrete product of one kind or another. The generic options include the use of I-beams, box beams, solid slabs, T-beams including double and quad stemmed members, pipe and precast box culverts (single and multi-cell), ASTM standard culverts, three sided open frame culvert structures, bridge deck panels and channel beams. One of the local agencies responded that they prefabricate their own reinforced concrete bridge beams, 3 ft wide by 16 in. deep for use in bridges with spans up to 31 ft. These beam-slabs are designed for HS 25 loading. The slabs are supported by county-built abutments and are constructed at a cost of 50 – 70% of that if completed by a contractor. In addition to the generic products described, various proprietary products were also mentioned such as the ConSpan and Bebo concrete arch systems as well as the HySpan concrete frame structure.

3.2.2.2 Prefabricated Metal Products

In contrast to the precast concrete products, where most of the options cited were for beam-type structures, the structural steel prefabricated bridge options generally fall into two categories: trusses and pipe / arch culverts. The majority of systems mentioned are trusses with U.S. Bridge, Acrow, Mabey, Continental Bridge, and Wheeler Consolidated listed as common fabricators of pre-engineered and prefabricated truss bridges. The other commonly cited solutions were corrugated metal (steel and aluminum) pipe culverts and structural plate arch structures such as those manufactured by Contech. In addition to these commonly cited solutions, corrugated steel decking and steel and aluminum grid decks were mentioned; decks manufactured by IKG Gruelich and Exodermic bridge decks were also specifically noted.

3.2.2.3 Timber Products

The responses noting use of timber structures came almost exclusively from local agencies. Various types of products were mentioned such as laminated timber decks, glulam timber panel bridges, nail laminated timber panels fabricated by local forces, glulam and dowel laminated bridge caps, railings and decks.

The project survey indicated that culverts are the most popular bridge for LVR applications for those sites where their use is appropriate. To help standardize the design, fabrication and construction of culvert structures, ASTM maintains a standard for precast box sections, ASTM C1433/1433M.

A U. S. Bridge company representative was interviewed to determine the company's impressions of the LVR bridge market from the perspective of a supplier. The intent of the interview was to learn their perspective on bridge replacements for LVR bridges. One of the issues was the difficulties that local agencies have in procuring replacement structures in a quick and efficient fashion to replace existing deficient bridges. Several examples were cited where a prefabricated truss was supplied and paid for with 100%

local funds and the total cost was less than the local agency 20% match for a "DOT compliant" bridge. The typical locally funded bridge might be a single lane bridge designed for HS 20 loading placed on existing substructures. A new structure that would be considered eligible for matching funds would be a much wider bridge on a new substructure and might have additional environmental and right-of-way costs that increase the total cost above what the local agency feels is appropriate for a particular location. The long process of getting a local bridge on the State Transportation Improvement Program and obligating the local match funds was cited as a significant impediment.

It was mentioned in the course of the interview that new trusses, or in some cases trusses removed from another location, rehabilitated and supplied to a new owner were substantially less expensive than other "conventional" replacement options. It was mentioned however that states are reluctant to accept bridges that are not among the typical bridges they construct.

Approximately 2/3 of the state and local agency survey responses indicated that stockpiling and reuse are performed in their jurisdictions. Many different components were listed as being recycled. These include small items such as bridge drains to the salvage of entire bridges, typically truss bridges. The most frequently cited recycled elements were bridge decks and deck panels, bridge and approach railings and superstructure beams (steel, concrete and timber). Several responses indicated that the recycled elements are intended for future use in other bridges while others use the salvaged elements for falsework and shoring on future jobs. There were several responses that indicated component recycling is not possible since disposed items from construction become the property of the contractors who use these materials on future projects.

3.2.2.4 Substructure Options for Low Volume Road Bridges

In general, this project focused on superstructure related issues. This is a reflection in large part on the amount of literature available with respect to bridge superstructures versus substructures. Concerning abutment type preferences (question BT-9), various options for abutment construction exist. The most common, and most preferred by both state and local agencies, is CIP concrete with states expressing a much stronger preference for this type of construction. However, other options such as soldier piles and lagging, sheet pile abutments and pile bents are used and considered more favorable by local than by state agencies. Again, CIP concrete structures were the most favored option by both state and local agencies, but local agencies prefer pile bent structures.

Regarding substructure types, there is much less of a spread from the most to least favorable substructure types for both abutments and piers for local agencies. Again, this may be a reflection of limited state experience with many of the proposed substructure types. Counties may have more experience with various substructure types and therefore a greater tendency to more evenly rank the alternative types.

Another aspect of bridge substructure construction is the consideration of bridge scour. Due to various problems with spread footings in scour prone locations, it is generally advised that except for the case of sound rock, footings should be founded on deep foundations. In order to ascertain the percent of agencies that construct footings on deep foundations when required due to scour potential, agencies were specifically queried about this issue in survey question BT – 10. The responses indicate that in 81% of the states and 73% of the local agencies, footings on deep foundations are required except when footings can be founded on non-erodible rock. Several inferences can be drawn from these percentages. One could infer that scour prone footings continue to be constructed (i.e. responses less than 100%) which is certainly undesirable from a safety and future maintenance perspective. The other inference is that spread footings are being constructed but that the footings are protected in some other way such as by placement of rip rap, sheeting, or stream bed paving and protection. Many of these protective measures, though used as maintenance solutions, are generally not considered to be permanent scour countermeasures as described in the FHWA Report *Bridge Scour and Stream Instability Countermeasures, Experience, Selection, and Design Guidance, Second Edition, HEC-23*.

3.2.3 Low Volume Road Bridge Design Aids

Survey question BT – 5 was included to obtain information on the use of engineering software or other types of design aids which are obviously closely related to the pre-engineered products. It was of interest to determine how bridges were being designed on LVRs and what resources the local agencies have available to expedite the engineering design process. Any tools that can shorten the design process save funds which can be used for bridge maintenance, rehabilitation, new construction, etc.

Various design aids are available and were cited in the survey responses. In general, the responses indicate the use of State standards as design guides plus several available from other sources. The other sources include standard plans for timber structures available from the United States Department of Agriculture Forest Products

Laboratory, the standard plans and software for short span steel bridge design developed by the American Iron and Steel Institute and the design manuals for precast concrete frame structures such as the Bebo and ConSpan systems. The lack of use of the many other design aids is probably an indication that they are an underutilized resource.

With regard to software programs, both those that are commercially available and those developed by state agencies, a number of programs exist and were cited. A listing of cited software is presented in Appendix D in the response to survey question BT – 5. A number of respondents reference the Pennsylvania Department of Transportation's software programs. The Pennsylvania DOT makes their numerous engineering design programs available to other government agencies free of charge. There are a wide array of programs covering various bridge superstructures, substructures and bridge rating functions. Information on these programs may be obtained from the Pennsylvania DOT.

3.2.3.1 Bridge Owners Survey Results: Rehabilitation/Strengthening Work Performed

The agencies surveyed indicated the types of rehabilitation/strengthening work they have performed and whether they used their own resources or contracted for the work. The responses are based on various types of rehabilitation/strengthening work performed and are summarized for state and local respondents. Each of the state and local responses are presented three ways; 1) the total percent response who've performed the work, 2) the percent of those respondents who've used their own resources for the work and 3) the percent of those respondents who've used contract forces for the work. Note that the responses related to use of own resources and use of contract resources are independent; therefore, the percent total is not additive for items 2 and 3. In other words, all of the responses represent only those who have used the various means to accomplish the work.

At the state level, most states performed a high percentage of rehabilitation/ strengthening on all of the bridge components shown, indicating that main member replacement and strengthening are at the lower end of their efforts. The majority of the work is performed by contract forces, rather than using their own resources, except for deck overlay where an equal percentage of work is performed by both. Both state and local agencies have completed large numbers of strengthening projects.

Local agency responses were similar to those from the state in terms of the total percent of effort in the rehabilitation of various bridge components. One exception was that the local agencies performed significantly less work related to deck overlays, deck joint replacements and main member replacement than did the state agencies. The local agencies also tended to use more of their own resources than did the states. In particular, the local agencies relied more on their own resources than on contract resources for deck replacement, railing replacement, main member replacement, substructure replacement and strengthening.

4. MAINTENANCE/REHABILITATION/STRENGTHENING

4.1 Introduction

This chapter focuses on the maintenance of bridges to ensure adequate load capacity for the safety of the traveling public. The term maintenance is often used generically to describe bridge repair activities, which frequently raises the issues of rehabilitation and/or strengthening. In this chapter, the terms maintenance, structural rehabilitation, stiffening, and structural strengthening will be used. The following definitions are provided to clarify the terms as used in this report.

Maintenance: The technical aspect of the upkeep of the bridges; it is preventative in nature. Maintenance is the work required to keep a bridge in its present condition and to control potential future deterioration.

Rehabilitation: The process of restoring the bridge to its original service level.

Repair: The technical aspect of rehabilitation; action taken to correct damage or deterioration on a structure or element to restore it to its original condition.

Stiffening: Any technique that improves the in-service performance of an existing structure and thereby reduces inadequacies in serviceability (such as excessive deflections, excessive cracking, or unacceptable vibrations).

Strengthening: The increase of the load-carrying capacity of an existing structure by providing the structure with a service level higher than the structure originally had (sometimes referred to as upgrading).

4.2 Strengthening/Rehabilitation

If through rating, it is determined that a bridge is inadequate to carry design live loads, it is possible to strengthen the bridge. The live-load capacity of various types of bridges can be increased by using different methods, such as (1) adding members, (2) adding supports, (3) reducing dead load, (4) providing continuity, (5) providing composite action, (6) applying external post-tensioning, (7) increasing member cross section, (8) modifying load paths, and (9) adding lateral supports or stiffeners. Some methods have been widely used, but others are relatively new and have not been fully developed.

All strengthening procedures presented in this section apply to the superstructure of bridges. Although bridge span length is not a limiting factor in the various strengthening procedures presented, the majority of the techniques apply to short-span to medium-span bridges. The techniques used for strengthening, stiffening, and repairing bridges tend to be interrelated so that, for example, the stiffening of a structural member of a bridge will normally result in its being strengthened also.

In recent years, the FHWA and National Cooperative Highway Research Program (NCHRP) have sponsored several studies on bridge repair, rehabilitation, and retrofitting. Since some of these procedures also increase the strength of a given bridge, the final reports on these investigations are excellent references. These references, plus the strengthening guidelines presented in this chapter, will provide information an engineer can use to resolve the majority of bridge strengthening problems. The FHWA and NCHRP final reports related to this investigation are References: Dorton and Reel, 1997; Klaiber, et al., 1987; Sprinkle, 1985; Shanafelt and Horn, 1985; Shannafelt and Horn, 1984; Applied Technology Council, 1983; Sabnis, 1983; Univ. of Virginia Civil Engineering Department et al., 1980; Mishler and Leis, 1981; Shanafelt and Horn, 1980; Berger, 1978; Fisher et al., 1979.

Four of these references, Dortan and Reel, 1997; Klaiber et al., 1987; University of Virginia Civil Engineering Department et al, 1980; Berger, 1978, are of specific interest in strengthening work. Although not discussed in this chapter, the live load capacity of a given bridge can often be evaluated more accurately by using more refined analysis procedures. If normal analytical methods indicate strengthening is required, frequently more sophisticated analytical methods (such as finite element analysis) may result in increased live load capacities and thus eliminate the need to strengthen or significantly decrease the amount of strengthening required.

Data obtained in the load testing of bridges frequently reveals live load capacities considerably larger than what would be determined using analytical procedures. Load testing of bridges makes it possible to take into account several contributions (such as end restraint in simple spans, structural contributions of guardrails, etc.) that cannot be included analytically. In the past few years, several states and counties have started using load testing to establish the live load capacities of their bridges. Although most states also have some type of Bridge Management System (BMS), to the authors' knowledge, very few states are using their BMS to make bridge strengthening decisions. At the present time, there are not sufficient base line data (first cost, life cycle costs, cost of various strengthening procedures, etc.) to make strengthening/replacement decisions.

Examination of National Bridge Inventory (NBI) bridge records indicates that the bridge types with greatest potential for strengthening are steel stringer, timber stringer, and steel through-truss. If rehabilitation and strengthening cannot be used to extend their useful lives, many of these bridges will require replacement in the near future. Other bridge types for which there also is potential for strengthening are concrete slab, concrete tee, concrete stringer, steel-girder floor-beam, and concrete-deck arch. In this section, information is provided on the more commonly used strengthening procedures as well as a few of the new procedures that are currently being developed.

4.3 Bridge Rehabilitation

In conjunction with their publication concerning bridge maintenance on local roads, the National Association of County Engineers (NACE 1995) publishes *Bridge Rehabilitation on Local Roads*, a guide which presents effective bridge rehabilitation methods applicable to bridge types commonly found on the local road system. This guide, which may be obtained for a nominal fee, is divided into sections concerning planning activities, bridge decks and railings, trusses, beams and girders, expansion joints and bearings, substructures, waterways, and finally support activities. The guide has been developed in response to the large number of bridges considered deficient on the local road system and considers the budgetary constraints counties face when addressing bridge rehabilitation needs.

Bridge rehabilitation is only part of a broad bridge administration policy that includes bridge inspection, maintenance, rehabilitation and long range planning. Bridge rehabilitation generally begins with a routine bridge inspection which catalogs existing conditions. Following inspection, an analysis of the inspection data identifies the required rehabilitation work and considers the costs of the needed improvements for purposes of programming. A consideration of the impacts of rehabilitation should also be made with regard to required permits, needs for detours, engineering and contractual documents, and work zone safety. Due to the very extensive nature of the NACE rehabilitation guide, only brief descriptions of the key items will be presented in this report.

4.3.1 Bridge Deck Rehabilitation

Bridge decks are addressed with respect to proper inspection and appropriate means of rehabilitation. The most common deterioration in bridge structures is in the bridge deck. In addition to being most common, it is the most objectionable to the traveling public due to its impact on ride quality; for these reasons, it is one of the more important rehabilitation activities. Although timber and steel grid decks are mentioned, the majority of coverage is on concrete decks since they are the most prevalent.

4.3.1.1 Concrete Decks

For concrete decks, chloride contamination is the most common cause of deterioration. Once chlorides reach the reinforcing steel, especially in older non-coated bridge decks, the reinforcing steel begins to corrode resulting in spalling, more extensive cracking, and an acceleration of deck deterioration. To determine the appropriate deck rehabilitation, a deck condition survey must be performed. The condition survey should determine the amount of bridge deck spalling and surface deterioration while a more complete assessment would also involve a determination of bridge deck delamination, half-cell potential, and chloride content. Following the survey of the bridge deck, it is rated on a scale of 0 to 9 for purposes of recording on an SI&A sheet. Bridges with decks rated less than 4 (extensive deterioration) are classified as structurally deficient.

In order to prevent bridge deck deterioration, or at least delay its onset since total prevention is unlikely, various treatments may be used. These include the application of penetrating concrete sealers and the use of waterproofing membranes. Neither of these solutions is permanent since they themselves are subject to wear and deterioration.

Additionally, they are only appropriate for bridge decks with minor deterioration and no indications of chloride penetration.

For more extensive deterioration, additional rehabilitation measures are required. When deck deterioration is isolated to discrete areas, deck patching is frequently warranted with either partial or full-depth patches being used as appropriate. However, in some instances, deck patching may actually compound the problem. Patching typically only repairs the deteriorated concrete and may be ineffective in arresting reinforcing steel deterioration or remove chlorides from surrounding concrete. For decks where the riding surface is more uniformly deteriorated or where additional concrete cover is desired, deck overlays may be used. These overlays are typically constructed with either dense Portland cement concrete or latex-modified concrete. In addition to normal deck overlays that bond directly to the old deck, overlays may also be constructed with an intermediate membrane between the old and new deck courses. Prior to application of the membrane, delaminations, active corrosion areas and chloride contaminated areas should be repaired so that they are not trapped under the membrane and wearing course.

In the case of extensive deck deterioration, the most feasible and prudent alternative may be a complete deck replacement. Various options exist for replacing deteriorated concrete decks with new concrete decks. In some cases, a deteriorated noncomposite deck and can be replaced with a new composite deck. The new deck may be placed as either a cast-in-place deck, the traditional method, or with precast concrete panels with blockouts to allow for grouting to the shear connectors. Various schematic options for precast deck panel replacements are presented in the NACE guide.

4.3.1.2 Timber Decks

Though not as common as concrete decks, there are still a large number of timber decks on LVR bridges. They are a viable deck system especially when the current high quality pressure treated lumbers (available in glue laminated, spike laminated or stress-laminated panels) are used in bridge construction. Timber bridge decks may be used on short span timber bridges or as decks on short to medium span steel stringer bridges. The United States Department of Agriculture's Forest Products Laboratory publishes an extensive amount of literature on the design, construction and performance of timber bridge decks. The literature covers species selection, design capacities, construction details and includes case histories of in-place performance.

4.3.1.3 Steel Decks

In addition to timber decks, steel grid decks whether composed of open steel decks or partially filled grids are frequently used. In Ohio, galvanizing the steel grid decks has been shown to increase the deck life by 15 or more years. The open steel grids come in various gages and depths providing various spanning capabilities; however, they have the downside of exposing the bridge superstructure directly to the effects of snow, rain and other contaminants. They are also susceptible to fatigue. A refinement of the open deck system, that still results in a lightweight deck system, is the partially filled deck. This steel grid is filled with several inches of concrete; the system is a more durable option for deck replacements in which both durability and dead load reduction are desired. Finally, a low cost non-composite type of bridge deck is one that employs stay-in-place metal deck forms similar to those used to form concrete decks; however, in this application the forms are filled with bituminous material. Due to the porous nature of the bituminous paving material, corrosion of the deck pans is a concern and should be considered and monitored accordingly.

4.3.2 Truss Rehabilitation

Truss bridges are still present on numerous LVRs. Nationally, according to NBI data published in August, 2000, approximately 17,000 trusses are in service. Of these 17,000 trusses, 801 of these are deck trusses, the remaining 16,735 are thru-trusses. For the deck trusses, 40% are considered structurally deficient and an additional 20% are functionally obsolete. The statistics for thru-trusses indicate that almost 64% of thru-trusses are structurally deficient and an additional 17% are functionally obsolete. These bridges are generally old, narrow, composed of riveted steel and carry low volumes of traffic though not necessarily low loads. Due to the large numbers of these bridges on LVRs, effective means of rehabilitation is still of interest. An additional complication with rehabilitating of old truss bridges is their potential historic status.

When evaluating truss bridges, one needs to remember the fact they are generally considered to be nonredundant structures due to there only being two main trusses. Many of the truss members are fracture critical indicating that failure of any of these key members would theoretically result in the collapse of the bridge. Old trusses may be composed of multiple eyebars and pin connected members, especially vulnerable to deterioration, fatigue and fracture. These members can be difficult to inspect and repair. A careful consideration of the existing strength as well as material sampling is important for an accurate evaluation of some older trusses.

4.3.2.1 Floor System

The floor system of a truss is a combination of transverse floorbeams, supported at truss panel point locations, and the longitudinal stringers that typically rest on the top flanges of the floorbeams and support the bridge deck. Deterioration in floor system, is usually caused by the leakage of water through the decks and onto the members of the floor system. Additionally, the floorbeams/truss connections and end floorbeams that are under expansion joints are particularly vulnerable due to their more direct exposure to the elements. In rehabilitation, replacement of deteriorated stringers and floorbeams, failed decks, and drainage systems is a common solution. If the deterioration is only localized and deemed repairable, the addition of cover plates to stringers and/or floor beams is a common solution.

4.3.2.2 Truss Elements

Deteriorated truss members (web verticals, diagonal web members, and top and bottom chords) as well as truss connections may be strengthened with the addition of cover plate material, by the addition of longitudinal bars to the cross section, and by post-tension strengthening of the deficient tension members. Members that have been damaged by vehicle impact may be heat straightened. Through-trusses with deficient bottom chord members may be strengthened either by addition of cover plates or supplemental reinforcing material along the bottom chords; they can also be strengthened by the application of an external truss such as a single post king post truss or multiple post queen post trusses. The NACE guide gives various illustrations and recommendations for the repair of damaged truss members.

4.3.3 Steel Beams and Girders

Steel beam and girder bridges are very common, representing about 1/3 of the Nation’s bridge population; they are subject to various forms of deterioration including overstress, damage, corrosion, and fatigue and fracture. They may be repaired by various methods including the addition of bolted or welded cover plates or supplemental members. Field welding should only be performed with care; it should never be performed on fracture critical members, or members where fatigue is a concern. Beam end corrosion under failed expansion joint drainage systems is a problem that typically requires remedial action due to the local failure of the beams.

A procedure for rehabilitating steel bridges that has the possibility of reducing future maintenance costs and improving the bridge rating is the conversion of multiple simple spans into a continuous bridge. This is accomplished by removing the expansion joints at piers and splicing the stringer ends with web and flange splice plates and adding reinforcement to the created a negative moment region in the deck. This procedure results in the bridge behaving as simply supported for non-composite dead load and continuous for live load. Another method for increasing the capacity of deficient steel bridges is through addition of an intermediate bent which shortens the span and thus reduces live load stresses. This modification does create negative moment stresses in the vicinity of the added bent, which must be appropriately considered. Again, similar to truss bridge strengthening, steel stringers may be rehabilitated by external post tensioning or by the addition of external king post trusses.

4.3.4 Concrete Beams

The most common forms of deterioration in concrete structures are those related to loss of inadequate cover which then results in deterioration of the reinforcing steel. Hairline cracks are common in reinforced concrete beams, however, there should be no evidence of cracking in prestressed concrete beams. Cracking in prestressed concrete may be indicative of a more serious problem such as loss of prestressing and should be investigated more thoroughly. Similar to steel beams, concrete beams are especially subject to deterioration at the beam ends where they are subjected to run off from expansion joints and failed drainage systems. Deterioration will result in steel deterioration and loss of section of the beam ends. A possible solution includes the removal of damaged concrete, adding supplemental reinforcement in the beam by epoxy doweling, and then restoring the section with a durable concrete or grout. Another solution for failed beam ends is to extend the supports out into the span. This might involve the building of corbels on the front face of an abutment to support a still sound portion of the beam or could involve construction of similar brackets and corbels on the faces of bridge pier caps. The NACE guide has several schematic illustrations of pier and abutment modifications for support relocation.

4.3.5 Timber Beams

Timber beam rehabilitation is addressed in the NACE; while timber bridges represent only 6% of the Nation’s bridge population the vast majority of these structures are located on LVRs. In Iowa, there are approximately 3200 timber bridges which is close to 13% of the state bridge population. According to the NACE guide, timber bridge beams may be rehabilitated as long as their original load carrying capacity has not been reduced by more than 50%. Rehabilitation options include partial replacement of damaged sections by splicing in new material. Timber stringers in multi-beam bridges are typically damaged on their top surface due to moisture penetration; this also results in loosening of the connection between the deck and stringers. Engineers are strongly cautioned against rotating damaged stringers so that the “good side” is at the top. By placing the moisture and nail damaged portion of the stringer on the bottom, the tension zone, the capacity of the beam has been significantly compromised. More advisable techniques include the addition of timber or steel side helper beams, the addition of a bottom steel cover plates, and the epoxy repair of damaged timbers. If only a few defective stringers in a span need to be strengthened or replaced, this can be done from above by removing the deck in the vicinity of the deflective stringers. As an alternative, stringers may be replaced from below by slightly jacking up the deck in the vicinity of the damaged stringer, removing the damaged stringer, followed by adding the new stringers. The new stringers should have the corners slightly beveled to facilitate their installation under the existing deck.

4.3.6 Joint and Bearing Rehabilitation

4.3.6.1 Bridge Joints

As mentioned numerous times, leakage and failure of joints is detrimental to the underlying superstructure. However, failure of the joints themselves is problematic. Common problems with expansion joints include the presence of debris which restricts expansion and contraction that could cause damage to abutment backwalls and bridge decks. In short bridges, it is not uncommon to have small open joints at the expansion locations with armored edges to protect the exposed concrete. These angles if damaged can be replaced by removing a short section of concrete, attaching a bent angle welded to both the longitudinal reinforcing and the armor angle, and re-casting the deck and approach concrete in the vicinity of the repair. An effective procedure for repairing sliding plate joints, which typically leak or become frozen from deterioration, is to cut-off the sliding plate at the face of the angle and fill in the gap with a compression seal or strip seal expansion joint.

4.3.6.2 Bridge Bearings

Bridge bearings have the dual function of providing vertical support as well as providing for longitudinal movement or restraint. Older bearings, such as rollers, pintle bearings, rockers, pin bearings, fixed shoes, roller nests, etc., are all composed of multiple steel pieces whose strength and movement must be accessed. Rehabilitation of bridge bearings typically involves providing some means for lifting the bridge so that bearings can be replaced, repaired, realigned or otherwise rehabilitated. Bridge bearing rehabilitation may also involve bearing seat repairs. Older bridge bearings that rely on multiple parts moving relative to each other need to be properly cleaned, lubricated and aligned to function properly. In many cases, especially in short and medium span bridges where neither the vertical loads nor expected movements are significant, problematic expansion bearings may be effectively replaced with low cost and low maintenance elastomeric bearings.

4.3.7 Bridge Substructure Rehabilitation

Bridge substructures are critical elements of a bridge in that they provide support for the entire structure. Although they generally require less maintenance than the bridge superstructure components, their repair and rehabilitation when required, may be more expensive, time consuming and much more difficult than repairing bridge superstructure elements.

4.3.7.1 Abutments and Backwalls

Bridge abutments and backwalls serve the purpose of supporting the end of the bridge, retaining the approach fills and resisting the effects of longitudinal and transverse forces imposed on the bridge superstructure. Typical surface damage in reinforced concrete backwalls such as spalls and cracking can be effectively repaired as long as they have stabilized. Typical surface repairs include concrete patching and epoxy or latex crack injection. In the case of more extensive deterioration, a concrete jacket may be placed over the entire height of the abutment; this concrete should be doweled to the existing structure and placed with the aid of a bonding agent to an intentionally roughed surface. For timber abutments, routine member replacement is the likely rehabilitation option since timbers, even treated, are prone to decay. Selective replacement of damaged members and the addition of helper elements is frequently employed.

4.3.7.2 Piers

Bridge piers are used to support the interior ends of bridge spans for multi-span structures. Bridge pier repairs can be at the cap level due to leaking joints, can be to the columns or wall stems due to vehicular impact, or to the foundation due to loss of support such as from undermining. Footing scour results in the loss of soil or erodible rock from underneath a bridge pier. This results in either a loss of foundation capacity or the creation of potentially problematic unbraced lengths for pile foundations. Solutions include placement of tremie concrete seals under the pier footing or hand placed of grout bags or bags of cement. These bags constitute the formwork for the grout underpinning of the bridge footing. Following underpinning, the footing needs to be protected with stone rip rap.

4.3.7.3 Piles

There are cases in which due to environmental effects bridge piles themselves require rehabilitation. For steel or timber piles with lost capacity, it may be possible to splice over the damaged area with retrofit material to replace the lost capacity, or in some instances, remove the length of pile damaged and replace it with a similar material. Bearing piles that also form part of a pile bent are frequently damaged and also may require rehabilitation or replacement. In replacement situations, typically the new piles are driven through an opening in the deck of the bridge. The new piles can then be integrated into the bent as a whole thus restoring lost capacity.

4.3.7.4 Waterway Rehabilitation

Rehabilitation of waterways in the vicinity of bridges is sometimes required to eliminate bridge maintenance problems. In addition to bridge scour problems which are common for waterway crossings, there are also the challenges of meandering stream channels that may encroach upon previously “protected” foundations and the problems associated with general streambed lowering. All efforts should be taken to maximize the hydraulic opening in the vicinity of a bridge. Additionally, the stream cross section should not change substantially in the vicinity of the bridge which could lead to changes in the flow characteristics. If it is determined that stone rip rap is required, predicted velocities from a hydraulic analysis should be completed to appropriately size the stone. The stone should be placed beginning below the level of the natural streambed and extend up the sides of the foundation and in certain situations, cover it for the desired protection. An expansion of this protection scheme is the use of a stone blanket or apron across a major portion of the stream and in some cases up the banks. This is used where there is general degradation of the stream cross-section as opposed to isolated scour. If the source of the problem is a poor stream alignment, changes in the flow pattern may be required through use of hydraulic structures such as spur dikes, wing dikes, check dams, jack fields and flow retarders. These measures, individually or in combination, are used to slow the flow near stream banks and hence reduce the erosion, or they are used to reduce the flow velocity in generally thus reducing the potential for foundation scour.

4.4 Bridge Strengthening

Several of the topics presented in this section were briefly previously reviewed in the NACE publication, *Bridge Rehabilitation on Local Roads* (NACE 1995). In this section, numerous strengthening procedures as well as additional details on the procedures previously noted are presented.

4.4.1 Lightweight Decks

One of the more fundamental approaches to increase the live-load capacity of a bridge is to reduce its dead load. Significant reductions in dead load can be obtained by removing an existing heavy concrete deck and replacing it with a lighter weight deck. In some cases, further reduction in dead load can be obtained by replacing the existing guardrail system with a lighter weight guardrail. The concept of strengthening by dead-load reduction has been used primarily on steel structures, including the following types of bridges: steel stringer and multibeam, steel girder and floor beam, steel truss, steel arch, and steel suspension bridges; however, this technique could also be used on bridges constructed of other materials.

Lightweight deck replacement is a feasible strengthening technique for bridges with structurally inadequate, steel stringers or floor beams. If, however, the existing deck is not in need of replacement or extensive repair, lightweight deck replacement is not economical. Lightweight deck replacement can be used conveniently in conjunction with other strengthening techniques. After an existing deck has been removed, structural members can readily be strengthened, added, or replaced. Composite action, which is possible with some types of lightweight decks, can further increase the live-load carrying capacity of a deficient bridge.

4.4.1.1 Open-Grid Steel Decks

Open-grid steel decks are lightweight, typically weighing 15 to 25 psf for spans up to 5 ft. Heavier decks, capable of spanning up to 9 ft, are also available; the percent increase in live-load capacity is maximized with the use of an open-grid steel deck. Open-grid decks are often perceived unfavorably by the general public because of the poor skid resistance, poor riding quality and increased tire noise.

4.4.1.2 Concrete-Filled Steel Grid Decks

Concrete-filled steel grid decks weigh substantially more, but have several advantages over the open-grid steel decks, including increased strength, improved skid resistance, and better riding quality. The steel grids can be either half or completely filled with concrete. A 5 in. thick, half-filled steel grid weighs 46 to 51 psf, less than half the weight of a reinforced concrete deck of comparable strength. Typical weights for 5 in. thick steel grid decks, filled to full depth with concrete, range from 76 to 81 psf. Reduction in the dead weight resulting from concrete-filled steel grid deck replacement alone only slightly improves the live-load capacity; however, the capacity can be further improved by providing the previously noted composite action between the deck and stringers.

4.4.1.3 Exodermic Deck

Exodermic deck is a prefabricated, proprietary modular deck system that has been marketed by a major steel-grid-deck manufacturer. The first application of Exodermic deck was in 1984 in New Jersey (DePhillips, 1985). The bridge deck system consists of a thin upper layer (3 in. minimum) of prefabricated concrete joined to a lower layer of steel grating. The deck weighs between 40 and 60 psf and is capable of spanning up to 16 ft.

Exodermic decks and half-filled steel grid decks have the highest percent increase in live-load capacity among the lightweight decks with concrete surfaces, and can be quickly installed as a prefabricated modular deck system. Because the panels are fabricated in a controlled environment, quality control is easily maintained.

4.4.1.4 Laminated Timber Deck

Laminated timber decks consist of vertically laminated 2-in. (nominal) dimension lumber. The laminates are bonded together with a structural adhesive to form panels that are approximately 48 in. wide. The panels are typically oriented transverse to the supporting structure of the bridge. In the field, adjacent panels are secured to each other with steel dowels or stiffener beams to provide continuity between the panels and to allow for load transfer.

A steel-wood composite deck for longitudinally oriented laminates was developed in 1985 by Bakht and Tharmabala. Individual laminates are transversely post-tensioned in the manner developed by Csagoly and Taylor (1980). The use of shear connectors provides partial composite action between the deck and stringers. Because the deck is placed longitudinally, diaphragms mounted flush with the stringers may be required for support. Design of this type of timber deck is presented by Taylor et al (1982), and the Canadian Ministry of Transportation and Communications (1983a and 1983b). The laminated timber decks used for lightweight deck replacement typically range in depth from 3 1/8 to 6 3/4 in. and from 10.4 to 22.5 psf in weight. A bituminous wearing surface is recommended.

Wood is a replenishable resource that offers several advantages: ease of fabrication and erection, high strength to weight ratio, and immunity to deicing chemicals. The most common problem associated with wood as a structural material is its susceptibility to decay, however, with the use of modern preservative pressure treatments, the expected service life of timber decks can be extended to 50 years or more.

4.4.1.5 Lightweight Concrete Deck

Structural lightweight concrete, concrete with a unit weight of 115 pcf or less, can be used to strengthen steel bridges that have normal-weight, noncomposite concrete decks. Special design considerations are necessary for lightweight concrete. Its modulus of elasticity and shear strength are less than that of normal-weight concrete, whereas its creep effects are greater (Mackie, 1985). The durability of lightweight concrete has been a problem in some applications.

Lightweight concrete for deck replacement can be either cast in place or installed in the form of precast panels. A cast-in-place lightweight concrete deck can easily be made to act compositely with the stringers. Lightweight precast panels, fabricated with either mild steel reinforcement or transverse prestressing, have been used in deck replacement projects to help to minimize erection time and resulting interruptions to traffic. Precast panels require careful installation to prevent water leakage and cracking at the panel joints.

4.4.2 Composite Action

Modification of an existing stringer and deck system to a composite system is a common method of increasing the flexural strength of a bridge. The composite action of the stringer and deck not only reduces the live-load stresses but also reduces undesirable deflections and vibrations as a result of the increase in the flexural stiffness from the stringer and deck acting together. This procedure can also be used on bridges that only have partial composite action, because the shear connectors originally provided are inadequate to support today's live loads.

Although numerous devices have been used to provide the required horizontal shear resistance, the most common connection used today is the welded stud. Composite action can effectively be developed between steel stringers and various deck materials, such as normal-weight reinforced concrete (precast or cast-in-place), lightweight reinforced concrete (precast or cast-in-place), laminated timber, and concrete-filled steel grids. Because steel stringers are normally used for support of all the previously noted decks, they are the only type of superstructure reviewed. The condition of the deck determines how one can obtain composite action between the stringers and an existing concrete deck.

If the deck is in good condition, one method of obtaining composite action is to use high strength bolts as shear connectors. By coring through the concrete deck, drilling through the top flange of the steel beam, inserting and double nutting a high-strength bolt in the flange hole, and finally filling the deck hole with a non-shrink grout – one can obtain the desired composite action. In Iowa DOT project HR-238, this type of connection was shown to have more strength than welded shear studs (Klaiber et al., 1983). If the deck is badly deteriorated, composite action is obtained by removing the existing deck, adding appropriate shear connectors to the stringers, and recasting the deck. An easy way of obtaining composite action in new construction or the cases where an existing deck has been removed is to use the Alternative Shear Connector (ASC) which was developed and tested in Iowa DOT project TR-410 (Klaiber et al., 2000). Since its development, the ASC has been used in several demonstration bridges which are part of current research projects.

To reduce construction time, precast concrete panels can be used. The panels are made composite by positioning holes formed in the precast concrete directly over the structural steel. Welded studs are then attached through the preformed holes. Composite action is obtained by filling the holes, as well as the gaps between the panels and steel stringers, with fast-curing concrete.

If the concrete deck does not need replacing, composite action can be obtained by coring through the existing concrete deck to the steel superstructure. Appropriate shear connectors are placed in the holes; the desired composite action is then obtained by filling the holes with nonshrink grout.

4.4.3 Improving the Strength of Bridge Members

4.4.3.1 Addition of Steel Cover Plates on Steel Stringer Bridges

One of the most common procedures used to strengthen existing bridges is the addition of steel cover plates to existing members. Steel cover plates, angles, or other sections may be attached to the beams by means of bolting or welding. The additional steel is normally attached to the flanges of existing sections as a means of increasing the section modulus, thereby increasing the flexural capacity of the member. In most cases, the member is jacked up during the strengthening process, relieving dead-load stresses on the existing member. The new cover plate section is then able to resist both live-load and dead-load stresses when the jacks are removed which ensures that less steel will be required in the cover plates. If the bridge is not jacked up, the cover plate will carry only live-load stresses, and more steel will be required.

The most commonly reported problem encountered with the addition of steel cover plates is fatigue cracking at the toe of the welds at the ends of the cover plates. In a study by Wattar et al. (1985), it was suggested that bolting be used at the cover plate ends. Tests showed that bolting the ends raises the fatigue category of the member and also results in material savings by allowing the plates to be cut off at the theoretical cutoff points.

Materials other than flange cover plates may be added to stringer flanges for strengthening. For example, the Iowa DOT prefers to attach angles to the webs of steel I-beam bridges (either simple supported or continuous spans) with high-strength bolts as a means to reducing flexural live-load stresses in the beams. In some instances the angles are attached only near the bottom flange. Because the angles are bolted on, problems of fatigue cracking that could occur with welding are eliminated.

4.4.3.2 Addition of Steel Shapes on Reinforced Concrete Bridges

One method of increasing flexural capacity of a reinforced concrete beam is to attach steel cover plates or other steel shapes to the beam's tension face. The plates or shapes are normally attached by bolting, keying, or doweling to develop continuity between the old beam and new material. If the beam is also inadequate in shear, combinations of straps and cover plates may be added to improve both shear and flexural capacity. Because a large percentage of the load in most concrete structures is dead load, for cover plating to be most effective, the structure should be jacked prior to cover plating to reduce the member's dead-load stresses. The addition of steel cover plates may also require the addition of concrete to the compression region of the member.

A successful method of strengthening reinforced concrete beams has involved the attachment of a steel channel to the stem of a beam. Taylor (1976) performed tests on a section using steel channels and found it to be an effective method of strengthening. The channels can also be easily reinforced with welded cover plates if additional strength is required. It should be noted that the bolts are placed above the longitudinal steel so that the stirrups can carry shear forces transmitted by the channels. If additional shear capacity is required, external stirrups could also be installed. It is also recommended that an epoxy resin grout be used between the bolts and concrete. The epoxy resin grout provides greater penetration in the bolt holes, thereby reducing slippage and improving the strength of the composite action.

4.4.3.3 Increasing the Shear Strength of Beams

The shear strength of reinforced concrete beams or prestressed concrete beams can be improved with the addition of external steel straps, plates, or stirrups. Steel straps are normally wrapped around the member and can be post-tensioned. Post-tensioning allows the new material to equally share both dead and live loads with the old material, resulting in more efficient use of the added material. A disadvantage of adding steel straps is that cutting the deck to install the straps leaves them exposed on the deck surface and thus difficult to protect.

Timber stringers with inadequate shear capacity can be strengthened by adding steel cover plates. *NCHRP Report 222* (University of Virginia Civil Engineering Department et al., 1980) demonstrates a method of repairing damaged timber stringers with inadequate shear capacity. The procedure involves attaching steel plates to the bottom of the beam in the deficient region and attaching it with draw-up bolts placed on both sides of the beam. Holes are drilled through the top of the deck, and a steel strap is placed on the deck surface and at the connection with the bolts.

4.4.3.4 Epoxy Injection and Rebar Insertion

The Kansas Department of Transportation has developed and successfully used a method for repairing reinforced concrete girder bridges. The bridges had developed shear cracks in the main longitudinal girders (Stratton et al., 1982). The procedure used by the Kansas DOT not only prevented further shear cracking but also significantly increased the shear strength of the repaired girders.

The method involves locating and sealing all of the girder cracks with silicone rubber, marking the girder center line on the deck, locating the transverse deck reinforcement, vacuum drilling 45-deg holes that avoid the deck reinforcement, pumping the holes and cracks full of epoxy, and inserting reinforcing bars into the epoxy-filled holes.

An advantage of using the epoxy repair and rebar insertion method is its wide application to a variety of bridges. Although the Kansas DOT reported using this strengthening method on two-girder, continuous, reinforced concrete bridges, this method can be a practical solution on most types of prestressed concrete beam and reinforced concrete girder bridges that require additional shear strength.

4.4.3.5 Addition of External Shear Reinforcement

Strengthening a concrete bridge member that has a deficient shear capacity can be performed by adding external shear reinforcement. The shear reinforcement may consist of steel side plates or steel stirrup reinforcement. This method has been applied on numerous concrete bridge systems.

A method proposed by Warner (1981) involves adding external stirrups. The stirrups consist of steel rods placed on both sides of the beam section and attached to plates at the top and bottom of the section. In some applications, channels are mounted on both sides at the top of the section to attach the stirrups. This eliminates drilling through the deck to make the connection to a plate.

In a study by Dilger and Ghali (1984), external shear reinforcement was used to repair webs of prestressed concrete bridges. Although the measures used were intended to restore the deficient members to their original flexural capacity, the techniques applied could be used for increasing the shear strength of existing members.

4.4.3.6 Post-Tensioning Various Bridge Components

Since the 19th century, timber structures have been strengthened by means of king post and queen post-tendon arrangements; these forms of strengthening by post-tensioning are still used today. Since the 1950s, post-tensioning has been applied as a strengthening method in many configurations to almost all common bridges.

Post-tensioning can be applied to an existing bridge to meet a variety of objectives. It can be used to relieve tension overstresses with respect to service load and fatigue-allowable stresses. These overstresses may be axial tension in truss members or tension associated with flexure, shear, or torsion in bridge stringers, beams, or girders.

Post-tensioning also can reduce or reverse undesirable displacements. These displacements may be local, as in the case of cracking, or global, as in the case of excessive bridge deflections. Although post-tensioning is generally not as effective with respect to ultimate strength as with respect to service-load-allowable stresses, it can be used to add ultimate strength to an existing bridge. Most often post-tensioning has been applied with the objective of controlling longitudinal tension stresses in bridge members under service-loading conditions.

The axial force, shear force, and bending moment effects of post-tensioning have enough versatility in application so as to meet a wide variety of strengthening requirements. Probably this is the only strengthening method that can actually reverse undesirable behavior in an existing bridge rather than provide a simple patching effect. For both these reasons, post-tensioning has become a very commonly used repair and strengthening method. Since the 1960s, external post-tensioning has been applied to reinforced concrete stringer and tee bridges. In the past 20 years, external post-tensioning has been added to a variety of prestressed, concrete-stringer and box-beam bridges.

Most uses of post-tensioning for strengthening have been on the longitudinal members in bridges, however post-tensioning has also been used for strengthening in the transverse direction. After the deterioration of the lateral load distribution characteristics of laminated timber decks was noted in Canada in the mid-1970s, (Taylor and Walsh, 1984), transverse post-tensioning was used to strengthen the deck. A continuous-steel channel whaler on each edge of the deck distributes the post-tensioning forces from threadbar tendons above and below the deck, thereby preventing local overstress in the timber. A similar tendon arrangement was used in an Illinois bridge (Lamberson, 1983) to tie together spreading, prestressed-concrete box beams.

The brief overview of uses of post-tensioning for bridge strengthening given above identifies the most important concepts that have been used in the past and indicates the versatility of post-tensioning as a strengthening method.

When post-tensioning is used as a strengthening method, it increases the allowable stress range by the magnitude of the applied post-tensioning stress. If maximum advantage is taken of the increased allowable-stress range, the factor of safety against ultimate load will be reduced. The ultimate load capacity thus will not increase at the same rate as the allowable-stress capacity. For short-term strengthening applications, the reduced factor of safety should not be a limitation, especially in view of the recent trend toward smaller factors of safety in design standards. For long-term strengthening applications, however, the reduced factor of safety may be a limitation.

Post-tensioning does require relatively accurate fabrication and construction and relatively careful monitoring of forces locked into the tendons. Either too much or too little tendon force can cause overstress in the members of the bridge being strengthened.

A large percentage of the single span composite steel-stringer bridges constructed in the United States between 1940 and 1960 have smaller exterior stringers. These stringers are significantly overstressed for today's legal loads; in some cases, the interior stringers are also overstressed to a lesser degree. Thus, most likely post-tensioning is only required for the exterior stringers, since through lateral load distribution, a stress reduction is also obtained in the interior stringers.

By analyzing an under-capacity bridge, an engineer can determine the overstress in the interior and exterior stringers. This overstress is based on the procedure of isolating each bridge stringer from the total structure. The amount of post-tensioning required to reduce the stress in the stringers can then be determined if the amount of post-tensioning force remaining on the exterior stringers is known. Researchers at Iowa State University, through research sponsored by the Iowa DOT, have developed a procedure for quantifying this through the use of force and moment fractions, (Klaiber et al., 1983; Dunker et al., 1985a; Dunker et al., 1985b; Dunker et al., 1986). This strengthening procedure has been used on several bridges in the states of Iowa, Florida, and South Dakota. In all instances, the procedure was employed by local contractors without any significant difficulties. (Beck et al., 1984; Klaiber et al., 1990).

Similar to the single span bridges, there are a large number of continuous span composite steel-stringer bridges that also have excessive flexural stresses. Through laboratory tests at Iowa State University on an one-third scale three span continuous bridge (Dunker et al., 1987; Dunker et al., 1990), it was determined that the desired stress reduction in most situations could be obtained by post-tensioning the positive moment regions of the various stringers. In the cases in which there are excessive overstresses in the negative moment regions, it may be necessary to use superimposed trusses on the exterior stringers in addition to post-tensioning the positive moment regions. Similar to single span bridges, force fractions and moment fractions are used in continuous span bridges to determine the distribution of strengthening forces in a given bridge. As one would expect, the design procedure is considerably more involved for continuous span bridges as one has to consider transverse and longitudinal distribution of forces (Klaiber et al., 1990; Klaiber et al., 1993a; Klaiber et al., 1993b; Planck, et al., 1993), El-Arabaty, et al., 1996; Wipf et al., 1995).

4.4.4 Developing Additional Bridge Continuity

4.4.4.1 Addition of Supplemental Supports

Supplemental supports can be added to reduce span length and thereby reduce the maximum positive moment in a given bridge. By changing a single-span bridge to a continuous, multiple-span bridge, stresses in the bridge can be altered dramatically, thereby improving the bridge's maximum live-load capacity. Even though this method may be quite expensive because of the cost of adding an additional pier(s), it may still be desirable in certain situations.

This method is applicable to most types of stringer bridges, such as steel, concrete, and timber, and has also been used on truss bridges (Sabris, 1983). Each of these types of bridges has distinct differences.

If a supplemental center support is added to the center of a 80 ft longer steel stringer bridge which has been designed for HS20 loading, the maximum positive live load moment is reduced from 1,165 ft-kips to 360 ft-kips, which is a reduction of over 69%. At the same time, however, a negative moment of 265 ft-kips is created which must be taken into account. In situations where the added support can not be placed at the center, reductions in positive moments are slightly less.

Depending on the type of bridge, there are various limitations in this method of strengthening. First, because of conditions directly below the existing bridge there may not be a suitable location for the pier, as, for example: the presences of a roadway or railroad tracks, poor soil conditions, the presence of a deep gorge, or the stream velocity.

This method is most cost effective with medium- to long-span bridges and thus may have limited application on LVRs.

The type of pier system employed greatly depends on the loading and also the soil conditions. A method employed by the Florida DOT (Roberts, 1978) can be used to install the piles under the bridge with limited modification to the existing bridge. This method consists of cutting holes through the deck above the point of application of the piles. Piles are then driven into position through the deck. The piles are then cut off so that a pier cap and rollers can be placed under the stringers.

Another major concern with this method is how to provide reinforcement in the deck when the region in the vicinity of the support becomes a negative moment region. If there is a noncomposite deck, the concrete deck does not carry any of the negative moment and therefore needs no alteration. For composite decks, the deck in the negative moment region should be removed and replaced with a properly reinforced deck.

4.4.4.2 Modification of Simple Spans

In this method of strengthening, simply supported adjacent spans are connected together with a moment and shear-type connection. Once this connection is in place, the simple spans become one continuous span, which alters the stress distribution. The desired reduction in the positive moment, however, is accompanied by the development of a negative moment over the interior supports.

This method can be used primarily with steel and timber bridges. Although it could also be used on concrete stringer bridges, the difficulties in structural connecting adjacent reinforced-concrete beams makes the method impractical. The stringer material and deck type obviously dictate construction details. This method also reduces future maintenance requirements because it eliminates a roadway joint and one set of bearings at each pier where continuity is provided (Berger, 1978).

The main disadvantage of modifying simple spans is the negative moment developed over the piers. To provide continuity, one must design for and provide reinforcement for the new negative moments and shears, as well as the increased vertical reactions at the interior piers.

When providing continuity for shear and moment transfer in timber stringers, steel plates can be placed on both sides and on the top and bottom of the connection and then secured in place with either bolts or lag screws. Additional strength can be obtained at the joint by injecting epoxy into the timber cracks as is suggested by Avent et al. (1976).

4.4.5 Recent Strengthening Developments

4.4.5.1 Epoxy Bonded Steel Plates

Epoxy-bonded steel plates have been used to strengthen or repair buildings and bridges in many countries around the world. The principle of this strengthening technique is rather simple: an epoxy adhesive is used to bond steel plates to overstressed regions of reinforced-concrete members.

Although this procedure has been used on dozens of bridges in other countries, to the authors' knowledge, it has not been used on any bridges in the United States due to concerns with the method. Some of these concerns are plate corrosion, long-term durability of the bond connection, plate peeling, difficulties in handling and installing heavy plates.

A summary of work around the world utilizing epoxy-bonded steel plates for bridge strengthening is given by Eberline, et al. (1989). The authors state that a number of countries have used epoxy-bonded steel plates for strengthening of concrete bridges. However, since little work has been completed in the United States, a complete summary of applications had not previously been completed. Information on the following topics are included; bonding procedures, impact of plate geometry, and effects of cyclic loading. In addition, numerous applications of this technique are presented along with an extensive table summarizing where specific information can be found.

In recent years, the steel plates used in this strengthening procedure have been replaced with fiber reinforced plastic sheets; the most interest has been in carbon fiber reinforced polymer (CFRP) strips.

4.4.5.2 CFRP Plate Strengthening

CFRP strips have essentially replaced steel plates as CFRP has none of the previously noted disadvantages of steel plates. Although CFRP strips are expensive, the procedure has many advantages: less weight, strengthening can be added to the exact location where increased strength is required, strengthening system takes minimal space, material has high tensile strength, no corrosion problems, easy to handle and install, and excellent fatigue properties. As research is still in progress in Europe, Japan, Canada and the United States on this strengthening procedure, and since the application of CFRP strips obviously varies from structure to structure, rather than providing details on this procedure, several examples of its application will be described in the following paragraphs.

The techniques of FRP plates are now established as a relatively simple rehabilitation/strengthening procedure that can significantly improve the shear and flexural performance of various types of structural elements. Bridge beams and slabs in particular have been strengthened using this technique. Swiss researchers are generally credited with doing the initial research on the use of FRP for strengthening (Meier and Kaiser, 1991). There are literally dozens of articles published on laboratory studies on the use of FRP for strengthening reinforced and prestressed concrete elements. Only a few of these are presented in this report. The majority of the articles presented will be on the field applications of FRP. One of the more comprehensive studies of an FRP strengthening system (essentially all aspects of materials, design, and analysis were covered) was undertaken in the United Kingdom (Hollaway and Leening, 1999).

Recently, a prestressed concrete (P/C) beam in West Palm Beach, Florida which had been damaged due to being struck by an overheight vehicle, was repaired using CFRP. This repair was accomplished in 15 hours by working three consecutive nights with minimal disruption of traffic. The alternative to this repair technique was to replace the damaged P/C with a new P/C beam. This procedure would have taken close to one month, and would have required some road closures. This procedure of using CFRP has also been used in Iowa as was reported in the final report for Iowa DOT project TR-428 (Wipf et al., 2004). In this report, information on the repair of three P/C bridges that were damaged by overheight vehicles is presented. This report also provides information on the design and application of a CFRP strengthening system.

The use of FRP deck panels as a means of increasing live load capacity during the rehabilitation of an old thru-truss bridge is discussed in Alampalli and Kunin (2001). As a response to the bridge conditions in the State of New York, which as of 2001 indicated that 38.9% of the bridges in the State were functionally deficient or functionally obsolete, innovative solutions are being sought to extending the service life of existing structures. Over 26% of these bridges were classified deficient due to poor deck conditions or weight restrictions. One of the bridges retrofitted with a lighter deck system is a simply supported Warren steel truss, 140 ft long, 25 ft wide curb-to-curb and skewed 27 degrees. The deck system is a cellular core product manufactured by Hardcore Composites of Delaware. The existing deck and asphalt overlays weighed a combined 170 psf while the FRP retrofit deck weighed only 32 psf. This light deck, in conjunction with minor retrofit to the steel superstructure, was sufficient to remove the load restrictions and extend the service life of the bridge.

Since this was the first FRP deck used in the US on a state highway, conservative design assumptions were used and a field-testing program was implemented to verify performance of the completed structure. The field test objectives were to determine if composite action occurred between the deck and floorbeams; determine the effectiveness of the joints between panel segments; verify the deck load rating; and acquire strain data for calibration of a finite element model. Test results indicated no composite action between the deck and floorbeams. Additionally, the test results indicated incomplete load transfer between adjacent panels along the epoxied longitudinal joint.

The study of redecking an aging truss bridge with FRP deck panels indicated that the deck was effective in keeping a previously load restricted bridge in service. The deck was installed in one month, reduced the bridge dead load by 265 tons and cost $800,000 as opposed to $2.2 M for a replacement structure.

Work completed at Georgia Institute of Technology by Zureick (1999) has determined that FRP materials can make bridges 30-40% stronger than the original design. On-going work includes exposing FRP components to extreme environmental conditions. Results from these studies will be used to develop predictive models for FRP life spans. Zureick is also developing national guidelines for the use of FRP materials in repair projects.

A unique project is described by Halstead, et al. (1999) in which six FRP manufacturers participated in a demonstration project to determine if the application of FRP wraps provides an efficient, cost-effective solution for the short-term rehabilitation of bridges. At the Owego, New York test site, a series of deteriorated columns were evaluated after four different repair systems were installed. The four options, column replacement, concrete repair, steel jacketing and wrapping with FRPs, were evaluated for their cost-effectiveness. FRP was found to be the most economical; the long-term performance of the six FRP repairs are being monitored and will be reported on in the future.

Based on numerous field projects, Shahawy, et al. (2001), present a series of guidelines that an engineer can follow when recommending construction of FRP based projects. These recommendations, which are based on 10 years of field applications, give information on the selection of FRP components from both an environmental and economic viewpoint. They also cited the general factors that must be considered during installations (e.g., ambient temperature, condensation, surface defects and corners, primer and resin, handling of FRP sheets, and section preparation) as well as the general procedures required to install and inspect these materials.

In a field test to determine the effectiveness of externally bonded FRP plates on a reinforced concrete bridge, Stallings et al. (2000) found that the retrofit was a simple and straightforward process that reduced reinforcing steel stresses 4 to 12% and girder deflections 2 to 12%. Using classical cracked-section moment of inertia calculations, the moment of inertia was determined to increase by only 5%. Thus, it was concluded that more advanced procedures are needed to accurately determine the benefit of FRP plates.

Triantafillou and Antonopoulous (2000) have presented a simple design model for determining the contribution of FRP to the shear capacity of strengthened R/C elements. The proposed model predicts the FRP contribution in an analogy to conventional shear reinforcement. It was shown that the proposed model gives results that are in better agreement with most available test results than previously proposed models. The use of a FRP composite deck on existing pre-cast concrete beams took place on Five Mile Road in Hamilton County, Ohio. This project was unique in that a method of attaching the FRP deck panels to the existing concrete beams was developed. The concrete beams also had to have a thicker top flange added to increase their stiffness as the FRP deck didn't contribute to the structural rigidity the way concrete decks do.

Rizkalla and Hassan (2002) investigated the effectiveness of five different FRP systems in strengthening half-scale models of prestressed concrete bridge slabs. Systems investigated were:

- Two types of CFRP bars installed and bonded in shallow near surface grooves
- Externally mounted CFRP strips
- Near surface mounted CFRP strips
- Externally bonded CFRP sheets

Based on their experimental investigation, the following conclusions were made:

- Externally bonded CFRP sheets are the most efficient technique in terms of increased strength and lower construction costs
- Use of near surface mounted CFRP reinforcement is feasible for strengthening or repairing prestressed concrete girders or slabs
- Stiffness and strength of concrete slabs strengthened with CFRP materials were substantially increased
- Magnitude of strength increase was influenced by the type and configuration of the CFRP materials
- Strengthening using externally bonded CFRP strips provided the least increase in strength (11%) due to peeling of the strips from the concrete.

Currently, the authors are working on design guidelines for determining the developmental length needed for the various proposed FRP strengthening techniques. Through laboratory tests, Miller et al. (2001) determined the effectiveness of bonding CFRP plates to the tension flanges of steel bridge girders to increase their stiffness and strength. The durability of the bonded CFRP plates to various environmental conditions and fatigue was also determined. Increases in stiffness ranging from 10 to 37% were achieved in the laboratory. As a result of the successful laboratory study, one of the steel beams of the I-95 bridge over Christiana Creek outside of Newark, Delaware was strengthened with CFRP plates. To determine the effectiveness of the added CFRP plates, diagnostic load tests were performed before and after their installation. Based on test results, the retrofit produced a 11.6% increase in the global flexural stiffness. Test results to date, indicate the procedure is very promising.

Hag-Elsafi et al. (2001) of the New York DOT reported on the use of FRP laminates to contain freeze-thaw cracking and to improve the flexural and shear strength of a reinforced-concrete T-beam bridge built in 1932. Based on load tests conducted before and after the laminates were installed, it was determined that when the bridge was subjected to service loads, the strengthening system slightly reduced the stresses in the longitudinal reinforcement and moderately improved transverse live load distribution. In this project, the FRP strengthening was found to be cost-effective ($300,000 for the rehabilitation vs. $1.2 M for a replacement structure) with essentially no interruption of traffic.

Additional documents are currently available for assisting engineers in the use of FRP in the repairing of reinforced concrete structures. ACI Committee 440 – Fiber Reinforced Polymer Reinforcement has prepared guidelines for the use of FRP in the strengthening of concrete structures (ACI Committee 440, 2003). Although these guidelines are primarily for strengthening buildings, a significant portion of the guidelines are also applicable to bridges. Mirmiran (2003) has submitted the final report for NCHRP Project 10-59 “Construction Specifications for Bonded Repair and Retrofit of Concrete Structures Using FRP Composites." This report is currently being reviewed by the project panel.

5. BRIDGE REPLACEMENT ALTERNATIVES

5.1 Introduction

This chapter explores the various aspects of LVR bridge replacements. Included in the discussion are the design rules as they apply to the geometric and structural design of the bridges, an extensive discussion of literature relevant to off-system bridge replacements, and finally a section on the use of software, standard plans and design aids to expedite the design and construction of off-system bridges.

5.2 Previous Work

There has been previous work to determine economical solutions to low-volume road bridge problem. Several of these investigations are initially discussed to provide general information while specific solutions are presented later in the chapter.

NCHRP 222 and 243 (University of Virginia 1980, 1981) are companion reports specifically addressing the problems of bridge rehabilitation and replacement on low volume roads. Both of these reports were products of NHCRP Project 12-20 “Bridges on Secondary Highways and Local Roads – Rehabilitation and Replacement”. The focus of the project was to identify common local road bridge deficiencies, evaluate feasible corrective procedures, evaluate economical bridge replacement systems and to develop decision trees to assist local agency engineers in making repair or replacement decisions. Only the findings and recommendations relevant to bridge replacement are presented in this chapter.

In NCHRP 222, the focus is on the repair and replacement of bridge superstructures. The first step is determining the “most appropriate” alternative for the specific project objectives. The objectives identified for consideration are required structural capacity, traffic volume, anticipated future use, labor required for construction and finally cost. Additional factors can include familiarity with the bridge type considered, available contractors, budget, material availability and environmental priorities. These are the most significant factors one should consider in the development of work priorities on a system of bridges and/or in the selection of appropriate replacements.

In one chapter of the NCHRP 222 report, a series of bridge replacement systems including concrete, steel and timber bridge superstructure replacement systems as well as construction of other bridge elements such as bridge substructures, deck forming, bridge railings and buried pipes and conduits are presented. A total of 27 bridge replacement systems are identified and briefly discussed; each replacement system is cross-referenced to other references for more detailed information.

In general, the NCHRP 222 bridge replacement options are fairly standard forms of construction used on LVR. The concrete bridge options include precast slabs and box beams, as well as prestressed concrete products such as double-tee, channel beam, multi-stem beam, single tee, bulb tee and I-girders. For steel bridges, some of the options presented are no longer likely to be cost-effective due to their complicated fabrication. Some of the options presented: steel decking with asphalt paving on top of multiple stringers, timber decks of several forms over steel beams, steel grid decks, and several types of precast concrete decking in addition to conventional CIP concrete are appropriate for LVRB replacements. The timber bridge options include glue laminated timber I-beam construction, nail laminated timber slab bridges with a wearing surface, solid sawn timber bridges with decking and plywood decking on top of timber planks which rests on the main stringers. All of these systems have pluses and minuses, many of which are identified in the NCHRP report.

A follow-up to the NCHRP 222 report is NCHRP 243. This report essentially focuses on an expansion of repair techniques and presents only a few minor additions to the list of replacement systems identified in NCHRP 222.

Wipf, et al. (1994) present research results concerning the evaluation of suitable options for county bridge replacements and also developed new bridge concepts based on the desired characteristics of county bridge replacements. The study endeavored to determine the reasons for bridge replacements, bridge replacement types and costs, participation of local forces in design and construction, expected life, foundation types, and degree of satisfaction of county bridge owners with various bridge types. Following this information gathering process, some new bridges were developed that met the objectives of county engineers. Additionally, standard solutions already in use were presented along with a brief discussion of their design and construction. Based on the project survey conducted in Wipf et al, which surveyed county engineers in Iowa and surrounding Midwestern states, the most common reasons for bridge replacement were insufficient load capacity, excessive deterioration and inadequate roadway width. Over three quarters of the respondents indicated insufficient capacity was the primary reason for replacement.

Concerning the selection of replacement options for inadequate bridges, the most common replacement option noted by the survey respondents was a continuous concrete slab bridge with 36% of deficient bridges being replaced with this kind of new construction. In second place, with a use rate of 31%, were prestressed concrete beam bridges. Concrete culverts were third with their use being cited as the replacement option 17% of the time. Other types such as timber, reinforced concrete, corrugated metal pipe and low water stream crossing round out the balance of choices. Costs of these bridges were also compiled. Using cost data from the early to mid-1990's, prestressed concrete girder construction was the most expensive with unit costs for the entire bridge averaging approximately $\$58/ft^2$. The next most expensive type was a concrete slab at a cost of $\$50/ft^2$. In descending order after that are precast reinforced concrete bridges, steel stringer bridges and timber stringer bridges.

In regard to a county's ability to construct the various types of bridges, only the two most common types of bridges were examined. The sample size for steel stringers, reinforced concrete girders and timber bridges were not large enough to draw meaningful conclusions. Only 12% of reinforced concrete slabs and 14% of prestressed concrete beam bridges were constructed by county forces; the primary reasons for this were the lack of appropriate heavy equipment, and/or requirements for extensive formwork. Counties were also queried as to their capabilities to construct as it relates to available equipment. With in-house equipment, the typical bridge that could be constructed would be one in the 40 ft. range and would be constructed on timber piles. With rented equipment, both the size of bridge and pile size could be increased; other types of piles could be installed as well.

Types of foundations used were also examined, and it was determined that the two most common foundation types were steel H-piles and timber piles. Steel piles were usually only used on contractor constructed bridges while timber piles were used on both contractor and county built bridges. Spread footings and concrete piles are used sparingly and neither has been constructed with county forces.

GangaRao and Hegarty (1987) present a series of recommendations for design and construction of LVRBs. The premise of their work was that the existing AASHTO specifications, at the time only the Standard Specifications were in existence, did not reflect the uniquely different requirements for the design of LVRBs.

GangaRao and Hegarty examined four critical decisions that impact the total cost and anticipated value of LVRBs: design specifications, number of components, materials, and safety features.

One of the statements the authors make is that the AASHTO Standard Specifications are too conservative with respect to the design of LVRBs; specifically, the provisions related to fatigue, impact, lane load and deflection criteria of existing codes are too conservative.

For fatigue, they recommend the use of the lowest number of fatigue cycles or neglecting fatigue entirely. They justified this statement by considering the low traffic volumes and the infrequent cycles of heavy vehicles. The impact factor was discussed in their study and recommended to be taken as a constant 30%. This is the high end of the impact factors specified in the Standard Specifications and slightly less than that used in the LRFD Specifications, 33%. A discussion of the relevance of the AASHTO Standard Specification lane load is presented. Since this loading is intended to represent a string of vehicles, i.e. a truck train, it was suggested it could be neglected in the design of LVRBs. This is not necessarily a change in the design of most LVRBs since lane load provisions do not control flexural or shear design of simple span bridges of usual span lengths.

Finally, concerning deflection, a relaxation of the live load deflection requirements was proposed to a level consistent with that used in building design, L/360. This is based on the infrequent use of a given bridge by more than one vehicle at a time.

Concerning bridge geometric standards, GangaRao and Hegarty advocate consideration of the design of one-lane bridges with roadway widths of 12 to 15 ft. with the caveat that for bridges with significant agricultural or commercial use, high speeds or poor alignments, or in the vicinity of future development, design/construction of wider bridges should be considered. They also discuss the possibility of constructing one lane bridges on two lane roads though this is strongly discouraged by most other sources. The justification for a narrower bridge was economics and was based on anticipated savings in bridge materials. With regard to the selection of proper types of structures for LVRBs, this was approached from the perspective of economics and durable choices for bridge decks, superstructures and substructures.

For bridge decks, the most appropriate deck type will be one that is easily obtained from local sources, is familiar to construction and maintenance crews, and is economical. These are regional factors and thus the appropriate deck type is not a single choice. The several deck types that are considered viable, in addition to CIP concrete, include precast and prestressed concrete deck panels, open steel grid decks and glulam deck panels. Concrete filled steel decks were discounted by GangaRao and Hegarty though there is evidence that they also are viable choices.

Concerning bridge superstructures, the authors discounted cable supported structures, built-up steel sections (plate girders) and truss bridges. They additionally discounted precast reinforced concrete members due to their span length limitation and their structural inefficiency as compared to prestressed concrete members in similar span ranges. The availability of prefabricated truss bridges composed of weather resistant construction (galvanized or self-weathering steel) are economic choices in some locations. Also, although inefficient as compared to prestressed concrete bridges, the ability to locally fabricate precast reinforced concrete members using local forces, sometimes in close proximity to the bridge, is an advantage that sometimes overrides structural efficiency. GangaRao and Hegarty do advocate the use of prestressed concrete beams, as previously mentioned, as well as glulam stringers and rolled shape steel sections. Various combinations of these types of beams and before mentioned deck systems are appropriate depending on the required span length and the availability of the various materials.

Regarding bridge abutments, a comparison between stub and full height vertical abutments was made. It is concluded that unless an entire span can be eliminated through use of the full height abutment, the economics are usually in favor of the stub configuration. Also, deep foundations are not advocated for low-volume construction as the cost of piling could be offset by larger spread footings. Caution should be exercised, however, when scour is a consideration. Integral abutments on piles and jointless stub abutments are only briefly mentioned but they may be the most economical choice from both a first cost and total ownership cost perspective.

5.3 Design Rules for Off-System Bridges

The design policies for off system bridges are discussed in this section. The information presented is regulatory information relative to minimum design standards.

In terms of federal regulation, with the exception of bridges on the National Highway System which are not addressed in this study, there is no federal mandate regarding minimum design standards. In Title 23 USC 109, “Standards”, the regulations regarding design standards are established. Specifically, 23 USC 109(o) “Compliance with State Laws for Non-NHS Projects” states the following:

> "Projects (other than highway projects on the National Highway System) shall be designed, constructed, operated, and maintained in accordance with **State laws** (emphasis added), regulations, directives, safety standards, design standards, and construction standards."

Similar to the requirements of 23 USC 109(o) are those stated in 23 CFR 625 **–** *Design Standards for Highways*. Specifically, 23 CFR 625.2(b) concerns design criteria for 3R projects. It states that:

> "…[projects] shall be constructed in accordance with standards which preserve and extend the service life of highways and enhance highway safety. [Work] includes placement of additional surface material and/or other work necessary to return an existing roadway, including shoulders, bridges, the roadside, and appurtenances to a condition of structural or functional adequacy."

The FHWA Federal Aid Policy Guide NS 23 CFR 625, Non Regulatory Supplement indicates that for non-NHS projects "…the states are strongly encouraged to consider and apply these provisions [23 USC 109(o)] in developing and applying their non-NHS standards." The implication of this statement is clear that even though the federal regulations are not applicable to non-NHS projects, FHWA considers the NHS level standards as reasonable standards for non-NHS projects as well. The FHWA Guide further indicates, though this again is for NHS level structures, the following desirable objectives for new, reconstructed or rehabilitated bridges:

- Bridge Widths – The geometric standards referenced are those mandated by 23 CFR 625, specifically the AASHTO Green Book. Flexibility is provided for bridge width for 3R projects.
- Treatment of Existing Bridge on 3R Projects – Each bridge should be assessed for structural and functional adequacy considering minimum bridge widths for retention of the existing structure and the suitability of the existing rail system. Upgrading of obsolete railings is strongly encouraged. Rehabilitated bridges should be designed to a minimum of H15 and have a minimum service life of 15 years. Bridge replacements should be in accordance with the latest AASHTO standards.

Again, though not strictly applicable to non-NHS structures, these recommended practices represent a framework of reasonable design objectives and standards that can be modified on a case-by-case basis.

5.3.1 Geometric Design Rules

Geometric design rules and guidance from several sources are discussed herein. These include the traditional AASHTO Green Book (2001) and other sources.

5.3.1.1 AASHTO Guidance

Though not a bridge design manual, the AASHTO Green Book discusses minimum roadway widths at bridges as well as recommended minimum structural capacities for new bridges and existing bridges to remain in service. The recommendations are explicitly restricted to bridges less than 100 ft in total length. The recommendations are presented in the context of the roadway classification, specifically local roads and streets, collector roads and streets, rural and urban arterials and lastly freeways. Since the focus of this project is LVRBs, the freeway / interstate level structure criteria will not be discussed. It should be noted that AASHTO (AASHTO Geometric Design 2001) also recently published geometric design guidelines for LVRs with ADT < 400 v.p.d. These guidelines may be used in lieu of the Green Book, however, the new design values do not imply that existing roads are unsafe, nor do they mandate the initiation of improvement projects. These guidelines address issues where policies for very LVR and high volume roads differ and they are intended to provide a range of values for critical dimensions. Of particular value in this reference are the numerous design examples provided.

The Green Book establishes a two-level criteria for bridge geometrics and minimum acceptable capacities, one level for new or reconstructed bridges, the other for bridges to remain in place. For new structures the minimum recommended design loading for all classes of bridges is recommended as HS20. For bridges to remain in place, with the exception for very low volumes (0 – 50 v.p.d.) on local roads, the minimum recommended capacity for bridges is H15. When a road is to be reconstructed and the existing bridge meets the proposed alignment and profile, the bridge may remain in place when its structural capacity meets the tolerable criteria, an example of which is presented in Table 5.1 for county roads. Similar tables exist for other functional classifications such as collector and arterial roads.

When deciding whether to retain an existing bridge, some of the factors to consider include the aesthetic and historical significance of the bridge, cost of replacement, remaining life, consideration as to whether the highway improvements will promote design speeds inconsistent with bridge safety features and accident history. For structures in excess of 100 ft, although no specific recommendations are given with respect to roadway width and minimum design loading, additional criteria that may be relevant include pedestrian volume, snow storage, design speed, crash history and other unique site features.

5.3.1.2 FHWA Guidance

In order to both protect the scenic, historic and other environmental features of existing highways or along proposed routes in conjunction with promoting safety and levels of service required of a modern transportation facility, the FHWA has published a guide, *Flexibility in Highway Design* (FHWA, 1997) that address the choices engineers can make to achieve the various objectives. Essentially a guidebook tied to the AASHTO Green Book but illustrating the flexibility of applying the criteria instead of strict rigid interpretation, the FHWA publication provides a valuable commentary and has various case studies of successful projects that have integrated the various environmental and

Table 5.1. Recommended Minimum Geometric and Structural Capacities for Local Rural Roads (AASHTO 2001).

	DESIGN VOLUME (V.P.D)	MIN. CLEAR ROADWAY WIDTH OF BRIDGE	DESIGN LOADING
New or Reconstructed Bridges	< 400	Traveled way + 2 ft each side	HS20
	400 – 2000	Traveled way + 3 ft each side	HS20
	≥ 2000	Approach roadway width	HS20
Bridges to Remain in Place	0 – 50	20 ft	H10
	50 – 250	20 ft	H15
	250 – 1500	22 ft	H15
	1500 – 2000	24 ft	H15
	≥ 2000	28 ft	H15

safety aspects of transportation engineering. Much of the flexibility in highway and bridge design available to local road designers stems from the legislative provisions of the 1991 ISTEA legislation as wells as the NHS Act of 1995. Specifically, States may develop criteria they deem appropriate for projects not on the NHS system. Although the flexibility to develop standards apart form those recommended by the Green Book is present, many states have adopted design criteria for non-NHS structures that are similar to those used on the NHS system.

The FHWA guide briefly addresses the issue of tort liability. Published standards are typically used in tort cases as a basis for educating the public as to reasonable standards of care to be exercised in highway design. This does not imply that strict adherence to published standards is an absolution of liability nor does it imply that deviation from the standards constitutes liability. Defense of deviation from the standards is most effective when it centers around the inapplicability of the standards for a sound reason; economic hardship is not a persuasive argument.

5.3.1.3 Sample State Policies

Concerning state interpretations of the flexibility provided for non-NHS Bridges, the States of New York and Pennsylvania are compared. These states have some similarities to Iowa in that they both have large metropolitan areas, large bridge populations, extensive road networks, significant lane mileage, and a large number of bridges in largely rural areas. They also have significant LVR and LVRB problems and have the same flexibility to develop local road system design guidelines.

The New York State guidelines (NYSDOT 1999) are specifically restricted to the geometric design of locally owned low volume highways with ADT < 400 vpd and may be used on all such projects regardless of funding source. The NYSDOT Manual classifies low volume roads in a number of different categories including: Low Volume Collector, Residential Access, Farm Access, Resource/Industrial Access, Agricultural Land Access and Recreational Land Access. Depending on these classifications, the types of vehicles using the road and ADT, an Operational Type is assigned ranging from a Type A through Type C. Type A roads are two-lane, two-way facilities with the highest design speeds and provisions for opposing vehicles passing at safe operating speeds. Type B roads are two-lane, two-way roads with speeds and operational characteristics appropriate for local streets. Finally, the Type C roads are single-lane one-way or two-way roads with local road design speeds.

For approach roadway width and minimum bridge widths, the NYSDOT Manual recommends road widths from 10 ft for Type C roads to as much as 20 ft for the Type A roads. Lane, shoulder and clear zone widths are also specified for each road type as is the recommended paving material, either asphalt concrete or aggregate surfacing. It is recommended that in the case of anticipated farm vehicle use, a minimum bridge width of 20 ft be used.

The Penn DOT policy was also examined (PennDOT 2000). The PennDOT procedure does not subdivide LVRs into various types as was done in the NYSDOT manual, rather all bridges less than 400 vpd. are treated the same with distinctions made for urban or rural situations. For replacement bridges, similar to the criteria described previously for the State of New York, the minimum roadway width for collector and local roads is specified to be 24 ft whereas the NYSDOT maximum width is 20 ft. The minimum required structural capacity is a PennDOT modified version of the AASHTO LRFD loading designated PHL-93.

Comparison of these two states with very similar needs and existing conditions shows the great latitude these states have exercised in developing local road design standards. It appears reasonable that some latitude in selecting bridge widths and design loading for LVRs should be provided.

5.3.2 Structural Design Criteria for New Bridges

Structural design criteria are discussed with respect to the design for vertical loads as well as for the structural design of bridge railings. The current design policies commonly in use are presented in the following sections.

5.3.2.1 Design for Vehicular Load

At the present time there are two primary design specifications for bridge structures, the AASHTO Standard Specifications for Highway Bridges, 16th edition ("Standard Specifications"), and the LRFD Bridge Design Specifications ("LRFD Specifications"), 2nd edition. The Standard Specifications have been in continual use for seventy years and have the traditional allowable stress and load factor design approaches for highway bridges. The Standard Specifications are still the predominant bridge design specification in use today. Most likely these specifications are used almost exclusively for the design of LVR bridges due to their familiarity and relative simplicity. In 1994, AASHTO introduced a new specification, the LRFD Specifications, intended to replace the Standard Specification. This new specification was based on probability theory when possible and calibrated to successful past practices to assure a more uniform level of safety amongst structures of various materials, span ranges, bridge widths, etc. The new specification resulted however in significant changes in loading, load distribution, load combinations and in some cases design methods from those found in the Standard Specifications. At this time, the LRFD Specifications are not universally adopted by the states with various levels of adoption from full use to no use at all.

Regarding the actual design loads used by the various specifications, the Standard Specification uses either the common “H” or “HS” classes of loading. The H series loads prescribed by AASHTO are H15 and H20 the number representing the gross vehicle weight in tons. For the HS classes, the number represents the weight of the tractor portion of the semi-trailer combination. The AASHTO prescribed loads are HS15 and HS20. In recognition of heavier truck loads as routine vehicles and as special permit vehicles, a number of states and presumably some local agencies have increased the AASHTO loading class; the most common modified design load is an HS25 vehicle which is a 25% increase in loads over that prescribed by AASHTO.

Along with the introduction of the LRFD Specifications came a new set of live loads. The national loading, known as HL93, is a hybrid of the Standard Specification live loads as it involves a combination of truck and lane loads simultaneously. Instead of the live loads being an either / or choice of truck or lane loads, they are now combined together in a single live load model whose effects are significantly greater than the older HS20 loadings but not much different than HS25 when one considers the additional LRFD changes in load factors, load combinations, impact and load distribution to the individual girders.

5.3.2.2 Railing Design Loads

The design of railings has also evolved with changes in specifications. Properly designed railings must prevent the vehicle impacting the railing from leaving the bridge and as importantly from being redirected back into the roadway or into oncoming traffic. The railings must be designed for both structural and functional requirements.

Although static force design procedures have been used for years in railing design, i.e., the AASHTO Standard Specifications 10 kip criteria, modern design procedures use dynamic crash tests as more appropriate measures of railing performance.

In 1981, NCHRP published *NCHRP Report 230: Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances* outlining the crash test requirements for roadside hardware. At the time of introduction, NCHRP 230 procedures did not mandate the use of crash testing in the design of roadside hardware. Following crash test failures of some systems designed in accordance with static design procedures, since 1986 the FHWA has required that all bridge railings used on Federal-aid projects meet crash test criteria and be tested accordingly. A tentative list of 22 crash tested bridge railings was released with the 1986 memorandum.

In 1989, AASHTO published the first national design specification for bridge railings based on crash tests. The *Guide Specifications for Bridge Railings* (Guide Spec) prescribed a series of Performance Levels for bridge railings ranging from the PL-1 to PL-3 levels, PL-1 being the least demanding criteria and PL-3 the most demanding. Subsequent to the Guide Spec publication, NCHRP published *NCHRP Report 350: Recommended Procedures for the Safety Performance Evaluation of Highway Features* which prescribed six Test Levels from TL-1 to TL-6, with TL-1 being the least restrictive and TL-6 the most. For several years the conflicting PL and TL systems existed until the publication of the second edition of the AASHTO LRFD Specifications in 1998 at which time AASHTO adopted the TL railing designations.

In a FHWA memoranda issued in 1990 and again in 1997, and in conjunction with the aforementioned changes in crash test criteria, the list of acceptable railings was updated so that as of the 1997 memorandum, 74 railing systems had been crash tested (FHWA 1997). These systems are listed in an Appendix to the 1997 FHWA Bridge Rail Memorandum and include the following types:

- W-Beam Bridge Rail (2 types)
- Thrie Beam Bridge Rail (9 types)
- Metal Tube Bridge Rail (25 types)
- Vertical Concrete Parapet (25 types)
- F-Shape Concrete Barrier (4 types)
- Timber Bridge Rail (9 types)

Due to the various railing design criteria that have existed through the years (NCHRP 230 and 350, AASHTO Guide Spec, LRFD), and the various times that individual rail systems were introduced, a correlation matrix was established for the previously tested rail systems to indicate accepted equivalencies between the various test requirements. The FHWA indicates (FHWA 1996) that railings tested under NCHRP 230, the Guide Spec or the LRFD Specifications will be accepted as meeting NCHRP 350 standards as described in Table 5.2.

Table 5.2. Bridge Railing Test Level Equivalency (FHWA 1996).

Bridge Railing Testing Criteria	**Accepted Equivalencies**					
NCHRP Report 350	TL-1	TL-2	TL-3	TL-4	TL-5	TL-6
NCHRP Report 230		MSL-1 MSL-2		MSL-3		
AASHTO Guide Specifications		PL-1		PL-2	PL-3	

Of the numerous crash tested systems now recognized by the FHWA, there is significant flexibility in selecting the appropriate railing design for a project. Typically, states will have standard railings that have been subjected to the crash testing requirements of one of the aforementioned reference standards. However, latitude is presented to select alternate systems, some of which are open rails, have special aesthetic detailing, and range from low cost to expensive systems depending on the railing design and means of attachment.

Bridge rails can be an expensive component of a project whether it is new construction or bridge rehabilitation. Due to the cost of the railing and the various functions the rail serves (structural, functional, aesthetic, etc.), care should be taken when selecting the appropriate railing.

5.4 Bridge Replacement Options

In this section some of the standard and innovative solutions for replacement of LVRBs found in the literature are presented. These solutions may consist of the use of traditional materials and construction techniques, innovative materials, time-saving construction techniques, use of standardized solutions requiring minimal or no design, or combinations thereof. Due to the combination of limited budgets and bridges that are generally small to medium in size, non-traditional structures and techniques may be more prevalent off-system than on more heavily traveled highways. Administrators of LVRBs need to find ways to reduce the expensive engineering and construction costs typical of normal design and construction processes while still obtaining durable, safe structures. The use of innovative solutions is one way this can be accomplished.

5.4.1 Substructure Options for Low Volume Road Bridges

In general, this project focuses on superstructure related issues. This is a reflection in large part on the amount of literature available with respect to bridge superstructures versus substructures. Substructures, however, are a very important element that should be addressed to the extent possible. Although they are not as prone to maintenance problems, when substructures do have a problem it is typically an expensive problem and one that is difficult to remedy. Concerning new structures, the choice of substructure type has a profound impact on structure cost. This is especially true in locations where deep foundations are required due to unsuitable soil and rock conditions or concerns with scour. The construction of bridge substructures should be appropriate for the site, focus on durability and stability, and pose minimal maintenance problems.

5.4.2 Prefabricated Bridges

There is a significant amount of information in the literature concerning the use of prefabricated bridge products. Prefabricated products include everything from ordinary precast concrete I-beams and rolled steel shapes to the more innovative products, systems and assemblies that can be used to expedite construction and provide long life. The application of some of these concepts is presented in the following sections.

5.4.2.1 Prefabricated Concrete Bridges

The most common material prefabricated for bridge construction is one made of concrete. Whether using short span precast reinforced concrete elements or precast and prestressed concrete elements, the use of concrete is common in prefabricated bridges. A description of some of the findings relative to the use of prefabricated concrete bridges follows.

The development and load testing of a short span bridge concept using precast double-T beams transversely post-tensioned through the slabs for load distribution is discussed in Shahawy (1990). The concept is additionally innovative in that a cast-in-place topping is not required; the precast flanges form the riding surface. The edges of the precast slabs (of the double-T sections) are slightly beveled so that a cast-in-place closure joint can be poured to join adjacent sections. Post-tensioning is then applied to create a transversely continuous section. The bridge type was developed as a combined effort of the Florida DOT and local precasters and is an effective bridge replacement concept for spans up to approximately 65 ft Although longer spans may pose handling or shipping problems, it is anticipated that the system can be used on spans up to 80 ft. A savings of approximately 15% was realized in the construction of the first prototype bridges due to reduced erection time and the elimination of a CIP deck.

In response to a need for cost-effective shallow structure alternatives to CIP continuous concrete slab bridges, an inverted tee (IT) girder system was developed by the University of Nebraska and the Nebraska Department of Roads; it is discussed by Mounir and Tadros (1996). The lightweight precast units have a high span to depth ratio, are relatively easy to fabricate, and require minimal fieldwork. The intended applications of the system is in rural environments where erection of heavy units is difficult, in new construction where superstructure depth must be minimized and conventional forming and construction are not practical, and in superstructure replacement situations where greater spans or load capacities are required with comparable depths.

The IT system is available in various girder depths ranging from 12 in. to a maximum of 35 in. When used in simple span construction, the maximum span of the 35 in. deep section is approximately 110 ft. The girders may be made continuous, either before or after casting the deck, to extend the span several feet. The girders use a standard bottom 24 in. wide flange form and variable web sizes to achieve the various section depths. Although the girders are placed side-by-side, they are not connected together mechanically. The short slab span can either be formed with traditional formwork that cannot be recovered afterwards or the void between adjacent beam webs can be filled with an expanded polystyrene. The precast beam concrete has a specified strength of 7,500 psi at 28 days, while the deck concrete is 5,000 psi at 28 days. The heaviest precast element, the 35 in. deep section, weighs 334 plf and thus is light enough to be easily transported and erected.

Following testing to validate the section strength and overall behavior, the Nebraska Department of Roads adopted the IT system as a standard for new or replacement bridges. The system has been successfully used on projects in Nebraska as well as in Iowa, Kansas and Florida. With regards to economics, the construction costs of a IT system bridge was determined to cost 20% less than a three-span continuous concrete slab bridge. Due to longer span capability, additional savings are possible due to a reduction in the number of substructure units required.

Several proprietary precast concrete bridge replacement systems are described herein. Discussion of these systems is not intended to be an endorsement or preference for these structures. The discussion is intended to illustrate the options available for large scale precast concrete structures, and discuss the pre-engineered aspect of the products.

Cretex Concrete Products – Midwest (Cretex-Midwest, 2002) makes a prestressed quad tee section that has been used extensively on Iowa LVRs. All units are 3 ft – 11 ¾ in wide, thus the width of bridge can be increased or decreased by the number of units erected. Adjacent units are connected by field welding a connecting bar to the weld ties which are cast in the units. After the welding is completed, the shear keys in the units are grouted. Seven different lengths of the precast units are available ranging from 21 to 51 ft.

The Bebo system of precast concrete bridges is a product intended for bridge replacements over streams and small roads. A complete product package as well as a CDROM including design and installation information is available (Bebo 2000). Available in various spans from 12 to 84 ft, the arches come in various forms. Circular arch bridges are produced with spans of between 30 and 42 ft and rises between 11.5 to

26.5 ft. The hydraulic areas of these sections are large however, which results in a high profile structure. Elliptical shaped arches which have span capabilities of up to 84 ft with relatively flat span to depth ratios are also available. For spans of up to 48 ft, the structure is a single piece arch while for longer spans the structure is precast in two halves which are connected in the field with a CIP closure joint. The arches are typically designed for HS 25 loading but can be designed for special live loads if required. The standard arches can accommodate fill heights of 1.5 to 15 ft.

The structures are generally easy to construct on small spread footings (grade beams) or pile supported grade beams in the case of poor soils conditions. Once the footings are constructed, the arches are erected on the footing against each other to form the desired roadway width. No post-tensioning or mechanical connection is required between adjacent sections. Depending on the span of the arch, the length of the segments ranges from 4 to 8 ft. to limit their self weight and to simplify moving and installation. In addition to the standard precast concrete arches, precast spandrel walls and wingwalls are also used. The spandrel wall, arches and wing walls interlock and support each other. Since the arches require interaction between the structure and soil to derive their capacity, only well draining backfills with low plasticity are acceptable. Backfilling should generally be done symmetrically on both sides of the arch to balance the lateral loads; layers should be placed and compacted in layers not exceeding 1 ft. The hydraulic capacity of all of the Bebo structures is computed in accordance with FHWA HEC 5, *Hydraulic Design of Highway Culverts*.

In general, little needs to be done by a county interested in installing a Bebo span. The products are pre-engineered, site specific data are provided by the county to the precaster so that the desired structure can be fabricated. Additionally, a complete hydraulic evaluation is performed to assure hydraulics are adequate.

Another similar product is the Conspan system (Conspan 2000). Although similar in concept to the Bebo system, there are some differences in shape and span capabilities between the two systems. The Conspan structures are three-sided open structures with natural bottoms. The lowest portion of each "arch" is composed of straight sided walls while a series of compound circular curves form the rest of the structure. Conspan structures are available in a range of spans from 12 to 48 ft and rises ranging from 3 to 14 ft. For long spans, the arches can be placed in "series" creating multiple openings across the channel. The arches may also be placed on pedestal walls to increase their vertical underclearance. Like the Bebo system, the Conspan structures use precast spandrel walls and wingwalls but with different proprietary details. No post-tensioning is required between adjacent units; the standard design load is HS 25.

The units are typically set on a strip foundation with a keyway and grouted in place. Backfilling is strictly controlled in the critical backfill zone due to the need for soil-structure interaction requirements which are generally similar to the Bebo system. A series of charts are provided in the Conspan design manual for determining the strip footing size as a function of arch span, cover height, design live load and allowable bearing pressure. Lateral thrust is also computed for the various design cases. Sample design calculations are provided for determining the reinforcing required in strip footings.

Concerning hydraulic capacity, there are several approaches presented for the Conspan system. A free program is available for determining inlet and outlet control depths using FHWA culvert analysis procedures. Additionally, the FHWA HDS5 and

HY-8 procedures are applicable for analysis. The waterway area and wetted perimeter for culverts running full or at various partially full depths are given. Capacity curves relating culvert size, headwater depth and discharge, “Q”, are given for the standard products.

The Conspan manual provides details on the construction of skewed and curved structures made from the standard elements. The faces of the elements are typically beveled to accommodate the skew and /or curve. A number of project profiles are provided illustrating various applications of the systems. For a number of years, the construction of new or replacement bridges in the form of single or multi-cell culverts has been a popular choice. In order to help standardize the design, fabrication and construction of culvert structures, the ASTM maintains a standard for precast box sections, ASTM C1433/1433M.

The ASTM Standard includes a number of pre-designed box culvert sections designed to accommodate the AASHTO HS 20 loading, the Interstate loading or various earth loadings. Standard culvert designs are presented for single cell box structures as large as 12 ft x 12 ft with cover up to a depth of 18 ft. The required concrete strength and reinforcing size and spacings are presented for all of the design scenarios. A search of various precaster web sites indicates that the culverts commonly available around the country include the standard ASTM designs. The ready availability of such sections, their standard design and almost universal acceptance makes them an attractive choice for small to medium size stream crossings provided that a sufficient number of boxes, are used to accommodate anticipated flows.

5.4.2.2 Prefabricated Steel Bridges

Prefabricated steel bridges are not as common as precast concrete structures. They are typically available in some form of truss configuration. Though prefabricated trusses are usually be associated with temporary “Bailey Bridge” type applications, other types of trusses are available for temporary or permanent installations. Another prefabricated steel bridge system using traditional multi-stringer construction is also described below.

A number of fabricators of prefabricated bridge trusses were identified during the conduct of this study. These include Acrow (Acrow, n.d.), U.S. Bridge (U.S. Bridge, n.d.) and Wheeler Consolidated (Wheeler, n.d.). There are other truss manufacturers however the fabricators noted provided substantive information on the design, fabrication, construction and performance of steel truss structures.

The Acrow Panel bridge is a descendant of the Bailey Bridge developed for military use during WW II as a rapid bridge replacement system. The bridge is composed of three general components, all of which are stock items assembled as required to form bridges of various sizes. The truss is composed of a standard truss panel 10 ft. long, 7.2 ft. high and 6.5 in. wide. Numerous panels are joined together to create bridges of varying lengths. Maximum spans of 230 ft are available in configurations that support up to three lanes of HS 25 live load. With some restrictions in the number of lanes and/or live loading, simple span designs are tabulated for spans up to 250 ft. In order to accommodate these heavy loads and long spans, multiple trusses are used side-by-side. Although very long spans can be achieved, the standard trusses for LVR replacements are likely to be much shorter. Spanning between the trusses are similar standard floorbeams. The common decking is a prefabricated orthotropic panel that spans longitudinally between the floorbeams although other decks such as wood or steel grids can be accommodated. The system can either be used in through truss or deck truss configurations; truss top chords are stable in through truss configurations and do not need lateral support. All components are galvanized for weather resistance.

The U. S. Bridge system is similar in that prefabricated trusses are used, however, the method of fabrication and construction is significantly different. Whereas the Acrow system uses small prefabricated panels, sometimes several panels wide, and is field bolted to form a crossing, the U.S. Bridge System is primarily an all welded truss system. The only bolted connections on the trusses are where the prefabricated truss panel assemblies are joined in the field. Depending on the bridge size, several panels of the truss are welded together using conventional W-shapes for the truss members; the entire assembly is hot dip galvanized. Alternate materials such as weathering steel or painted trusses are available but the standard product is a galvanized truss.

The trusses are available in standard lengths of up to 150 ft. and in various widths up to three lanes wide. The trusses are through type with either parallel chords, or in the case of the longer spans, a curved top chord or “camelback” configuration. The typical deck system uses underslung floorbeams, simply supported stringers, and a deck system of galvanized corrugated deck pans with asphalt fill. Other decks such as traditional concrete filled pans or timber decks are permitted and can be accommodated.

A prefabricated steel bridge system with potential uses in LVRBs as well as in broader applications is the Inverset™ system (Fort Miller 1995). The Inverset system is a prefabricated bridge patented in the 1980’s that takes advantage of composite action and the use of rolled shapes to create prefabricated bridges that can be used in single or multi-span applications. The most innovative feature of the system is the method of fabrication.

There is a significant amount of the rolled shape section that is inefficiently utilized. This is due to the symmetry of the beam and the large compression flange. The Inverset system was developed to increase beam efficiency and also create more durable decks. The casting sequence involves creating a grid of the longitudinal stringers and intermediate diaphragms with the entire unit fabricated upside down. The slab is cast using formwork which is hung from the beams. This method of construction produces compression in the eventual bottom flange and tension in the eventual top flange, which is opposite to normal construction techniques. Once the slab cures, a crane is used to invert the entire unit. The resulting condition is similar to shored composite construction; however, it has the additional benefit of the stringers being partially prestressed by the weight of wet concrete. Final stresses at erection are near zero in the bottom flange, are tensile in the top flange, and are compressive in the slab due to the inversion. The net compressive stress in the slab increases slab durability as it delays the onset of deck cracking. By having zero stress in the tension flange under dead load, the section is much more efficiently utilized for live loading, the dominant load in short to medium span bridges. The net result is that either heavier design loads can be carried per stringer, as compared to a typical field cast unshored structure, or stringers smaller than required in typical construction can be used. Either way, greater efficiency is obtained. Additional external prestress force in addition to self weight can be applied to the system during casting resulting in an even greater stringer efficiency.

A typical unit consists of two stringers spaced at a desired distance with cantilever slab projections of 18 in. The units are placed adjacent to each other in the field and connected with diaphragms. The overhanging slabs, which have keys cast in their exposed edges, are grouted together with non-shrink grout. The combination of a fixed overhang distance with a user specified beam spacing results in an uneven beam spacing in the completed bridge. Various skew, horizontal and vertical curvature alignments can be accommodated in the casting process. Additionally, the units can be used in multi-span construction with compression or strip seals at the piers and abutments or can be made continuous for live load by casting a field closure pour which connects the ends of units in adjacent spans. This method is analogous to that used to construct prestressed concrete structures poured continuous for live load. These units can also be used to replace a deteriorated floor system in truss or through girder bridge with the units spanning transverse to the direction of traffic.

The design guide for the Inverset system includes a design example of a typical single span bridge, discussions of the various options for meeting project specific geometric constraints, lists shipping and installation procedures, details the typical materials and construction features used with regard to anticipated durability. Regarding installation and erection, the width and length of typical units results in crane picks that are manageable with readily available equipment. In the example bridge, three units (each weighing 55 kips) are required in a bridge which has a total width of 26 ft. and a span length of 55 ft. A single crane is typically used for loads and units of this size though the bridge could be slid from one abutment to the other on slider beams and lifted at each end with two smaller cranes. A typical installation is described as requiring a crew of five or six with the delivery, rigging, lifting and placing of each unit taking only approximately one hour; shorter times are possible depending on logistics and site constraints.

5.4.2.3 Prefabricated Timber Bridges

Modern timber bridges are much different than their older sawn timber counterparts. They typically are constructed with engineered lumber of some form (glued laminated, laminated veneer lumber (LVL), or parallel strand lumber (PSL)), are connected for enhanced load distribution and performance (by use of spreader beams or transverse post-tensioning) and are almost universally pressure treated for enhanced durability. Additionally, they are usually prefabricated. Examples include the use of prefabricated timber slabs for use in slab-on-beam construction, longitudinal slabs for use in short span bridge replacements, glulaminated rectangular beams for multi-stringer construction, laminated veneer or PSL T-beams or box beams, and other novel forms such as glulaminated arches for longer spans.

The development of modern timber bridges has been significantly advanced in the past 15 years by the Timber Bridge Initiative and its successor the Wood in Transportation Program. These programs have resulted in the production of numerous design aids for timber bridges, the development of specifications and standard design procedures for various forms of sawn and engineered timber and have promoted the development of modern timber bridges. As a result of these programs, there is an extensive database on the field performance of various timber structures including load test results. One of the other advancements by the timber bridge set aside programs has been in the area of timber bridge railings for use on timber and concrete deck bridges. These railings come in various configurations and have been tested to different crash test levels. Information on many of these developments is readily available. The single most comprehensive resource for information on timber bridge design, construction and performance is a CD-ROM including approximately 220 electronic documents (NWIT 2001). Several references included on this CD as well as other sources of timber bridge information are discussed in the following paragraphs.

Brungraber et al (1987) discussed the state of the timber bridge population in rural America as well as the prospects for increased usage of timber bridges as viable replacement options. Though the statistics are somewhat dated due to the time of this publication, the reference still provides valuable insights into the problems and challenges in managing rural bridge populations. It also documents the importance of rural roads and bridges on the overall economy.

At the time of publication, Brungraber et al, noted the number of timber bridges in the United States was approximately 65,000. These bridges were primarily in the Midwest and South Central portions of the United States and off the federal-aid system. As of August 2000, the NBI data indicated only 34,541 timber bridges, approximately half the number 15 years earlier. This indicates a rapid decline in the number of timber bridges and that these bridges are being replaced by other types of structures in spite of the many advancements in timber bridge technology in this time period.

The Brungraber study cites the advantages for modern timber bridges as being logistical, performance and economic related. Logistic benefits involve the ease of fabrication, shipment and construction of timber bridges. They are typically small bridges, easily shipped and installed with small construction crews with average training. Timber bridges can be installed in adverse weather conditions as temperature extremes have no bearing on construction. The performance benefits include the excellent resistance of timber deck panels and timber railings to deicing salts and/or inclement weather.

The economic benefits of timber bridge construction are site specific and regionally variable. In the Midwest where timber bridges have an established base, their economics are a tangible benefit while in high traffic areas, they are not as common. Another economic advantage of timber bridges is that due to their light weight they can be used on old substructures, and can be more readily fabricated and installed by local forces.

An examination of the underlying reasons for the past poor performance of timber bridges is presented in Smith and Stanfill-McMillan (1996). A survey of timber bridge perceptions and performance in four states, Mississippi, Virginia, Washington and Wisconsin, was conducted. One objective of the research was to examine whether the high percentage of timber bridges considered deficient was due to the performance of the timber itself or other reasons such as roadway deficiencies, waterway inadequacies, or substructure conditions.

The survey concluded that timber bridges have a perceived inadequacy when compared to other types of bridges. Analysis of the NBI data for the four states indicated that Mississippi had the greatest number of timber bridges as well as the most deficient timber bridges. The state does not have a standard design process nor standard plans for timber bridges. The same can be said of Virginia and Washington. The only state with standard design plans for timber bridges among the study group was Wisconsin. The perception and documented performance of timber bridges in Wisconsin is much better than in the other surveyed states. More than 80% of Wisconsin's timber bridges have a satisfactory rating which the authors imply is largely due to the use of standard design rules and plans.

Considering timber bridges in general, the performance of timber bridges was demonstrably better for bridges on state and federal road systems than for those on local roads. This again is hypothesized to be due to lack of consistent design standards for off-system locally owned bridges and variable maintenance on these bridges. Timber stringer bridges are the most common type of timber bridge and are largely deficient while timber slab bridges, the second most common, are considered satisfactory over 85% of the time. Analysis of the rating of timber bridge as Structurally Deficient (SD) indicates that poor performance of the deck or superstructure is only the reason timber bridges are considered SD 11% of the time. Low SD ratings for timber bridges are primarily from substructure deficiencies (20%) and from inadequate structural or waterway capacity (39%).

In the states with no timber bridge design standards, the vast majority of bridges are designed for HS15 live loads or less, and in many cases the design load is unknown. Over 90% of the bridges with these low design loads in each of the three states are considered deficient. Conversely, in Wisconsin, the only state with standards for timber bridges, the satisfactory ratings of timber bridges are excellent with the satisfaction rating for bridges designed for less than HS 15 being 61%; for bridges designed for at least H 20 the satisfaction rating is 94%. This implies a very strong correlation between eventual performance as judged by NBI criteria and the existence of minimum design standards.

Ritter et al., (1996) report on the design, construction and testing of several LVL T-beam bridges; details are provided for six separate structures. The LVL T-beams are made of thin wood veneers (individual laminates measure between 0.10 to 0.25 in.) where the individual veneers are all oriented in the same direction and then glued together to form a beam. LVL T-beams can either be fabricated in a single T-shaped section (these shapes can also be formed using PSL) or can be fashioned into a T-section by placing slabs between adjacent solid rectangular beams and transversely post-tensioning the entire assembly together. Instead of the deck resting on top of the beams, it is compressed between the tops of the beam webs.

The concept for these bridges was pioneered by Trus Joist Macmillan in the late 1980's, and by the early 1990's about 20 of these bridges had been constructed in the Midwest and western United States. An obstacle to greater acceptance was the lack of AASHTO standards (which have now been created) for the design of such bridges. The objective of the Ritter et al. study was to assess the performance of some of the existing bridges. The bridges ranged in length from 26 to 44 ft and had widths ranging from 16 to 37.5 ft. Beam sizes depended on the particular application and box beams were used as fascia stringers on two bridges to improve their stability. The bridges were similarly transversely prestressed with high strength steel rods, were pressure treated and with one exception had asphalt wearing surfaces. Evaluation of the six bridges indicated that the bridges were performing well. There was no observed deterioration, however, there was a slight loss of prestressing; however, it was not significant enough to compromise the performance of the bridges.

Based on the success of the LVL T-beam bridges, similar prefabricated beams have been developed using PSL technology. An example of these beams are those developed again by the Trus Joist Macmillan Company (Trus Joist Macmillan, n.d.). The PSL beams start first with thin veneers that are split into thin fibers. These fibers are then aligned longitudinally and pressed, with adhesive, into rectangular billets which are then assembled with adhesive into T-shaped beams of various width, depths and widths of flanges. Standard designs use fabricated elements measuring 2 ft wide; beam depth is a function of span length. Pre-engineered designs are available for beams up to 66 ft long designed for up to HS 25 loading.

Wacker et al (1997) discuss the design and construction of a timber box beam bridge in Spearfish, SD. The box beam bridge was composed of glulam timber webs while the top and bottom flanges were composed of multiple vertical sawn lumber elements stacked next to each other. The Southern Pine glulam webs and Ponderosa Pine sawn timber flanges (made of nominal 2 x 6 in. timbers) are first glued together into modules each having three webs and two interior flanges. The modules are then post-tensioned together to form a continuous unit. The completed bridge was 65 ft long, 39 ft wide, and the boxes were 31.5 in. deep. A total of six prefabricated modules were used to build the entire bridge. Superstructure construction was completed in a single day including stressing. The bridge was retensioned three and seven weeks following construction due to relaxation of the tendons and creep of the timber. Loss of post-tensioning has continued to be a problem for this bridge as it has been retensioned two additional times, one year and three years after completion. It is hypothesized that this is due to moisture loss in the sawn timber flanges and stress relaxation in the timbers. This is not unusual for stressed timber bridges and can be corrected with periodic checks with retensioning as necessary. The bridge has been load tested and its behavior is linear elastic; measured deflections and overall performance are as expected.

Research was undertaken in Iowa to demonstrate the feasibility of using locally available timber resources, in this case pressure treated cottonwood, in the construction of economical bridge replacements. The concept of exploring the use of local native materials in timber bridge construction is an outgrowth of the Timber Bridge Initiative enacted by Congress in 1988. The construction and testing of several solid deck cottonwood bridges constructed in southern Iowa is discussed in Lee and Ritter (1997). In general, several years after construction, the bridges were found to be in good condition, and load test results are consistent with the performance of stress-laminated bridges made of other species. The broader conclusion is that native materials of various timber species and grades can be adapted to the design and construction of engineered timber bridges for county bridge replacements. The inexpensive local materials, coupled with county labor forces, result in inexpensive bridges.

The reconfigured Wood in Transportation (WIT) Program (Cesa et al) is an extension of the Timber Bridge Initiative program. The WIT Program has three main goals: (1) demonstration projects, (2) research and (3) technology transfer. The demonstration project portion of the program is directed at promoting economy of scale. By focusing on economy of scale and refinement of concepts to a commercial status, the broader objective of developing a sustainable class of bridge construction is advanced. With an annual budget of less than $2 M per year for the time period of 1996 – 2000, there was limited opportunity for the WIT program to fully fund projects. The resources were redirected toward commercialization projects where the USDA Forest Service assists local entities in the design of structurally adequate and economical structures to demonstrate the viability of wood as a transportation material. Examples of commercialized projects are the replacement of several similar bridges where the same engineers, contractors, material suppliers, etc. are involved in all bridges. Cesa et al discusses three such commercialized projects.

Ida County, Iowa received a $124,500 grant from the USDA WIT Program to fund the replacement of five deficient bridges using locally available cottonwood. Four of the bridges use cottonwood decks on recycled salvage steel stringers while the fifth is an all-cottonwood structure. The structures were designed and mostly constructed by county crews; this was the first time the crews had ever constructed a new bridge. The structures ranged in length from 29 to 47 ft, had a roadway width of 24 ft and were designed for AASHTO HS 20 loading. The abutments consisted of gabion baskets filled with stone installed by the county forces. By using recycled steel beams, the cost of the bridges was kept low, the cost for the first bridge being only $61,539, or $26.70/ft^2.

These two commercialization projects demonstrated a concept used in several projects so that lessons can be learned, economics of scale realized and various deficient structures remedied. In addition to research into timber bridge systems, extensive work has been carried out in the area of developing low-cost crash tested timber bridge railings for use on timber bridges. Some of this work is described in the following paragraph.

As of 1990, a total of 47 bridge railings had been successfully crash tested and approved by the FHWA for use on federal-aid projects, only one of which was for attachment to a timber deck. Recognizing the trend towards crash tested railing systems, Faller, et al, (1999) and Ritter, et al. (1995) summarize several years of development of cost effective crash tested railing systems intended for use on longitudinally and transversely laminated timber deck bridges.

In Ritter, et al. (1995), the research objective was to develop five crash tested rails, three meeting the AASHTO PL-1 criteria, one meeting the AASHTO PL-2 criteria and the last meeting the NCHRP 350 TL-4 criteria. With the given criteria, post and rail details were developed for the identified scenarios. All of the systems have several common features, particularly the connection to the deck. The attachment of the post, which is outside of the deck and not on top of it, consists of threaded steel rods inserted through bore holes in the deck and anchored some distance away from the edge of the deck in a routed pocket.

For the PL-1 criteria, the three rails developed consist of two all timber options, a timber rail and timber post with a curb and a timber rail with timber post without curb, as well as a w-beam post with spacer block and steel thrie beam railing. All three rails were tested successfully to the specified test criteria. For the PL-2 level, a single rail system was tested, essentially a slightly strengthened version of the thrie beam railing tested at the PL-1 level. Some localized stiffening and strengthening was all that was required to upgrade the railing to the PL-2 level. Finally, the TL-4 railing was a timber railing system with upgraded posts, railing section, and additional attachments through the curb section connecting the curb to the bridge deck. Costs for the PL-2 steel railing and the glulam TL-4 railing are also presented. The steel thrie beam railing material costs $53/ft while the glulam timber railing material costs were $108/ft. Additionally, vehicle repair costs were higher for the glulam railing as were the anticipated repair costs.

The results of this testing indicate that cost-effective crash tested railings exist for timber bridge structures. The PL-1, PL-2 and TL-4 crash tested rails discussed are those that appear in the *Plans for Crash-Tested Bridge Railings for Longitudinal Wood Decks* published by the Forest Products Laboratory of the U.S. Department of Agriculture (Ritter et al., 1995).

Faller et al., (1999) summarize the development of eleven cost effective crash tested railing systems intended for use on longitudinally and transversely laminated timber deck bridges. For longitudinal wood decks, nine crashworthy rails were developed, five of which were described previously. The additional four railings include three railings tested to the TL-1 level and one railing tested for low volume forest roads at a level below the test requirements for TL-1. For the transverse panel bridge decks, two rail systems were tested to the TL-2 level and an additional two to the TL-4 level. For the development of all of the railings, glulam decks were used as the base structure.

Subsequent modification of the same railing systems but for attachment to concrete decks was done. These modified details are published as *Plans for Crash-Tested Wood Bridge Railings for Concrete Decks* (Ritter et al., 1998).

A study by the USDA Forest Service documenting the costs of timber bridges constructed from 1989 – 1995 is presented in National Wood in Transportation Information Center (1996). At the time the report was prepared, cost data for 112 vehicular bridges were available; over half of the bridges were from three states, West Virginia (46 bridges), Pennsylvania (10 bridges) and New York (9 bridges). The remaining bridges were scattered throughout 23 other states.

With respect to structure type, the least expensive bridge type is a dowel laminated structure which costs $43.97/ft^2 followed by laminated veneer lumber (LVL) / parallel strand lumber (PSL) T-beam bridges which has an average cost of $44.15/ft^2. The next least costly bridges are longitudinal glulam bridges with an average cost of $47.57/ft^2. The cost of stress laminated deck bridges is $51.07/ft^2 followed by transverse glulam decks over glulam stringers at a cost of $53.02/ft^2. The most expensive bridge types are stressed box and stressed T type bridges with costs of $64.83 and $68.18/ft^2, respectively. Cost by structure type tends to indicate that the dowel laminated bridges and structures using LVL and PSL lumber are the most cost effective.

5.4.3 Bridge Recycling

An important "tool" employed by bridge owners and maintenance crews is the concept of bridge recycling. Frequently as part of a reconstruction project, many structurally sound bridge components can be saved. Various components (bridge beams or trusses, deck components, railing hardware, bridge bearings, etc.) can be saved, rehabilitated and reused in either already known locations (i.e. planned recycling) or simply stocked for a "rainy day". With budget constraints and the innovative thinking of county maintenance forces, bridge recycling can have a significant impact on local bridge populations.

A variation of the recycling theme is the construction of bridges using recycled components not originally used in bridge structures. In "Canada Puts the Squeeze on Its Trees" (1997), a different type of recycling is described. As part of the routine replacement of timber utility poles in Canada, hundreds of thousands of timber poles are replaced annually. A system has been devised to recycle these discarded poles into bridges for use on low volume roads.

First, the discarded timbers are trimmed on two opposing sides so that they can be positioned next to each other. The timber poles are drilled so that lateral post-tensioning can be installed thus creating a transversely stressed log bridge. The post-tensioning is composed of FRP tendons. The stressed log bridges cost approximately $20,000 each, which the authors cite as approximately half the cost of typical Canadian bridge replacement projects using steel and concrete. The quality of the timber in utility poles is superior to sawn lumber and the poles typically have been or can be easily treated with preservatives. Obviously with the poles having a given capacity, their live load capacity is a function of the span length. Such a bridge may not be suited for all applications but the recycling of previously discarded materials into a new bridge structure is both environmentally ‘friendly’ and economical.

An illustration of component recycling in the construction of off-system bridges is discussed in Wipf et al., (1999). Unlike other forms of recycling where various bridge components are removed, stocked and reused in future construction, Wipf et al. discuss the use of recycled railroad flat cars in LVR bridges. Information on follow-up study and demonstration project (TR-444) was presented in Chapter 2.

5.4.4 Component Stockpiling

Component stockpiling is different from the storage of old items for potential reuse described in the previous section and involves the stockpiling of new bridge components for rapid replacement of damaged structures. Components could either be commercially available items such as precast concrete products, steel I-beams or H-piles, or could also include locally constructed items.

The construction of short span bridges using precast components manufactured and erected by county personnel is discussed in McLin (1990). Borrowing from a concept developed by the Oklahoma State University extension services, the Davies County, IN Highway Department built a set of forms that allows them to manufacture small bridge units in pieces that can then be assembled in the field to various widths.

The bridge system involves the precasting of a series of double-T units, 17 in. deep, 41.5 in. wide and 24 ft long. The precast units are then bolted together at the third points using a 1 in. diameter all thread rod. Additionally, the top flanges of the double T's are notched to allow for the pouring of a shear key in the field. The units which are designed for HS 20 loading have been used in side-by-side applications up to a bridge width of 24 ft. The outside beams are specially modified to allow for the installation of guiderails. The units themselves are lightweight, approximately 5.75 tons, thus allowing for their casting, movement and erection using small equipment owned by the county.

Because construction of each unit is simple and inexpensive, the county can cast units during winter seasons or other down times in anticipation of scheduled or emergency bridge replacements. Additionally, due to the quick turnaround time for casting, these units are an attractive option compared to commercially precast / prestressed concrete products.

5.4.5 Bridge Elimination

The focus of this chapter as previously noted was aimed at finding new and innovative ways of constructing low cost and low maintenance short span bridges to replace aging and deteriorated structures. However, a new bridge still has an initial cost and associated maintenance. An alternative is to replace a short span bridge over a low flow stream with a roadway. This concept, which is discussed in detail in Lohnes et al., (2001) (briefly reviewed in Chapter 2) should only be considered if the number of reoccurring floods is tolerable. For small streams, especially ephemeral streams with intermittent flow or low continuous flow, removal of the bridge may be a desirable solution.

Low Water Stream Crossings: Design and Construction Recommendations is a report authored by Lohnes et al. that documents the implementation of various types of low water stream crossings. The concept of low water stream crossings is based upon the idea that streams (or drainage ditches) with very low flows, or ones that are dry except for occasional flooding, may be effectively closed by filling in the old bridge opening with roadway embankment. Lohnes discusses three options for low water stream crossings: unvented ford, vented ford and low water bridges. A brief description of these options follows.

The unvented ford is essentially a dammed stream. Its application is appropriate for streams where the normal stream depth is less than 6 in. and where the proposed roadway crossing is less than 4 ft above the level of the streambed. The embankment may be constructed of crushed stone, riprap, precast concrete slabs or other suitable materials. The vented ford is a variation on the previous concept but is used for somewhat higher flows. In the vented ford, several vent pipes are placed in the embankment. For low flow conditions, the pipes remain above the normal stream elevation. For moderately higher flows, the pipes convey a portion of the flood waters; in high flow situations, the road will be temporarily overtopped. The last option is the low water bridge crossing. These bridges may be flat slabs or several other low profile options described in the aforementioned reference. The bridge structure allows for the greatest conveyance prior to overtopping but the roadway profile is still kept at a minimal freeboard over the stream and will be overtopped during some flood events.

The recommended site conditions for implementing low water stream crossings include roads that are generally unpaved (gravel or dirt), field access roads, roads with no inhabited dwellings, low volume roads, and roads with available detours. These criteria may be tailored by individual agencies. Additionally, the stream should have a stable channel, grade of approach roadways shall be less than 10%, height between the proposed roadway and stream bed should be less than 12 ft, costs should be compared to a bridge and the site should not be in an area where future development might require construction of a bridge. Following determination of the suitability of the site in general, Lohnes presents design procedures for the three crossing types.

Also included in this report are recommendations for appropriate traffic signing in the vicinity of the potentially flooded “bridge” and a discussion of legal concerns about such a type of construction. Of the 225 low water stream crossings in Iowa, some in service for over 20 years, only three legal claims have been filed. In two cases involving crashes, the counties were absolved and in the third case the issue concerned a right-of-way issue and not the crossing itself. It is advised, however, that a stated design criteria be established as a minimal protection against tort liability.

5.4.6 Standard Plans

The use of standard plans has the potential of reducing the total cost of bridge replacements by minimizing required engineering. Equally as important if not more so, is the construction of various standardized components that have been performance tested over time. The combination of minimization of engineering and construction familiarity reduces costs.

A number of concepts for standardization are presented in the following sections. They vary in their actual presentation from pre-engineered complete systems to more generic design aids.

5.4.6.1 FHWA Standard Plans

In the past, design tools used by both state and local agencies were the FHWA's *Standard Plans for Highway Bridges* (FHWA 1976, 1979, 1982, 1984). Published in several volumes and including designs and details for various types of bridges including concrete, steel and timber simple span bridges as well as LFD designs for continuous concrete slabs, prestressed concrete I–beams, steel rolled shapes and steel plate girders - the plans were intended to be used as a guide in the development of local standards. Though no longer published by the FHWA due to the availability of state standards and the difficulty in keeping the plans updated for code changes, these plans are likely still available at various DOTs and consulting firms and can still be of value in the construction of LVR bridges. The preface to the plan sets states the following:

> "These plans are intended to serve as a useful guide to state, county, and local highway departments in the development of suitable and economical bridge designs for primary, secondary, and urban highways. The plans

> should be particularly valuable to the smaller highway departments with limited engineering staffs."

With the above description, it appears clear that the plans may be of some use in the design and construction of LVR bridges. There are definite engineering issues that need to be addressed with some of the plans due to changes in design specifications through the years; there are also details presented that are no longer used due to their obsolescence or inefficiency. However, for some of the bridges in the plans, the general design does not vary significantly from modern designs. In that context, the plans can be used to evaluate the feasibility of various bridge options and to determine relative cost between bridge types; these plans can also be used for preliminary designs. Users of the FHWA plans should use them as no more than a guide and preliminary engineering tool due to their obsolescence. Consultation with the Iowa DOT Office of Structures may also reveal the existence of bridge standards very similar to the FHWA plans but updated to current codes. Also, various industries such as the steel, timber and concrete bridge trade associations (AISI, AITC, USDA FPL, CRSI, PCA, PCI, etc.) have published their own plans in recent years similar to the FHWA plans but with updated details and modern design criteria. Some of these design aids are briefly discussed in this report as well.

Volume I of the plans (FHWA 1976) contains six sets of standard plans for various concrete superstructures. Included are designs for cast-in-place T-beam bridges in three span lengths: 30, 40 and 50 ft and two deck widths: 28 and 44 ft. In addition, cast-in-place box girder designs are provided for longer spans: 80, 100 and 120 ft and the same deck widths. Precast reinforced concrete channel section designs are also given for short span bridges with spans of 20, 25 and 30 ft. These beams could easily be fabricated by county crews and stocked for emergency use or cast in slow seasons for later use. The units are light, with the maximum piece weighing 14,690 lbs., and could be easily placed using small equipment. Additional designs are included for various precast pre-tensioned and post-tensioned voided slabs, box sections and I-beams. Miscellaneous details for bearings and expansion joints are also presented. Plans for the 28 ft roadway were developed for an AASHTO H 15 loading while those for the wider 44 ft roadway are for a HS 20 loading.

Volume II (FHWA 1982) contains standard plans for structural steel bridges composed of rolled shapes or plate girders. All designs are for a minimum loading of HS 20. The rolled shape and slab bridge designs are for various simple span lengths ranging from 20 to 90 ft and bridge deck widths of 28 to 44 ft. Options are presented for both non-composite and composite beam designs with a concrete deck. Plate girder designs are given for spans in the 90 to 180 ft. range for simple span construction. The user of the standard steel bridge plans is cautioned that some of the rolled shape bridge girders listed in the design tables use welded cover plate details that are potentially fatigue prone as well as partial height diaphragm connection plates which are also strongly discouraged based on knowledge of the fatigue performance of these details. The designer is encouraged to either use a larger beam section without a cover plate or use cover plates which are bolted in the end regions. Although fatigue is not a likely concern on most LVR bridges, the potential user should be aware of the undesirable nature of some of the details in these plans.

Volume III of the FHWA plans (FHWA 1979) provides a series of sample designs for timber bridge structures including solid timber stringers, longitudinal laminated deck bridges, timber bridges with composite concrete decks and finally, laminated timber stringers with laminated deck panels. The bridges are all designed for a roadway width of 24 ft and various span lengths from 11 ft for the solid sawn timber stringer bridges to a maximum of 65 ft for the laminated stringer bridges with deck panels. In addition to the superstructure designs, designs and details for timber pile bent piers and timber pile / timber lagging abutments are also included. All of the timber bridge designs were based on design loads of either H 15 or H 20. Due to significant changes in timber design provisions of the AASHTO code and extensive research in the area of timber bridge design and construction since publication of the FHWA plans, the engineer is additionally directed to the various resources available from the American Institute of Timber Construction (AITC 1999) and the USDA Forest Products Laboratory (NWIT 2001) for additional guidance and code compliant designs including crash tested guardrails.

5.4.6.2 Iowa DOT County Road Bridge Standards

The Iowa DOT maintains an extensive set of bridge design standards that can easily expedite the creation of bridge plans. These plans include pre-engineered prestressed concrete I-beam bridges, standard drawings for integral and stub abutments, pre-designed continuous concrete slab bridges, barrier rails, and many predrawn transverse and longitudinal sections for various roadway geometries. The assembly of a completed plan set is a fairly easy process with the exception of the design of piers, abutment piles and quantity determination. In order to further expedite and standardize the design and construction of bridges on the county road system, the Iowa DOT also maintains a complete set of pre-engineered county bridge standards. These standard plans encompass a complete set of construction drawings for continuous concrete slab and prestressed concrete I-beam structures including various options for pier and abutment construction. Pre-engineered pile bent designs are also provided. A brief description of the various county bridge standards is presented in the following paragraphs.

The Iowa DOT J24 standard provides complete engineering including tabulated bridge quantities for a series of continuous concrete slab bridges with 24 ft roadway widths. The five pre-designed and detailed structures are all three span construction with the total bridge lengths ranging from 75 to 125 ft. Due to the size of these bridges, more than likely they would be contracted out. Thus, no additional details on these bridges are presented in this report. An extension of the J24 standards is the J30 series. The J30 plans encompass all of the same span length, skew, abutment and pier types as the J24 series but are intended to be used for bridges with a slightly wider roadway width, 30 ft.

A similar set of plans is provided for prestressed concrete I-beam bridges. Denoted the H24, H24S, H30 and H30S series of plans, these four sets of plans include complete designs, including piers and abutments for 24 ft and 30 ft wide roadway bridges. For the simple span bridges the span lengths vary from 30 to 80 ft. The three-span continuous designs have total bridge lengths ranging from 126 to 243 ft.

For the simple span bridge plans, the pre-designed abutment is a combination timber and concrete abutment structure. A single row of timber friction soldier piles is depicted extending up to just below the beam seat elevation. A concrete pile cap is poured to support the concrete I-beams. Timber lagging is placed between the driven piles to retain the fill. The timber wingwalls that are used are tied together with tie rods, and the abutment soldier piles are tied back to a deadman. For the continuous bridges, integral abutments are depicted. For shorter spans, timber piles may be used while for the longer three-span bridge designs, steel piles are required due to their vertical load capacity and their greater ability to accommodate the required lateral movements.

5.4.6.3 AISI Standard Plans

In response to a need for greater standardization of steel bridge design, the American Iron and Steel Institute has developed a series of standard plans for simply supported steel bridges (AISI 1998). The intent of the standards is to both standardize steel bridge design, and to promote steel structures as viable choices for short to medium span bridges.

The plans are generally organized by roadway width. Plans have been presented for roadway widths of 24, 28, 34, 40 and 44 ft and for total span lengths ranging from 20 to 120 ft. Depending on the number of beams and span length, various beam options are provided. These include non-composite and composite rolled beams without cover plates, composite rolled beams with welded or end-bolted cover plates, and composite plate girders with unstiffened or partially stiffened webs. The combination of beam spacing, span lengths, concrete deck types (normal or lightweight concrete) and beam types results in over 1,100 standard designs in the plan set. In addition to the design of stringers, the slab reinforcing is detailed as are shear stud size and spacing when required, stiffener and diaphragm connection plate sizes, cross frames or rolled shape diaphragms, pre-engineered elastomeric bearings and schematic details for jointless and integral substructures.

A decision tree is presented to aid in the selection from the various options relative to deck type, composite vs. non-composite design, rolled shape vs. plate girder, etc. The use of uncoated weathering steel is promoted as leading to both first cost and life cycle cost savings when its use is in accordance with the restrictions presented in the FHWA Technical Advisory T5140.22, *Uncoated Weathering Steel in Structures*.

5.4.6.4 Timber Bridge Standard Plans

The Wood In Transportation Program, a jointly funded cooperative research, development and technology transfer program supported and funded in part by the FHWA and the USDA Forest Products Laboratory has developed numerous design aids and standard plans for the implementation of modern timber bridges. These plans are briefly described in the following sections.

One of the first sets of standard plans developed were the *Standard Plans for Southern Pine Bridges* (Lee, et al., 1995). Standard designs are presented for three bridge types: stress-laminated sawn lumber slab bridges, stress-laminated glulam timber bridges and sawn lumber stinger bridges with transverse sawn lumber plank decks. Designs are presented for AASHTO standard loadings, HS 20 and 25.

For the stress-laminated sawn timber bridges, various design widths, (12 to 38 ft), and lengths, (10 to 20 ft) are presented. Construction of the bridges uses standard southern pine with nominal sizes ranging from 2 x 8 to 2 x 12 in. The stress-laminated glulam Southern Pine spans pick up where the sawn lumber spans leave off in terms of span length. The stress-laminated glulam spans range from 20 to 32 ft. in length and are available in the same deck widths. The panels are composed of a standard 24F-V3 Southern Pine combination with the deck thickness selected in accordance with the span length. Similar to the sawn lumber laminated bridge, design criteria and calculations are provided as backup for the standard plans and as a teaching aid for those unfamiliar with glulam stress laminated timber slab bridges.

The final bridge type is a series of longitudinal sawn lumber stringers with transverse timber decking placed flatwise across the deck with an asphalt wearing surface for protection. Depending on the size of the stringers, lumber grade and the stringer spacing, simple spans of up to approximately 23 ft are possible. Railing and curb options are provided for all three bridge types, as are fabrication details.

A refinement and extension of these plans is presented in the *Standard Plans for Timber Bridge Superstructures* (Wacker and Smith 2001). The plans have been developed in conjunction with several government agencies as well as with commercial partners to provide simplified designs of timber bridges and bridge components. Included in the plans are seven types of bridge superstructures, five longitudinal deck systems and two beam systems. The designs are prepared in accordance with the AASHTO Standard Specifications for design loads of HS 20 or HS 25.

The Standard Plans are general so that bridges of various widths, lengths and span configurations can be constructed using various wood species; they are no longer restricted to Southern Pine. Design options are presented for the following bridge types: nail-laminated decks, spike-laminated decks, stress-laminated sawn lumber decks, stress-laminated glulam decks, and longitudinal glulam decks, as well as for glulam stringers with transverse glulam decks and transverse glulam decks for steel stringer bridges. For these bridge types, deck type bridges range from 10 to 58 ft with bridge roadway widths from 12 to 36 ft. For the glulam timber decks on steel beams, there is no stipulation of bridge length since it is a function of the stringer capacity.

In general, the Standard Plans allow for the use of any of the woods listed in the AASHTO Specifications. For each bridge type, with the exception of the glulam stringers with glulam decks, minimum required bending properties are listed as a function of the span length and the governing deflection criteria. Any species listed in AASHTO and meeting the allowable bending stress and minimum required Modulus of Elasticity provided for a particular design can be used. For the glulam stringer and glulam deck bridges, the designs presuppose the use of western species or southern pine and the appropriate width and depth standard combinations are listed for each of these materials. Additional details provided include:

- the layout and force requirements for transverse stressing bars for stress-laminated construction,
- location and size of transverse stiffener beams for multiple panel bridges,
- the steel or wood diaphragm layouts for glulam stringer bridges,
- the substructure connection details,
- the asphalt wearing course details, and
- references to other Forest Service Plans for various bridge railing options for both longitudinally and transversely laminated deck panels.

5.4.7 Additional Design Aids

As a supplement to the use of standard plans is some additional information (i.e. design aids) that is also presented in this report. These design aids include handbooks illustrating the design of various bridge types and design examples prepared by various industries indicating the efficient use of various materials in the construction of economical and durable bridges. In the following sections, these design aids are tabulated and discussed by material type.

5.4.7.1 Concrete Bridges

The Concrete Reinforcing Steel Institute (CRSI) has a series of design aids for use in the design and construction of off-system bridges. These design aids are available in both published form and in the form of computer programs for the design of bridges.

In *A New Look at Short Span Reinforced Concrete Bridges* (CRSI 1983), a series of guidelines and pre-designed, cast-in-place reinforced concrete short span bridge structures are presented. Though the economic data and some of the detailing presented are somewhat dated, the designs presented are still reasonable standard designs for purposes of preliminary designs and in some cases final designs. The designs presented are intended to assist the engineer in selecting the appropriate balance between span length and substructure cost and in determining reasonable costs. The parameters of the designs considered a roadway consisting of two 11 ft lanes with 6 ft shoulders and concrete parapets. Additionally, details were developed for bridges having total lengths up to 130 ft with individual span lengths up to 40 ft. It is noted that the designs are based on a minimum AASHTO loading of HS20 since it was determined that designing for a lesser load of HS15 resulted in a material savings of less than 2% of the total bridge cost. The lesser design load is also likely to lead to future load capacity problems.

Flat slab bridges are pre-engineered for three typical span arrangements, all having a balanced span arrangement with span lengths, 0.8L – 1.0L – 0.8L, which are typically used to economize three-span structures. The slabs have drop panel caps at the piers and are detailed with or without haunches at the piers. Cost estimate tables as well as total quantities are provided so that engineers can use their own unit costs for concrete, reinforcing steel and formwork to estimate the cost of the structure.

Designs for three multi-stringer bridges are also presented, with spans of 20, 30 and 40 ft. These simple spans can be used in a multi-span arrangement with the bridge behaving as a series of simply supported spans. These designs detail the use of rectangular concrete beams with notched top flanges that can support transverse deck panels. The design allows for either cast-in-place construction, precasting of the reinforced concrete elements, or placement of precast concrete stay-in-place forms and casting of the deck. The flexibility allows owners and contractors to minimize erection time and maximize the use of forms by precasting the beams remotely or on site. If desired, several sets of forms can be built to minimize total construction time. Again, quantities and details are presented for the several sample designs.

For stream crossings where, due to stream width and berm lengths, a two-span bridge is required, an alternate solution is suggested that uses tall wingwalls and abutments in conjunction with a shorter main span. Obviously, the cost of tall wings and abutments needs to be compared with the alternative with short abutments, two spans and a pier in the stream. Hydraulics are an important consideration when considering this possibility since the waterway opening will be significantly reduced with the tall wings.

In addition to the standardized superstructure options presented, a series of standard substructure designs are included for use in conjunction with the pre-engineered superstructures. Bridge piers both with and without a cap beam are illustrated. For the piers without a cap beam, the column reinforcing terminates within the confines of the bridge slab which makes the piers integral with the superstructure. In addition to pier spacing, and column/cap detailing, sample footings on piles along with footing size and reinforcing are presented for the several example bridges. Options are also presented for a pile bent pier, a solid wall pier and for an option that uses site-precast concrete columns in conjunction with the site-precast stringer bridge system.

5.4.7.2 Steel Bridges

Other than the AISI Standard Plans previously discussed in this chapter, there are no other design aids to the authors' knowledge to expedite the design of steel bridges.

5.4.7.3 Timber Bridges

In addition to the various standard plans for timber bridges previously discussed, a number of timber design aids are available. Some of these come from timber product manufacturers and are in the form of design curves, charts, tables, etc. and some are from trade associations.

The American Institute of Timber Construction (AITC) publishes a guide that steps prospective timber bridge designers through the design of modern timber bridges and has additional information on various aspects of timber bridge design, fabrication and construction (AITC 1999). The main thrust of the AITC manual is the design of three bridge types: longitudinal deck bridges (without transverse post-tensioning), transversely post-tensioned timber slab bridges and glulam stringer and deck bridges. Design examples are presented for each of the three bridges. Additionally, tabulated designs are available for design loads of HS 15, 20 and 25 for each of these bridges.

The longitudinal deck bridge without transverse post-tensioning uses a spreader beam to tie the panels together and to assist in the load distribution which is similar to a design presented in the USDA FPL plans (Wacker and Smith 2001). Options are presented for the longitudinal decks that vary based on design live load and continuity (single span or multi-span continuous); when using the standard designs, one must make sure the species selected, after being modified for load duration, moisture condition, etc. meets the minimum bending stress, shear stress and Modulus of Elasticity specified. Once this has been accomplished, allowable spans are tabulated versus deck thickness. For this deck type, the maximum span for the heaviest live load, HS 25 is 26 ft for both simple and continuous designs. There is minimal increase in span length for lower design loads.

The longitudinal stringer bridge with transverse deck panels is an all-timber bridge with glulam products in all elements. The tabulated standard designs are based on Western Species and a 24F-V4 combination (a standard glulam beam as described by the NDS Specifications); Southern Pine sizes might be somewhat smaller than those tabulated. Again, a species must be selected that after application of the appropriate modifiers meets the minimum allowable stresses and material properties requirements. Span lengths up to 72 ft are listed for HS 25 live loading. For the lowest live loading HS 15, the maximum span listed is 80 ft – only 8 ft longer. Again, there is minimal benefit in using the smaller design load in terms of span length however larger stringer sizes are required for the heavier design load.

The final option is a stress-laminated glulam deck bridge. For this bridge type, tabulated options are listed for HS 20 and 25. In both cases, the maximum span is listed as 50 ft with a slightly thicker deck being required for the heavier live load condition. The required material properties and transverse stressing requirements are the same or essentially the same for either live load option. The designs for the stress-laminated bridge are based on Southern Pine; Western Species sizes would be slightly larger.

5.4.7.4 Other Sources of Information

A number of additional resources are available for bridge owners that may be useful in the design and construction (as well as maintenance and rehabilitation) of various types of off-system bridges. Although not specifically described herein, nor included in the reference list, a vast number of electronic and print resources are readily available that provide guidance in everything from foundation design, bridge inspection and rehabilitation, bridge hydraulics, and other areas. One of the most important sources of information are the state LTAP centers. Many of these centers maintain extensive lending libraries of print and electronic media freely available for loan. Additionally, staff engineers are available to provide advice and assistance on various topics. Three significant online sources of information have also been found during the course of this investigation: The U.S. Army Corps of Engineers (ACOE), the U.S. Navy Facilities Command (NAVFAC) and the FHWA.

The ACOE has numerous design guides that are simply written yet comprehensive. They include references concerning the geotechnical and structural design of footings, sheet piles, cofferdams, and include manuals on fixed bridge design, inspection and rehabilitation. These manuals are readily available from the ACOE website. The NAVFAC Manuals are similar in coverage and are also easily downloaded and printed from the NAVFAC website. Finally, the FHWA has numerous online (and print) design references that are readily obtained. These include design guidelines for geotechnical and substructure design and a large number of Hydraulic Engineering

Circulars (i.e. HEC 18) related to stream and river hydraulics, hydrology, design for scour and others. Additional references are available from other Federal bureaus such as the Bureau of Reclamation which publishes an extremely comprehensive manual on the repair of reinforced concrete, which also may be downloaded. These references individually and collectively provide a significant base of reliable information for design engineers. With the substantial increase in the use of the Internet and Web for information, there are many additional references and sources of information that have not been identified in this report.

5.4.8 Software

The engineering community has obviously changed markedly through the years, one of these ways being the proliferation of software as a replacement for rote hand calculations. For routine calculations, automation increases both the speed and accuracy provided the software itself has been checked and verified by the user. For counties, in particular, their ability to engineer, produce plans, manage inventories, compute estimates, project schedules, and many other functions can be enhanced by automation. Software allows for the creation of rapid “what-ifs” in the selection of economical replacement bridges.

A brief description of some of the software tools available for counties follows. It is by no means an exhaustive summary since a large percentage of software presumably exists only in the hands of the owner/developer of such programs, spreadsheets, etc. The information presented herein was discovered during the literature review for this investigation.

Engineering software can take many forms; it may be commercially available programs developed for sale. This software is generally regularly updated and runs the gamut from general purpose structural analysis and design programs to very specific stand-alone programs. In addition to commercial software is the category of software developed by State agencies for use by in-house staff and consultants working on their behalf. This software is frequently available for download from a State website for free or for nominal charge. The final type of software is that which is most difficult to document is the proliferation of spreadsheets, custom programs, MathCAD sheets, etc. that are in use by various agencies and consultants throughout the country. Some of this information can be found online.

One of the trends noticed in this investigation was the amount of software available for free or nominal charge on the World Wide Web. A summary of the programs located in this investigation is presented in Appendix D. The vast majority of the software pertains to structural engineering calculations. However, a review of the material in Appendix D reveals that free software is also available for various other disciplines such as geotechnical engineering, hydraulics, coordinate geometry, among others. Software available from various FHWA web sites has been included and catalogued.

One source of software not discussed in Appendix D is the engineering software that is available from the Pennsylvania Department of Transportation (Penn DOT). A great deal of software has been developed through the years by the Department under contract with various consulting firms covering a great array of topics. These include bridge rating, steel and prestressed concrete girder analysis and design, abutment and wingwall design, elastomeric bearing design, pier design, box culvert analysis and design, floorbeam analysis and rating, bridge geometry programs and others. The programs are available in both AASHTO Standard Specification and LRFD versions and depending on the software version can work in either US or US and SI units. A complete list of software is available online from the PennDOT web site. It is conditionally free – that is it is available for free to other government agencies, including support, and is sold for a nominal fee to private industry.

5.4.8.1 Concrete Bridge Software

Among the software reviewed in this investigation is that available from the Concrete Reinforcing Steel Institute (CRSI). Two specific programs, Computer Program for Box Culvert Design and Optimization (CUDO) and Design of Continuous Reinforced Concrete Slab Superstructures for Bridges (SLABBRDG) were reviewed (CRSI 1986, 1993).

The CUDO program was developed for the analysis and design of c.i.p. box culverts having from one to five cells and under varying fill heights. The program is only for structural purposes; hydraulic analysis is done separately. Limitations of the program include minimum and maximum span lengths (of the slab) ranging from 5 to 25 ft though it is indicated that neither of these extremes are cost effective. The design loading can be any combination of three point loads that move across the structure or a standard (HS 20) vehicle.

The CRSI SLABBRDG program which is used for the design of continuous concrete slab bridges having between two and five spans was also reviewed. The program considers constant depth slabs, slabs with haunches and slabs with constant depth drop panels and uses a series of one to three moving loads (AASHTO Standard Specifications); lane loading is not accommodated. The program designs using the LFD design method with required serviceability checks for crack width and fatigue of reinforcing steel.

5.4.8.2 Steel Bridge Software

As a supplement to the AISI Standard Plans is the AISIBEAM software (AISIBEAM 2000). Presently limited to the design of single span bridges using the AASHTO Standard Specifications, the software is an extension of the standard plans and allows for the design of bridges of almost any width and length with user specified dimensions. Additionally, the program can be used to compute the Inventory and Operating ratings of existing steel structures. The software, as well as digitized versions of the standard plans are available for download for a 30-day free trial from AISI after which a nominal fee is charged.

The combination of the AISI standard plans and software provide significant reductions in engineering design effort, especially for agencies with no comparable steel beam design software. Additionally, they are an effective replacement for the older FHWA standard bridge plans and allow for several options for bridge replacements.

5.4.8.3 Alternatives Analysis and Life Cycle Cost Estimating

A tool that has become more frequently used in recent years in all areas of infrastructure and capital equipment management is that of life cycle costing - an accounting methodology for analyzing the implications of various financial decisions. Life cycle costing allows for a systematic examination of the various costs of a project including its initial cost, future maintenance expenses and their time of occurrence, as well as disposal and reconstruction costs. A software tool, the Bridge Life Cycle Costing program, Bridge LCC, has been developed by the National Institute of Standards and Technology (NIST) for life cycle costing of bridge structures. The program and manual are available for download free of charge from the NIST website. Originally developed as a tool to help evaluate the economic differences between conventional construction and innovative material projects, the program has a capability of assisting with financial decisions.

The fundamental operation of the program can occur in either a “Basic” or an “Advanced” mode. In Basic mode, the user enters data for both a base scenario and for up to five alternatives. The Basic mode is typically used when the first costs and future costs are known with some certainty; in this mode a standard financial analysis of the various alternatives, is completed and they are reduced to an equivalent first cost. In the Advanced mode, the costs need not be known to the same degree of certainty, in fact one of the strengths of the Advanced mode is its ability to model uncertainty in some or all of the cost items. Results of the analysis will then be reduced to probabilistic costs. This sensitivity analysis is valuable if the costs are not accurately known and can only be projected to be in a given range which is the case for unusual types of construction, new materials, or for the estimation of future maintenance or disposal costs.

The advanced analysis features allow for significant expansion of the programs’ capabilities. For instance, costs can be entered for commonly recognized elements (CoRE) such as deck, superstructure and substructure if known, in lieu of lump sum bridge costs. Cost items can be specifically broken down by several criteria. A number of costs can be created, all of which are associated with an event. For instance, costs for a repainting job can include blasting, containment, repainting and disposal, all of which are tied to the event of painting. Each of the component costs can be specified as well as their individual uncertainties. The cost can also be assigned to a specific bearer, i.e., the owner, user, or third party entity so that the cost to each bearer can be tracked. Graphs of life cycle costs by component, bearer or simply total cost can be displayed. A project may be deemed more or less attractive to an owner depending on what costs can be minimized and the percentage of the cost that must be covered by the owner.

In addition to material costs, the program has the ability to model user costs, specifically user costs associated with lane closures and work zone impedances. Based on input data that can include speeds in the work zone, accident rates, driver costs, vehicle operating costs, accident costs, and others, rehabilitation and replacement options that require various work zone lane closures can be explored. The construction costs can be considered along with user costs in selecting an optimum solution. Again, one might select the least total cost project or examine alternatives that require greater construction funds but maximize safety and minimize user costs. Some of the cost items may be difficult to determine, specifically some of the lane closure and user costs. These may not be of great importance in many off-system bridge projects, however, the ability to systematically determine their influence on the overall financial picture is a valuable option for potential users of the software.

6. SUMMARY AND RESEARCH NEEDS

6.1 Summary

The overall objective of this phase (Phase I) of this project was to develop a reference that documents the state of practice in the area of maintenance/rehabilitation/strengthening and to address common problems encountered on the county bridge system. This reference report is intended to be a "user manual" or "tool box" of information, procedures and choices for LVR bridge owners to employ in the management of their bridge inventory.

Past Iowa DOT and Iowa HRB bridge related research projects were identified, reviewed, and summarized. In addition, literature reviews were performed to identify pertinent information related to LVR bridge design, rehabilitation, strengthening and replacement. Relatively detailed summaries of rehabilitation/strengthening methods were developed.

A questionnaire was sent to all Iowa county engineers to determine the various problems that are encountered on LVR bridges and their solutions to some of these problems. Fifty-two Iowa counties responded to the survey. A large percentage of the respondents indicated that they use in-house crews for bridge replacement or rehabilitation. A large part of the in-house work uses steel stringers and wood decks. Approximately one-half of the respondents indicated that they have experience with strengthening superstructure and substructure bridge elements; adding piling to the substructure was the most common response.

A questionnaire was also sent to other states to obtain similar information. The questionnaire was sent to state DOT's, county and local bridge owners, NACE members, and consultants involved with off-system bridge design and rehabilitation. Several hundred surveys were distributed electronically via e-mail. The response to the questionnaire was disappointing in that only a total of 20 states and 70 local agencies responded. One significant finding was that more appropriate decisions are required in all areas of bridge maintenance, rehabilitation, and replacement. “Data based” decisions through asset/bridge management as well as construction techniques, maintenance procedures, materials, etc. to promote extended bridge life are required. New high performance materials as well as FRP products are currently being researched. Several of these materials show promise for use in LVR bridges since they have excellent durability, require minimal maintenance, and appear to have long life.

Based on the evaluation of the information obtained from this study, a list of research needs was developed. The source of the research needs list was developed by evaluating the responses from the questionnaires, input from a research needs form held last year, and input from several county engineers. The research needs list should form the basis of a possible work plan to develop solutions for addressing LVR bridge problems.

6.2 Research Needs

As previously noted, the research needs list was compiled with input from three sources:

- Information obtained from the project questionnaire sent to all Iowa County Engineers.
- Information (pertinent to LVR bridges) from a structures research needs forum held last year.
- Input from conversations with several Iowa County Engineers.

This compilation constitutes a work plan for Phase II of this project. The research need statements have been numbered for identification and have not been prioritized.

6.2.1 Recommended Research Needs

The authors of this report have compiled a summary of pertinent research needs that could be of interest to Iowa County Engineers. This compilation constitutes a work plan for future consideration of the Iowa County Engineers.

1.) Develop inexpensive bridge replacements for county bridges that include standardized sets of prefabricated bridge details. Possibly neighboring counties could be organized so that duplication of formwork could be eliminated. County X could have the casting yard for pier caps, County Y could have the casting yard for the decking, County Z could have the casting yard for beams, etc.

2.) Identify cost-effective replacement techniques for small-span structures and develop design guides, design details and construction guidelines and procedures. For example, a composite timber stringer concrete deck (formed with reusable formwork) could be developed. For longer spans the timber could be reinforced with carbon fibers. This concept could be initially tested in the laboratory and then in demonstration projects.

3.) Identify cost-effective rehabilitation/strengthening techniques for common problems found on small span structures. This would also include the development of design guides, design details and construction guidelines and procedures.

4.) Develop techniques for load rating deteriorated timber substructures and develop associated repair/rehabilitation methods. This could include field-testing and associated analytical studies.

5.) Develop techniques for load rating deficient bridge superstructures. These techniques could include cost-effective load testing. Various “families” of bridges could be tested to provide strength and behavior data. This would be similar to the work completed in TR-440 where the strength and behavior of existing precast concrete deck bridges was determined.

6.) Develop low-cost procedures with associated design and construction guidelines for using FRP to strengthen existing deficient steel girder bridges.

7.) Develop techniques for converting jointed bridges to jointless.

8.) Prepare a comprehensive manual on maintenance practices for timber bridges.

9.) Develop a concise timber bridge manual similar to the current national timber bridge manual.

10.) Develop training courses on timber bridge inspection/rehabilitation.

11.) Develop design specifications specific for secondary road bridges.

12.) Evaluate the potential for use of on-road impoundments as a possible bridge replacement alternative.

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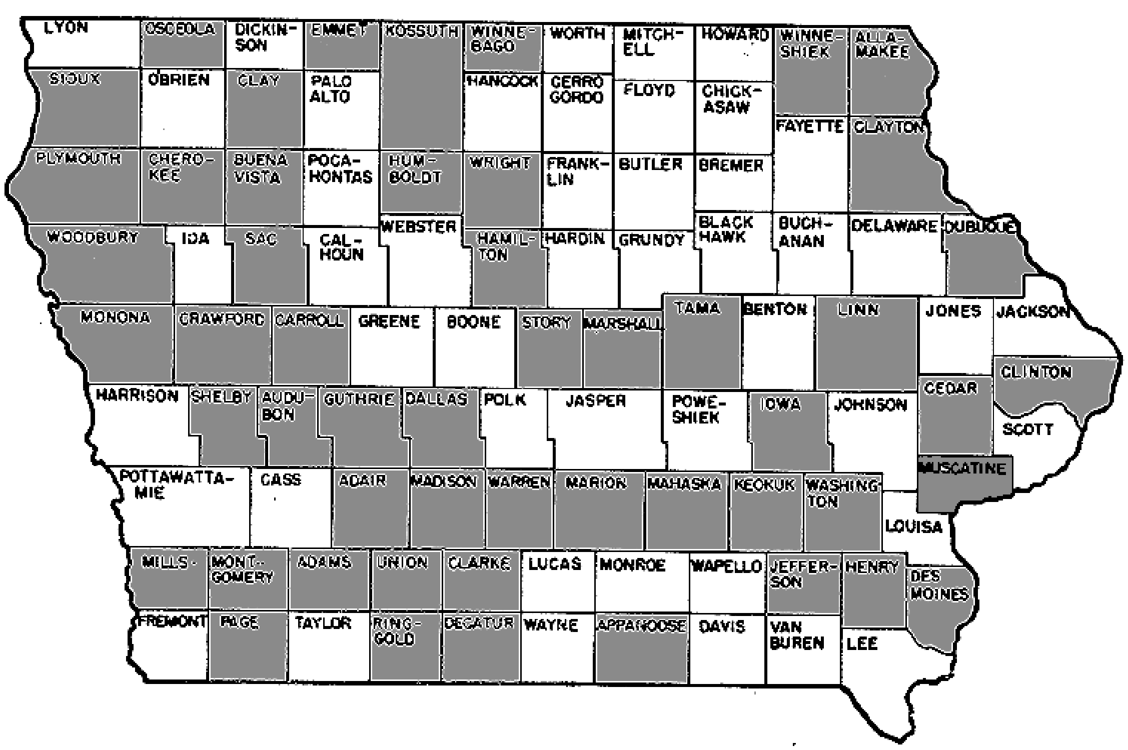
APPENDIX A

Iowa County Questionnaire

Responding Counties (52):

Adair	Adams	Allamakee
Appanoose	Audubon	Buena Vista
Carroll	Cedar	Cherokee
Clarke	Clay	Clayton
Clinton	Crawford	Dallas
Decatur	Des Moines	Dubuque
Emmet	Guthrie	Hamilton
Henry	Humboldt	Iowa
Jefferson	Keokuk	Kossuth
Lynn	Madison	Mahaska
Marion	Marshall	Mills
Montgomery	Monona	Muscatine
Osceola	Page	Plymouth
Ringgold	Sac	Sioux
Shelby	Story	Tama
Union	Warren	Washington
Winnebago	Winneshiek	Woodbury
Wright		



Summary:

Q-1) Does your county have experience with bridge rehabilitation?

Twelve respondents answered no. (Clay, Clayton, Des Moines, Guthrie, Hamilton, Humboldt, Monona, Muscatine, Mahaska, Story, Warren, Wright)

Q-2) Does your county use in-house crews to replace or rehabilitate deteriorated/inadequate bridges?

Thirty-four respondents answered yes, eighteen responded no. (Adams, Allamakee, Buena Vista, Clay, Clayton, Dallas, Des Moines, Emmet, Guthrie, Humboldt, Kossuth, Marion, Mahaska, Monona, Muscatine, Sioux, Winneshiek, Wright)

Q-3) IF you answered 'yes' to Q-2, what type of replacement superstructure(s) do you usually construct?

- Low water crossings (Washington)
- Replace bridge plank (Washington)
- Decks and approaches (Cherokee)
- Redecking (Dubuque)
- Deck overlay-Concrete, hot asphalt or cold patch (Mills)
- Rail reconstruction (Dubuque)
- Replaced or added beams (Dubuque)
- Timber approach spans (Montgomery)
- Wood stringers (Clarke, Page, Plymouth)
- Wood stringers and deck (Carroll, Madison, Osceola, Winnebago)
- Timber deck, piling, and stringers (Ringgold)
- Timber slab- 12" thick panels in 6' widths and 24' lengths (Lynn)
- Laminated wood and wood substructure (Union)
- Steel stringers (Montgomery, Page, Sac)
- Steel J-beams on old concrete abutments (Henry)
- Precast quad-tees on wood pile (Henry)
- Simple span steel beam (Warren)
- Steel stringers and wood deck (Carroll, Crawford, Madison, Cedar, Keokuk, Marshall, Osceola, Plymouth, Woodbury)
- Steel stringers with wood deck and wearing surface (Audubon, Shelby)
- Steel stringers and deck (Appanoose, Decatur)
- Steel piling, beams and deck (Ringgold)
- Steel stringers and concrete beams (Clarke)
- Steel stringers with concrete deck and fill (Lynn)
- Steel stringers with composite concrete reinforced deck (Audubon, Carroll, Shelby)
- Small pipe (Story)

- Precast culverts (Tama, Winnebago)
- Box culverts (Iowa, Woodbury)
- Pipe culverts (Iowa)
- Single or multiple structural plate pipe culverts or plate arch culverts (Hamilton)
- RRFC bridges (Winnebago)
- Railroad tank cars if D.A. allows it (Appanoose)
- CMP's (Keokuk, Winnebago)
- ConSpans (Winnebago)
- Precast beams (Clinton)
- Beam and slab (Appanoose)
- BISB (Clinton, Tama)
- BISB deck with steel shell piling and steel sheeting backwalls (Union)
- BISB deck with sheet pile backwalls and steel bearing piles (Adair)
- Corrugated metal culverts (Lynn)
- H-Beam Piling and Cap (Clarke)
- Oden slabs (Carroll)

Q-4) Have you developed a replacement superstructure(s) that you frequently install? If yes, please describe.

Nineteen respondents answered yes, thirty-three no.

Wood stringers with nailed wood deck, or steel stringers with a nailer for a wood deck. (Page)

Timber or steel pile substructure with steel or timber stringers and timber deck. (Woodbury)

Quad-tees and timber and steel decking (Dubuque)

Timber slab bridges in panels with timber abutments and W guardrail. (Lynn)

"Reynolds Deck"- Wood deck 6x6 installed at 15"-19" centers transverse, then 3x12 deck longitudinally. (Marshall)

Replace with county designed steel stringer bridge, steel H-piles with steel sheeting abutments, wood deck. (Keokuk)

Added steel beams to correct spacing, decked with sheet pile and covered with gravel. (Montgomery)

Eight steel beams evenly space for deck width of 24,' then transverse 4"x16" timber plank spaced at 28" off center then 3"x12" timber plank placed longitudinally. (Crawford)

Steel stringer bridges are constructed with new or used stringers and galvanized corrugated metal decks with a gravel or hot mix asphalt wearing surface. These are supplemented by steel handrail posts and galvanized W-beam guardrail handrails. Abutments are constructed using H-piling piers with channel caps, H-piling abutments with channel caps, and galvanized sheet piling backwalls. (Decatur)

For spans less than 60’, county constructs bridge using steel I-beams 24” x 79” 36 psi with 4” x 12” wood deck and 3” x 12” wearing. Private contractors bid for greater spans up to 80’ using “Oden Bridge” prefab. Prefab includes 30”-33” 50psi I-beams and sheer lugs. Concrete composite reinforced desk is poured on “stay in place” metal decking. Abutment uses sheet piles, interlocking backing wall and wing walls. (Audubon, Shelby)

Oden slab with Oden’s H-pile and sheet pile abutment design. Also, I-beam with Oden abutment design and concrete deck. (Carroll)

Structure uses I-beams at 2’ centers with steel diaphragms. Additional I-beams added to have a 26” or greater roadway width. Steel beams support W-beam Guardrail. (Cedar)

Steel deck on steel I-beams with 6” concrete on steel deck. (Marion)

Replace bridges with pipes. (Winnebago)

Will be BISB. (Union)

BISB. (Marion)

Seventy-foot BISB on stub abutments. Stubs are modification of H30-87 IDOT standard abutment (80’ beam choice) (Tama)

BISB using HP sections with PCC between. (Adair)

Q-5) Does your county have any experience in strengthening deficient bridge superstructures?

Twenty-one respondents answered yes, thirty-one no.

Bridge substructures?

Twenty-one respondents answered yes, thirty-one no.

Describe system/procedure used.

Replacing unusable old wood abutments with new timber abutments. (Cedar)
Strengthening existing member. (Buena Vista, Dallas, Marion, Sac)
Midspan support with piling and large beam. (Carroll)
Adding composite action. (Marion, Sac)

Replacing decks. (Appanoose)
Wood decking. (Mills, Tama)
Replacing concrete decks with open steel grid. (Lynn)
Lightweight deck replacement. (Dallas, Sac)
Adding members. (Audubon, Clarke, Decatur, Keokuk, Lynn, Marion, Mills, Page, Ringgold, Sac, Shelby, Warren, Woodbury)
Use 4"x16"x16' timbers spaced for legal loading. (Sioux)
Replacing wooden stringers. (Appanoose)
Replacing wooden stringers with steel beams. (Audubon, Shelby)
Adding cable support to truss members. (Dallas)
Adding plates to steel members. (Lynn)
Strengthening floor beams on truss bridges. (Madison)
On I-beam structures, welded steel plates to bottom flange to strengthen beam. (Kossuth)
Adding steel plating to high trusses and pony trusses. (Dubuque)
Adding additional piling. (Adair, Appanoose, Audubon, Carroll, Keokuk, Page, Ringgold, Shelby, Woodbury)
Piling jackets to strengthen wood pile. (Winnebago)
Replacing backwall plank. (Appanoose, Dallas, Keokuk)
Larger timbers for abutments and pile caps. (Sioux)
Jack up one side of bridge, remove 4' concrete deck and old cap, drive in new piles, replace cap and lower. (Tama)
Splice steel and wood piles on wood piles below stream line. (Henry)
Replacements of wood pile with steel H-pile. (Marshall)
Driving H-pile between deteriorated timber piling. (Crawford)
Replacement of rotten pile with concrete filled PVC pipes on top of existing pile, cut off below ground level. (Washington)
Rebuilding pile caps. (Appanoose, Audubon, Shelby)

Q-6) Ranking bridge types needing strengthening procedures.

Type (FHWA No.)	Votes for 1	Votes for 2	Votes for 3	Votes for 4	Not voted	Score
Timber stringer (702)	17	10	7	10	8	2.3
Steel stringer (302)	16	13	9	8	6	2.5
Steel pony truss (380) or Through truss (310)	10	16	11	7	8	2.3
Steel Girder plus floor beam system (303)	1	7	16	15	13	1.4
Wilson Precast	1	0	0	0	51	0.1

(Higher score indicates greater need)

Based upon the number of times a bridge type was ranked 1, 2, 3, or 4, and averaged by the number of votes, the steel stringer bridge type was found to be most in need of strengthening procedures. It was followed by the timber stringer, steel pony truss or through truss type, and lastly the steel girder plus floor beam system.

Kossuth County responded that they only replace structures.

Tama County reported that timber stringers are usually small enough that many repair choice already exist.

It was noted by Montgomery and Shelby Counties that steel pony truss or through truss type bridges are too narrow, and not worth strengthening. Tama County added that they are old enough that they have substructure problems as well, but are usually placed on large streams.

Keokuk County ranked Wilson precast bridges as in greatest need of strengtheningprocedures.

Q-7) What problems are frequently encountered on low volume bridges?

Superstructure:

- Beams
    - Rotted Stringers (Allamakee, Appanoose, Decatur, Dubuque, Madison, Page, Union, Warren, Washington)
    - Split wood stringers (Clayton, Union)
    - Rusted Stringers (Clayton)
    - Steel stringers too small, light or too great spacing. (Allamakee, Carroll, Decatur, Madison, Montgomery, Plymouth, Sac, Warren, Wright)
    - Advanced section loss at end of bridge on steel stringer bridges. (Keokuk)
    - Loss of section on truss members (Dallas)
    - Loss of section on stringers (Dallas)
    - Loss of connectors in diaphragms (Dallas)
    - Collision damage (Dallas, Dubuque)
    - Advanced beam deterioration. (Dubuque, Hamilton, Winneshiek)
    - Corrosion of exterior I-beams. (Clinton Dubuque)
    - Spikes come loose in older wood beams (Adams)
- Decks
    - Worn plank (Allamakee, Marshall, Mills, Union)
    - Broken wood decking and stringers due to excessive load-grain carts, steel-wheeled Amish vehicles. (Audubon, Decatur, Shelby, Washington)
    - Wood decking separating from wood stringers-clip and spikes working loose. (Audubon, Decatur, Shelby)
    - Deck deterioration. (Adams, Buena Vista, Des Moines, Hamilton, Lynn, Mills, Sac Tama)
    - Corrosion at bearing. (Winneshiek)
    - Holes in PCC decks (Clinton)
    - Maintenance of timber decks on steel beams. (Clinton)

  - Narrow or one lane (esp. for ag vehicles). (Allamakee, Cedar, Cherokee, Clark, Crawford, Des Moines, Hamilton, Lynn, Marion, Osceola, Sac, Tama, Winneshiek)
  - Loss of section on floor beam system (Dallas)
- Other
  - Rotted nailers (Allamakee, Appanoose, Decatur, Madison, Page, Union, Washington)
  - Split nailers (Union)
  - Railing damage. (Marion, Osceola)
  - Handrail damage on narrow bridges. (Audubon, Hamilton, Shelby, Woodbury)
  - Low capacity truss bridges that are fracture critical. (Appanoose, Des Moines, Union)
  - Farm machinery hitting truss members. (Dubuque, Washington, Woodbury)
  - Excessive rock on deck (Winneshiek)
  - No railing (Winneshiek)
  - Wilson precast sections too light to carry legal loads, deterioration, lack of composite action. (Keokuk)
  - Inadequate 50's or 60's design for 2003 standards, or newer calc with higher safety factor. (Humboldt)
  - Inadequate lateral support. (Carroll, Warren)
  - Load limits too conservative; restricts use by ag equipment, leading to limits being ignored. (Marion)
  - Inadequate load capacity. (Crawford)

Substructure:

- Piling
  - Rotten piling. (All counties)
  - Broken wood piling due to lateral pressure on abutments. (Adams, Decatur)
  - Section loss in timber piling (Dallas)
- Caps
  - Rolled caps. (Union)
  - Rotted caps. (Decatur, Page, Washington)
  - Substandard caps (Winneshiek)
- Backwalls/Wingwalls
  - Backwall failures. (Clinton, Cedar, Des Moines, Henry, Winnebago)
  - Rotted backwalls. (Dallas, Henry, Keokuk, Lynn, Madison, Marshall, Washington)
  - Rotted wing planks. (Dallas, Des Moines, Henry, Marshall)
  - Wings not tied back properly with deadmen. (Marshall)
- Stream/Erosion
  - Broken wood piling due to logs and ice chunks (Allamakee)
  - Drift accumulation. (Lynn, Winneshiek)

  - Undercut backwalls, causing soil loss. (Adams, Audubon, Carroll, Clark, Clayton, Hamilton, Lynn, Shelby, Union, Washington,)
  - Scour along piers and abutments. (Clayton, Mills, Winneshiek)
  - Poor alignment with stream (Winneshiek)
  - Wing problems due to roadway, ditch and stream erosion. (Clark, Madison, Washington)
  - Stream deepening causing shallow encasement appearance. (Adams, Adair, Allamakee, Clark, Mills, Madison, Plymouth, Warren, Woodbury)
  - Ice damage (Dallas)
- Abutments
  - Deterioration. (Hamilton)
  - Spalling and pop out of concrete abutments (Winneshiek)
  - Inward movement of high wood abutments (Sac)
  - Failure of high wood abutments (Humboldt, Winnebago)
  - Undermining of stub abutments (Sac)
  - Timber abutments not tied back (Carroll)
- Other
  - Exposed footing (Sac)

APPENDIX B

National Questionnaire

BRIDGE TYPES

BT-1) All things being considered equal with regards to geometrics do you favor one type of construction over another? Please rate in order of preference (1 being your first choice); indicate equals by using the same number.

States, Locals

19%, 15%	Structural Steel (rolled shapes, plate girders, truss, etc.)
15%, 17%	Reinforced Concrete (T-Beams, Slab Bridges, etc.)
21%, 20%	Concrete Box Culverts (precast and CIP)
13%, 16%	Structural Pipe/Steel Arch Culverts
22%, 19%	Prestressed Concrete (I-Beams, boxes, slabs, etc.)
9%, 12%	Timber (glulam and sawn beams, glulam deck bridges, etc.)
1%, 2%	Other (specify, e.g., proprietary system)

BT-2) For the same types of construction listed above, would you expect to use your own labor force and equipment or let a contract for construction?

State Responses:

13% Own	87% Con	Structural Steel (rolled shapes, plate girders, truss, etc.)
7% Own	93% Con	Reinforced Concrete (T-Beams, Slab Bridges, etc.)
25% Own	75% Con	Concrete Box Culverts (precast and CIP)
25% Own	75% Con	Structural Pipe/Steel Arch Culverts
13% Own	87% Con	Prestressed Concrete (I-Beams, boxes, slabs, etc.)
33% Own	67% Con	Timber (glulam and sawn beams, glulam deck bridges, etc.)

Other ____

Local Responses:

19% Own	74% Con	Structural Steel (rolled shapes, plate girders, truss, etc.)
14% Own	81% Con	Reinforced Concrete (T-Beams, Slab Bridges, etc.)
23% Own	73% Con	Concrete Box Culverts (precast and CIP)
43% Own	51% Con	Structural Pipe/Steel Arch Culverts
9% Own	83% Con	Prestressed Concrete (I-Beams, boxes, slabs, etc.)
37% Own	53% Con	Timber (glulam and sawn beams, glulam deck bridges, etc.)

Other ____

BT-3) In an effort to obtain information on the use of pre-engineered or prefabricated components in off-system bridge rehabilitation and replacements, please provide the names of prefabricated bridge/culvert products you have used in the past.

- Precast Concrete Products (Standard ASTM Culverts, Bebo, ConSpan, HySpan). In addition to proprietary products, various uses of standard precast concrete beams and slabs. Also documented use of owner fabricated precast concrete sections for short to medium spans.

- Prefabricated Steel Structures (prefabricated trusses, i.e., U.S. Bridge, Acrow Panel, Continental Bridge, Mabey Bridge, Bailey Bridge; steel and aluminum pipe arches; Inverset bridges)
- Timber Structures (timber beams, timber decks on timber beams, timber decks on steel beams, nail laminated panels, glulam pier caps and bridge rails)
- Others (steel and aluminum grid decks, exodermic decks, HDPE pipe culvert)

BT-4) In instances where you have used prefabricated or pre-engineered components to rehabilitate or replace a portion of a bridge or a bridge in its entirety, rank the following criteria in terms of importance. (Percentages indicate order of importance).

States, Locals:

20%, 23%	Total cost
21%, 17%	Speed of construction
20%, 18%	Traffic considerations
12%, 21%	Lack of engineering staff
17%, 12%	Anticipated durability
10%, 9%	Lack of other options

BT-5) Please list any commercial design software and/or standard plans you have successfully used for the design of off-system bridges.

- Software Programs
  - Concrete Structures – LEAP software, PCA SLABBRDG, PennDOT PSLRFD Software
  - Steel Structures – AISI Short Span Steel Bridge Plans & Software, MDX, Brass Girder, Merlin Dash, Georgia Beam, PennDOT STLRFD
  - Foundations – Seisab, GRL WEAP, Brass Pier, PennDOT ABUT 5 and ABLRFD, Leap RC Pier LA, Retain Pro
  - Other – STAAD, STRUDL, AASHTO Ware, PennDOT Box
- Standard Plans
  - Government standards - West Virginia, Pennsylvania, Iowa, Ohio, Washington, Federal Highway Administration
  - Industry standards - AISI Short Span Steel Bridge Plans & Software, Standard Precast Structures - ASTM Culverts, ConSpan, Bebo, USDA Standard Timber Bridge Plans, Pre-engineered steel trusses

BT-6) For the new structures rated in questions BT-1 and BT-2, rank your reasons for your material/structure preferences. (Percentages shown indicate order of preference).

States, Locals:

17%, 16%	Initial cost	13%, 16%	Life cycle cost
15%, 15%	Ease of construction	14%, 12%	Familiarity
10%, 11%	Ease of design	5%, 4%	Lack of competition
13%, 11%	Material availability	14%, 14%	Durability

BT-7) In new construction, indicate your preferred type of deck.

States, Locals:

31%, 27%	Cast-in-place reinforced concrete
18%, 23%	Full depth precast concrete panels
21%, 20%	Precast concrete elements with CIP topping
13%, 18%	Timber decking
17%, 13%	Steel grids

BT-8) What types of bridge railings do you use?

States, Locals:

89%, 48%	Concrete “Jersey Barriers”
47%, 42%	Timber railing
83%, 77%	Post and beam steel rails
18%, 9%	No railings

BT-9) Substructure Units:

List in the order of priority the preferred type of construction for a new or replacement abutment or pier. (Percentages shown indicate order of preference)

	Abutment (States, Locals)	Pier (States, Locals)
Cast-in-Place Concrete	53%, 27%	48%, 28%
Steel Piles and Lagging		
Timber	5%, 12%	4%, 11%
Concrete	3%, 17%	4%, 17%
Timber Piles w/Timber Lagging	3%, 11%	2%, 12%
Sheeting	3%, 10%	
Pile Bent:		
Steel	31%, 15%	34%, 21%
Timber	3%, 8%	8%, 11%

BT-10) For all new structures crossing a waterway, would a pile foundation be required for all sites except those where rock is found at or near the proposed bottom of footing elevation?

States:	81%	Yes	19%	No
Locals:	72%	Yes	22%	No

BT-11) What other types of scour protection do you use for new or rehabilitated structures?

	Yes (States, Locals)	No (States, Locals)
Sheeting	44%, 66%	50%, 34%
Stone Fill	94%, 97%	6%, 3%
Stream Bed Liners	50%, 19%	39%, 81%
Articulated concrete block pavers	33%, 6%	50%, 94%
Increased cover depth to the bottom of footing	89%, 80%	6%, 20%

BT-12) For bridge replacement projects over waterways, is a hydraulic analysis usually completed? If not, what type of hydraulic evaluation of a site is completed?

States:	100% Yes	0% No
Locals:	88% Yes	9% No

- Responses indicate use of WSPRO, HEC RAS, TR 20 and TR 55 procedures. For cases where hydraulic analysis is not conducted, historical assessment of flooding at the site is used with maintenance or enlargement of the opening where possible. Scour is investigated as a potential indicator of flow problems.

BT-13) Do you feel that your geographic area plays a large part in the selection of bridge types?

States:	67%	Yes	33%	No
Locals:	59%	Yes	36%	No

- Geography, geology and material availability were all cited as strong influences in the selection of structure types. Other issues cited included climatic influences such as freeze-thaw, road salting and flooding, and the availability of appropriate equipment for construction of various types of structures.

APPENDIX C

Iowa DOT Bridge Related Research Prior to 1980

As noted in Chapter 2, bridge related research sponsored by the Iowa DOT and the Iowa Highway Research Board prior to 1980 is presented in this appendix. The classifications used in that chapter are also used in this appendix. If there is no research in a particular classification, that classification has not been included (i.e. there is no research in the area of Bridge alternatives for Low Volume Roads, thus that classification (C.2) is not included.

For cross-reference, projects presented in Chapter 2 and in this appendix are all listed in Table C.1.

Table C.1. Iowa DOT Bridge Related Research Projects

Project Number	**Title**
TR-453	Low Water Stream Crossings: Design and Construction Recommendations
HR-453M	Low Water Stream Crossings: Design and Construction Recommendations and Design Guide (2003)
TR-445	Development of Bridge Load Testing Process for Load Evaluation
TR-444	Demonstration Project Using Railroad Flatcars for Low-Volume Bridge
TR-440	Field and Laboratory Evaluation of Precast Concrete Bridges
TR-436	Retrofit Methods for Distortion Cracking Problems in Plate Girder Bridges
TR-429	Evaluation of Appropriate Maintenance, Repair and Rehabilitation Methods for Iowa Bridges
TR-421	Use of Railroad Flat Cars for Low Volume Bridges
TR-410	Investigation of Two Bridge Alternatives for Low Volume Roads – Phase II Vol. 1 & 2
HR-397	Field/Laboratory Testing of Damaged Prestressed Concrete Girder Bridges
HR-393	Preventing Cracking at Diaphragm/Plate Girder Connections in Steel Bridges
HR-390	Testing of Old Reinforced Concrete Bridges
HR-385	Stream Stabilization in Western Iowa: Structure Evaluation and Design Manual
HR-382	Investigation of Two Bridge Alternatives for Low Volume Roads
HR-378	Metric Short Courses for the Office of Bridges and Structures
HR-373	Investigation of Plastic Pipe for Highway Applications, Phases I & II
HR-370	Pipe Rehabilitation With Polyethylene Pipe Liners
HR-365	Evaluation of Bridge Replacement Alternatives for County Bridges
HR-362	Design Methodology for Corrugated Metal Pipe Tiedowns

HR-354	An Engineering Study to Design Triple Box Culvert Standards
HR-353	Epoxy-Coated Strands in Composite Precast Prestressed Concrete Panels
HR-344	Potential Scour Assessments and Estimates of Maximum Scour at Selected Bridges in Iowa
HR-333	Design Methodology for Post-Tension Strengthening of Continuous Span Bridges
HR-332	Design Methodology for Corrugated Metal Pipe Tiedowns
HR-323	Development of Evaluation, Rehabilitation, and Strengthening Concepts for Low Volume Bridges
HR-319	Lateral Load Resistance of Diaphragms in Prestressed Concrete Girder Bridges
HR-313	Air Formed Arch Culvert Construction – Washington County
HR-314	Air Formed Arch Construction – Crawford County
HR-310	Composite Precast Prestressed Concrete Bridge Slabs
HR-308	Strengthening of Existing Continuous Span Steel Beam Concrete Deck Bridges by Post Tensioning
HR-307	Sediment Control in Bridge Water Ways
HR-306	Investigation of Uplift Failures in Flexible Pipe Culverts
HR-302	Alternate Method of Bridge Strengthening
HR-292	Validation of Design Recommendations for Integral Abutment Piles
HR-287	Strengthening Existing Continuous Composite Bridges
HR-273	Pile Design and Tests for Integral Abutment Bridges
HR-252	Design of Integral Abutment Bridges
HR-247	Design Manual for Low Water Stream Crossings
HR-239	Load Ratings for Secondary Bridges
HR-238	Strengthening Existing Single Span Steel Beam Concrete Deck Bridges
HR-237	Shelby County Evaluation of Control Structures for Stabilizing Degrading Stream Channels
HR-236	Evaluation of Control Structures for Stabilizing Degrading Stream Channels
HR-219	Settlement at Culverts
HR-214	Feasibility Study of Strengthening Existing Single Span Steel Beam Concrete Deck Bridges
HR-208	Alternative Method of Stabilizing the Degrading Stream Channels in Western Iowa
HR-192	Evaluation of Dense Bridge Floor Concrete Using High Range Water Reducer
HR-177	Concrete Bridge Deck Repair Using Injected Epoxy Resin
HR-169	Ultimate Load Behavior of Full Scale Highway Truss Bridges
HR-160	Feasibility Study on Dynamic and Ultimate Static Load Tests on Highway Bridges
HR-104	Field Observation of Live Lightweight Aggregate Pretensioned Prestressed Concrete Bridge Beams

HR-95	Repair and Protection of Concrete Bridge Superstructures
HR-89	Model Investigation of Channel Stabilization of Mosquito Creek
HR-74	Fatigue and Residual Stress Investigation of Composite Prestressed Steel Beams
HR-73	The Flexural Fatigue Strength of Prestressed Steel I-Beams
HR-61	Distribution of Loads in Beam-and-Slab Bridges
HR-51	Use of Aluminum in Highway Bridges
HR-43	Dynamic Tests of a Three-Span Continuous I-Beam Highway Bridge
HR-42	Dynamic Behavior of Two Continuous I-Beam Bridges
HR-36	Prestressed Concrete Bridge Beams
HR-31	Structural Behavior of a Model of a Bridge Wingwall of Constant Thickness (1956)
HR-24	A Generalized Model Study of Scour Around Bridge Piers and Abutments

C.1 Abutments

HR-31 Structural Behavior of a Model of a Bridge Wingwall of Constant Thickness (1956)

This investigation of the wingwall of concrete abutments was undertaken to investigate them experimentally and analytically. The literature review reveal minimal results. Instrumented models of the wingwall of various thicknesses were instrumented for measuring strains and deflections. The finite-difference method of analysis was used to obtain theoretical results. The results of this phase (Stage 2) of the investigation plus those of Stages 3 and 4 will make it possible to develop principal moment contours (for both constant and variable thickness reinforced concrete wingwall) that will assist engineers in the design of these elements.

C.4 Concrete Decks

HR-177 Concrete Bridge Deck Repair Using Injected Epoxy Resin

Maintenance of spalled bridge decks requires constant surveillance and the commitment of considerable manpower and equipment by maintenance forces. Maintenance cost for deck repair was $68,000 in Fiscal Year 1977 and $83,400 in Fiscal Year 1978. Patching of spalled areas with bituminous material is a temporary repair, at best. It will help reduce traffic impact loadings on the structure but will do nothing to prevent further deterioration of the decks. It is usually noted that concrete around the spalled area delaminates (or is delaminated at the time the bituminous material is placed) this, in turn, spalls increasing the area of deterioration.

Results to date indicate that pumping epoxy into delaminated areas to delay spalling in bridge decks is a viable maintenance procedure when large delaminated areas are present. Those instances that seem most adaptable to epoxy injection are bridges that have developed delaminated areas but do not exhibit very much spalling. Bridges with "v" type spalling over reinforcing steel or small (2 to 3 sq ft) hollow areas around spalls can be repaired more economically with partial depth PCC patches using low slump concrete. Continued observation and monitoring of repaired deck areas will be required to determine long term results.

Continued monitoring of the repaired bridge deck on I-80 near Grinnell will be necessary to determine the longevity of the repair. A year and a half after this repair was completed, it was estimated that 80% of the area remained "glued together". There has been very little change since then.

Additional training of personnel will be required to increase efficiency in accomplishing the re-bonding, however, it is believed that the procedure has definite application in bridge deck maintenance.

C.9 Prestressed and Reinforced Concrete Beams and Bridges

HR-104 Field Observation of Live Lightweight Aggregate Pretensioned Prestressed Concrete Bridge Beams

The use of lightweight aggregates in prestressed concrete is becoming more of a reality as our design criteria become more demanding. Bridge girders of greater lengths have been restricted from travel on many of our highways because the weight of the combined girders and transporting vehicle is excessive making hauls of any distance prohibitive. This, along with new safety recommendations, prompted the State of Iowa to investigate the use of lightweight aggregate bridge girders.

Until recently, it was possible to use 67 ft bridge girders to cross a two lane section of interstate highway, now it is necessary to have at least an 87 ft span to satisfy the new safety standards that require any obstruction such as columns or abutments be at least an additional 10 ft away from the edge of the pavement. If a skewed crossing is required, the length of the girders could conceivably be 90-95 ft in length. With these lengths, the selfweight of the girder due to the normal weight concrete would be more than the state law permits.

A series of three projects was started to investigate the possibility of using lightweight aggregate in prestressed concrete bridge beams. The objective of this project was to study the effect of lightweight aggregate concrete on the camber of bridge girders in a field situation.

HR-61 Distribution of Loads in Beam-and-Slab Bridges

A new procedure for predicting the strains and deflections of the beams in simple-span beam-and-slab bridges of the usual proportions has been developed. It divides the calculations into two primary steps:

1.) Temporary reactions are assumed at the beams to prevent deflections of the beams, and the loads are distributed to these reactions by the slab acting as a continuous beam.
2.) The temporary reactions are removed and the consequent effects on the beams are computed.

Since no deflections or moments are produced in the beams in Step 1, the entire effect on the beams is found in Step 2. This effect on a beam is assumed to be that of a loading consisting of:

1.) a concentrated or narrowly distributed force, the temporary reaction reversed, and
2.) a widely distributed force produced by the resistance of the slab to deformation.

Part 2 of the beam loading has been assumed to be sinusoidal, but any other form could be assumed. For the bridges tested the effects of Part 2 are relatively small; so the precision of the predictions of maximum strains and deflections is not sensitive to changes in the form assumed.

It is suggested that, pending further study, the use of the procedure be limited to bridges having a ratio of span to beam spacing of 2 or more, and also a ratio of beam to slab stiffness, H, of 2 or more.

To obtain checks on the predictions by the proposed procedure, by the present (1953) AASHTO specifications, and by the tentative revisions (T-15-50), four bridges were tested. Two are full-size bridges in use on a highway; their spans are 41.25 ft and 71.25 ft, and their roadways are 30 ft wide. The other two were built in a laboratory. They include crown, curbs, and diaphragms; their spans are 10 ft and 25 ft, and their roadways are 10 ft wide. Each of the four bridges has four beams equally spaced, has the interior beams larger than the exterior, and is of composite construction.

Strains and deflections were measured at a number of locations at each bridge for various positions of the loads. By comparing experimental results with those obtained using the new procedure, it was concluded that the proposed procedure provides improved predictions under a much wider range of conditions than do current specification methods. To understand and use it requires no special training, and the time required for its use is only about one hour per analysis; so it should be practical for practicing engineers to use it.

HR-36 Prestressed Concrete Bridge Beams

Before using prestressed concrete (P/C) beams in bridges on the primary highway system an extensive testing program of the beams was completed. It consisted of investigating the flexural strength, shear strength and development length in the P/C beams. The main objectives of the flexural tests were:

- Determining the flexural stresses in the concrete and midspan deflections due to superimposed loads.
- Investigating the stresses in the prestressing reinforcement.
- Determining flexural behavior before and after the concrete cracks.
- Determining the accuracy of theoretical analysis by comparing experimental and theoretical results.

Although there were numerous conclusions from the investigation, the main conclusion was that experimentally determined flexural stresses and deflections due to one live load were in close agreement with those predicted using theoretical analyses.

In the shear strength portion of the investigation, three P/C I-section without web reinforcement were tested to shear failure. The only variables investigated were the length of shear span and the concrete strength. From the results, the most critically stressed points were determined. Also, the magnitude of applied loading at failure was compared with the theoretical magnitudes determined using various theories of failure for concrete under combined stresses.

In the anchorage phase of the investigation, the objectives were:

- to investigate the distribution of stresses in the anchorage zone of a P/C beam when the tensioned steel cables are released.
- to determine the length required for the concrete to assume its full prestress.
- to determine the effect on the anchorage zone stress distribution when the beam is loaded to failure.

Based on the testing of one specimen, the following conclusions were made:

- the 3/8 in. diameter prestressing cables were determined to be completely anchored to the concrete beam in a length less than 6 in.
- the length of the beam required at each end for full prestress to be transferred to the concrete was determined to be slightly over 30 in.

C.10 Scour

HR-95 Repair and Protection of Concrete Bridge Superstructures

One of the objectives of this investigation was to investigate methods and materials for making durable repairs to concrete bridge floors. Experimental patching was completed on several different bridges using different material. Based on results to date the following conclusions can be made:

- Adequate preparation of the repair area is essential if durable repairs are to be accomplished.
- As of now, Portland cement concrete is the most satisfactory material for patching and resurfacing. The concrete should contain a water-reducing agent and an air-entraining agent.
- A successful repair program requires strict compliance with detailed repair instructions, quality materials, and good workmanship.

HR-89 Model Investigation of Channel Stabilization of Mosquito Creek

This investigation consisted of a series of hydraulic model tests to determine the most suitable design to stabilize the bed of Mosquito Creek downstream from a highway bridge on the county road connecting the towns of Earling and Panama, Shelby County, Iowa. In recent years, scour of the prototype channel bed and concomitant widening of the channel walls as a result of the upstream migration of a series of head cuts has caused partial exposure of the abutments of the main highway bridge. Without artificial channel protection, a replacement of the bridge or major repair works would be imminently necessary. This problem is typical in a large number of streams in southwestern Iowa and is probably the result of downstream channel straightening which was performed several decades ago. Although the present task consisted of conducting a model study to establish the proper protection for a specific site, it is probable that a successful field performance of the final design would engender widespread use of similar protective works on other streams in the same general area. This investigation, however, was primarily concerned with the model tests which were made in order to predict qualitatively the prototype behavior for various flood conditions at the stabilization site. Naturally, additional field information must be collected and structural designs must be completed before actual construction begins.

The basic structure, which was originally conceived by the Iowa Highway Commission to accomplish stabilization of the Mosquito Creek Channel, consisted of a trapezoidal weir or sill with an accompanying stilling basin. An obstruction to the natural flow in the channel was believed to be an economical means of creating an upstream pool of low velocity which would, in turn, cause sediment to be deposited upstream from the structure and thereby raise the channel bed some 14 ft above its present elevation. Although several modifications were incorporated in the final design, the basic components of a weir and stilling basin were retained as a consequence of a comprehensive set of model studies under various imposed flow conditions.

HR-24 A Generalized Model Study of Scour Around Bridge Piers and Abutments

Four classes of variables are apparent in the problem of scour around bridge piers and abutments - geometry of piers and abutments, stream-flow characteristics, sediment characteristics, and geometry of site. The laboratory investigation, from its inception, has been divided into four phases based on these classes. In each phase the variables in three of the classes are held constant and those in the pertinent class are varied. To date, the first three phases have been studied.

Typical scour hole patterns related to the geometry of the pier or abutment have been found. For equilibrium conditions of scour with uniform sand, the velocity of flow and the sand size do not appear to have any measurable effects on the depth of scour. This result is especially encouraging in the search for correlation between model and prototype since it would indicate that, primarily, only the depth of flow might be involved in the scale effect. The technique of model testing has been simplified, therefore, because rate of sediment transportation does not need to be scaled. Prior to the establishment of equilibrium conditions, however, depths of scour in excess of those for equilibrium conditions have been found. A concept of active scour as an imbalance between sediment transport capacity and rate of sediment supply has been used to explain the laboratory observations.

C.11 Steel Beams and Bridges

HR-74 Fatigue and Residual Stress Investigation of Composite Prestressed Steel Beams

Two composite, prestressed, steel beams, were fatigue tested to failure. The two composite beams were different in that two different fabrication procedures were used to attach the bottom cover plate. Stresses and deflections were measured at regular intervals and the behavior of each beam was documented during the fatigue tests. Residual stresses were then evaluated by testing segments of each beam. Although only two specimens were tested, several conclusions were made:

- High residual stresses in the weld area contributed substantially to eventual fatigue failure at points of high stress concentration.
- Impacting failure may not be detected by deflection or strain measurements.
- Inelastic behavior of the concrete slab did not adversely effect the fatigue results in either specimen.

HR-73 The Flexural Fatigue Strength of Prestressed Steel I-Beams

The purpose of this investigation was to study the flexural fatigue strength of two prestressed steel I-beams which had previously been fabricated in connection with a jointly sponsored project under the auspices of the Iowa State Highway Commission.

The beams were prestressed by deflecting them under the action of a concentrated load at the center of a simple span, then welding unstressed high strength steel plates to the top and bottom flanges to retain a predetermined amount of prestress. The beams were rolled sections of A36 steel and the plates were USS "T-1" steel.

Each of the two test specimens were subjected to an identical repeated loading until a fatigue failure occurred. The loading was designed to produce stresses equivalent to those which would have occurred in a simulated bridge and amounted to 84 percent of a standard H-15 live load including impact. One of the beams sustained 2,469,100 repetitions of load to failure and the other sustained 2,756,100 cycles.

Following the fatigue tests, an experimental study was made to determine the state of stress that had been retained in the prestressed steel beams. This information, upon which the calculated stresses of the test could be superimposed, provided a method of correlating the fatigue strength of the beams with the fatigue information available on the two steels involved.

HR-43 Dynamic Tests of a Three-Span Continuous I-Beam Highway Bridge

In this investigation, a three-span continuous I-beam highway bridge was tested to determine its response to live load. Live load stress frequency curves for selected points were determined as well as the static and dynamic load distribution to the longitudinal composite beam members. The six beam composite bridge has four traffic lanes with a roadway width of 48 ft. The WF beams have partial length cover plates at the piers.

Previous research has determined that beams with partial length cover plates have very low fatigue strength. In this study, it was found that the magnitude of the stresses due to actual highway loads were considerably smaller than those computed from specification loading. Also, the larger stresses which were measured occurred a relatively small number of times. These data indicate that some requirements for reduced allowable stresses at the ends of cover plates are too conservative.

The load distribution to the longitudinal beams was determined for static and moving loads and includes the effect of impact on the distribution. Lateral distribution of live load was found to be consistent with the specifications, however, there is longitudinal distribution, and therefore the specifications are too conservative. The effective composite section was found at various locations to evaluate the load distribution data. Composite action was observed in both the negative and positive moment regions.

HR-42 Dynamic Behavior of Two Continuous I-Beam Bridges

In the summer of 1956, an experimental investigation of two continuous I-beam bridges was initiated. The objectives of the tests were:

- to determine static and dynamic stresses and deflections.
- to determine increases in static stresses due to impact.
- to measure maximum amplitude of vibration and determine the natural frequency of vibration.
- to determine main cause of vibration.
- to determine damping characteristics of a bridge.

Since only two bridges were tested, observations rather than conclusions were made; the more significant observations were:

- both bridges had more ability to resist live load than assumed in design.
- safety curbs and the entire slab resisted a portion of the live load moments.
- computed deflections in all cases were greater than measured values (i.e. both structures were stiffer than assumed in design).
- in both structures, the impact percentages obtained experimentally were determined to be extremely variable.

C.12 Truss Bridges

HR-169 Ultimate Load Behavior of Full Scale Highway Truss Bridges

As a result of the construction of the Saylorville Dam and Reservoir on the Des Moines River, six highway bridges crossing the river were scheduled for removal. One of these, an old pin connected high-truss single-lane bridge, was selected for several tests which included ultimate load tests. The purpose of the ultimate load tests, which are summarized in this report, was to relate design and rating procedures presently used in bridge design to the field behavior of this type of truss bridge. The ultimate load tests consisted of ultimate load testing of one span of the bridge, of two I-shaped floor beams, and of two panels of the timber deck. The theoretical capacity of each of these components is compared with the results from the field tests. The bridge was rated using the present AASHTO Maintenance Manual. The ratings of the bridge and its components averaged about 25% of capacity. The ratings were fairly consistent except for the floor beams, where the assumption on lateral support conditions for the compression flange caused considerable variation.

HR-160 Feasibility Study on Dynamic and Ultimate Static Load Tests on Highway Bridges

As a result of the construction of the Saylorville Dam and Reservoir on the Des Moines River, six highway bridges had to be removed. Five of these were old high-truss single-lane bridges, each bridge having several simple spans. The remaining bridge was a fairly modern (1955) double 4-span continuous beam-and-slab composite highway bridge. The availability of these bridges provided an unusual opportunity for the investigation of the load behavior of full-scale bridges.

Because of the magnitude of the potential testing program, a feasibility study was initiated and the results are presented in a two-part final report: Part I summarizes the findings and Part II presents the supporting detailed information.

In brief, the following conclusions were drawn from the study:

Beam-and-slab bridge:

1.) testing to failure is not feasible.
2.) dynamic testing at design load and overload levels will provide useful data.
3.) testing of deck components under static and fatigue loads should be conducted.

High-truss bridges:

1.) ultimate load tests should be conducted on three selected spans.
2.) fatigue tests should be undertaken on complete component members selected from all truss bridges.
3.) tests should be conducted on in-place timber decks and timber stringers.

Significant information on the behavior of bridges designed for normal service can be obtained from a wide variety of tests on these bridges. An outline of these tests is presented.

APPENDIX D

Free Engineering Software Available

Free Engineering Software Available				
Software Category	**Title**	**Source**	**URL**	**Description**
Structural Engineering Superstructure Analysis & Design	Alaska Bulb T	Alaska DOT Bridge Section	http://www.dot.state.ak.us	Design of prestressed concrete I-beams.
	BARS-PC	Ohio DOT Structure Rating Group	http://www.dot.state.oh.us/srg/default.htm	AASHTO BARS-PC bridge rating program available for download for use by Ohio consultants.
	CONC	California Department of Transportation - Division of Engineering Services	http://www.dot.ca.gov/hq/esc/earthquake_engineering/CompProg/dosprog.html	Design or analysis of rectangular or flanged reinforced concrete sections for HS20 loading and Caltrans permit loads.
	LRFD Prestressed Beam Program	Florida DOT Structures Design Office	http://www11.myflorida.com/structures/proglib.htm	Analysis of prestressed concrete beams using the AASHTO LRFD Specifications.
	PGSuper	Washington State Bridge and Structures Office	http://www.wsdot.wa.gov/eesc/bridge/software/	Analysis and design of prestressed concrete beams using AASHTO LRFD Specifications including stress and stability during transportation.
	Plank for Windows	Colorado DOT Engineering Customer Support Unit	http://www.dot.state.co.us/DevelopProjects/DesignSupport/ecsu/	Computes the rating of plank bridges.
	PSG (Prestressed Girder)	Colorado DOT Engineering Customer Support Unit	http://www.dot.state.co.us/DevelopProjects/DesignSupport/ecsu/	DOS based program for prestressed girder design using the AASHTO Standard Specifications.
	Qcon Bridge	Washington State Bridge and Structures Office	http://www.wsdot.wa.gov/eesc/bridge/software/	Live load analysis and load combinations for simple or continuous bridges using AASHTO LRFD HL93 loadings.
	Slab Rating for Windows	Colorado DOT Engineering Customer Support Unit	http://www.dot.state.co.us/DevelopProjects/DesignSupport/ecsu/	Computes the rating of slab bridges.
	Timber Rating for Windows	Colorado DOT Engineering Customer Support Unit	http://www.dot.state.co.us/DevelopProjects/DesignSupport/ecsu/	Computes the rating of timber bridges.
	Colorado DOT COGO	Colorado DOT Engineering Customer Support Unit	http://www.dot.state.co.us/DevelopProjects/DesignSupport/ecsu/	Coordinate geometry program that interfaces with AutoCAD.
Structural Engineering Substructure	Drilled Shaft Design	Florida DOT Structures Design Office	http://www11.myflorida.com/structures/proglib.htm	Resistance of drilled shafts founded in sand or clay.
	LRFD Box Culvert	Florida DOT Structures Design Office	http://www11.myflorida.com/structures/proglib.htm	Design of culverts, headwalls, wingwalls and cutoff walls using AASHTO LRFD.

Free Engineering Software Available				
Software Category	**Title**	**Source**	**URL**	**Description**
	LRFD Retaining Wall	Florida DOT Structures Design Office	http://www11.myflorida.com/structures/proglib.htm	Design and analysis of cast-in-place retaining walls using AASHTO LRFD.
	Pile Bent Program	Florida DOT Structures Design Office	http://www11.myflorida.com/structures/proglib.htm	Analysis of fixed and pinned pile bents including lateral loads.
Structural Engineering Miscellaneous	Barlist	Washington State Bridge and Structures Office	http://www.wsdot.wa.gov/eesc/bridge/software/	Reinforcing steel estimating tool.
	BEToolbox	Washington State Bridge and Structures Office	http://www.wsdot.wa.gov/eesc/bridge/software/	Miscellaneous engineering utilities including horizontal and vertical curve elevations, section properties, pile loads in a pile group, precast girder analysis, built-up truss member properties and biaxial bending capacity of concrete sections.
Hydraulics	WSPRO	FHWA	http://www.fhwa.dot.gov/bridge/hydsoft.htm	Open channel flow water surface profile modeling. Can be used for flow at bridges, culverts and for scour computations.
	HY 8 Culvert Analysis	FHWA	http://www.fhwa.dot.gov/bridge/hydsoft.htm	Automated design of hydraulic structures in accordance with FHWA procedures.
	BOXCAR 1.0	FHWA	http://www.fhwa.dot.gov/bridge/hydsofta.htm#table	Design of reinforced concrete box culverts.
	PIPECAR 2.1	FHWA	http://www.fhwa.dot.gov/bridge/hyddescr.htm#pipecar_2_1	Design of reinforced concrete pipe culverts.
	CMPCHECK 1.0	FHWA	http://www.fhwa.dot.gov/bridge/hyddescr.htm#cmpcheck_1_0	Code check for design of corrugated metal pipes.
Geotechnical	SPT97	Florida DOT Structures Design Office	http://www11.myflorida.com/structures/proglib.htm	Static pile capacity calculator for concrete, H, pipe and cylinder piles.
	SPile	FHWA	http://www.fhwa.dot.gov/bridge/geosoft.htm	Determines ultimate vertical pile capacity using various methods.
	COM624P	FHWA	http://www.fhwa.dot.gov/bridge/geosoft.htm	Laterally loaded pile analysis.
	RSS	FHWA	http://www.fhwa.dot.gov/bridge/geosoft.htm	Analysis and design of reinforced soil slopes.
	ReSSA	FHWA	http://www.fhwa.dot.gov/bridge/geosoft.htm	An updated version of RSS to compute stability of reinforced slopes using various methods.