T. J. Wipf, F. W. Klaiber, D. M. Besser, M. D. LaViolette

Manual for Evaluation, Rehabilitation and Strengthening of Low Volume Bridges

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Iowa DOT Project HR-323
ISU-ERI-Ames-93062

Iowa Department of Transportation

report

College of Engineering
Iowa State University

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ABSTRACT

This report contains an evaluation and design manual for strengthening and replacing low volume steel stringer and timber stringer bridges. An advisory panel consisting of county and municipal engineers provided direction for the development of the manual. NBI bridge data, along with results from questionnaires sent to county and municipal engineers was used to formulate the manual.

Types of structures shown to have the greatest need for cost-effective strengthening methods are steel stringer and timber stringer bridges. 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, which can be used by county engineers, 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 an important part of the manual. Microcomputer spreadsheet software for several of the strengthening methods, types of replacement bridges and for the economic analysis has been developed, documented and presented in the manual. So the manual, Chp. 3 of the final report, can be easily located, blue divider pages have been inserted to delineate the manual from the rest of the report.

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1. INTRODUCTION AND RESEARCH APPROACH

1.1. Background

Numerous national studies have been completed detailing the substantial structural problems of large ADT (average daily traffic) highway bridges on the federal and state level. However, based upon existing literature, the problems local governments face daily have not been adequately addressed. Iowa officials have a special interest in addressing these concerns since an April 1989 Transportation Report (67) indicated that 86.4 percent of the rural bridge maintenance responsibilities are assigned to the local level; only 13 percent are assigned to the state, and the remaining 0.6 percent are assigned to "other" which denotes private or a combination of custodial responsibilities. Iowa is one of sixteen states in which the federal government has no bridge maintenance responsibilities. Iowa not only has the highest percentage of rural bridge maintenance problems assigned to the local level, but it is also the state with the highest percentage of rural bridge maintenance responsibilities assigned to the county level.

In 1989, the FHWA reported 23.5 percent of the nation's highway bridges were structurally deficient and 17.7 percent were functionally obsolete (56). A 1989 report by the National Association of Counties indicated 72 percent of all Iowa county bridges have SI&A sufficiency ratings less than or equal to 80 percent (85,86). It is important to note that while most of this 72 percent qualify for Federal aid, only a very small percentage may receive that help. In 1986, Galambos (27) reported on the scope of this financing problem by noting that $48.3 billion was needed for bridge problems, however, Congress only authorized $1.9 billion. Cooper (16) in 1990 estimated the cost of rehabilitating the nation's highway bridges to adequate service levels would be $52 billion; currently the amount budgeted is slightly over $1 billion per year.

1.2. Research Objectives

Previous investigations have concentrated on defining the national infrastructure deficiency problem, methods of financing possible solutions, and specific structural details which propose to solve the problems of long-span bridges (most frequently found on the primary highway system). The State of Iowa has 89,594 miles of county roads, most of which are unpaved, low traffic volume roads. Eighty two percent of the state's bridges are located on these county roads. This project concentrated on the unique problems associated with these low-volume road bridges.

The primary objective of this project was to develop a manual to assist the county engineer in making cost-effective bridge strengthening or replacement decisions. This manual includes several microcomputer software applications, which simplify the structural design and economic comparison of bridge replacement alternatives.

To perform a life cycle cost analysis of any civil engineering project, it is necessary to have a database of information available to estimate the service life and costs associated with a particular alternative. This manual has assembled a database of information for use in the economic analysis of low volume road bridges.

1.3. Scope of Investigation

The research project consisted of two phases; 1) the determination and prioritization of the critical problems on Iowa's secondary bridge system and 2) development of solutions for the problems identified in Phase 1.

Phase 1 required information related to structural deficiencies and functional obsolescence on both Iowa's county and municipal systems. To evaluate specific trends, the Iowa DOT's secondary structures and municipal structures computer tapes from January 1989 were obtained. Information on these tapes included structural type and material, age, serviceability, geometric data, and classification. After statistical trends were determined from the tape information, professional opinions on the scope of the Iowa county and municipality problems were obtained from several Iowa county and municipal engineers. A questionnaire was distributed to all ninety-nine counties and seventy-seven of Iowa's municipalities (all those with populations greater than 5000). While personal opinions were solicited from the questionnaires, a limited number were actually received. Therefore, opinions and additional insights were obtained from an advisory panel consisting of county and municipal engineers.

The conclusion from Phase 1 was that there are two bridge types, steel stringer and timber stringer, which make up the greatest percentage of problem bridges on the secondary road system. Therefore, Phase 2 focused on these two bridge types using field observations and statistical reviews of data. A design manual was developed to help evaluate strengthening and replacement options for these two bridge types.

This study investigated strengthening and rehabilitation procedures that can be used on low volume bridges. All strengthening procedures presented apply to the superstructure of bridges. The manual contains no information on the strengthening of existing foundations as such information is dependent on soil type and condition, type of foundation, and forces involved and, thus, is not readily presentable in a manual format.

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. To minimize misinterpretation of the meaning of strengthening, stiffening, and repairing, the research team's definitions of these terms are provided. In addition to these terms, the investigators' definitions of maintenance and rehabilitation, which are sometimes misused, are also given. The definitions given are not suggested as the best or only meanings for the terms but rather are the meanings of the terms as they are 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 eliminates 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).

In recent years, the Federal Highway Administration (FHWA) and National Cooperative Highway Research Program (NCHRP) have sponsored several studies on bridge repair, rehabilitation, and retrofitting. Inasmuch as 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 manual 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 include the following:

- *NCHRP Report 206*, "Detection and Repair of Fatigue Damage in Welded Highway Bridges," 1978 (*26*).
- *FHWA-RD-78-133*, "Extending the Service Life of Existing Bridges by Increasing their Load-Carrying Capacity," 1978 (*10*).
- *NCHRP Report 222*, "Bridges on Secondary Highways and Local Roads--Rehabilitation and Replacement," 1980 (*84*).
- *NCHRP Report 226*, "Damage Evaluation and Repair Methods for Prestressed Concrete Bridge Members," 1980 (*73*).
- *NCHRP Project 12-17 Final Report*, "Evaluation of Repair Techniques for Damaged Steel Bridge Members: Phase I," 1981 (*47*).
- *NCHRP Report 243*, "Rehabilitation and Replacement of Bridges on Secondary Highways and Local Roads," 1981 (*83*).
- *FHWA-RD-82-041*, "Innovative Methods of Upgrading Deficient Through Truss Buildings," 1983 (*68*).
- *FHWA-RD-83-007*, "Seismic Retrofitting Guidelines for Highway Bridges," 1983 (*7*).
- *NCHRP Reports 271*, "Guidelines for Evaluation and Repair of Damaged Steel Bridge Members," 1984 (*72*).
- *NCHRP Report 280*, "Guidelines for Evaluation and Repair of Prestressed Concrete Bridge Members," 1985 (*71*).
- *NCHRP Synthesis of Highway Practice 119*. "Prefabricated Bridge Elements and Systems," 1985 (*76*).
- *NCHRP Report 293*, "Methods of Strengthening Existing Highway Bridges," 1987 (*36*).

1.4. Research Approach

1.4.1. Task 1

The purpose of Task 1 was to obtain general information regarding bridge types and common bridge problems on low volume roads within Iowa. This included both county and municipal road systems. Information for compeletion of this task was obtained employing several methods: 1) review of National Bridge Inventory (NBI) for the state of Iowa, 2) meetings with several county and city engineering organizations, 3) meetings with an advisory panel consisting of city and county engineers, and 4) reviews of numerous bridges within the state using the Iowa DOT bridge embargo map as a guide. Chapter 2 provides details on the methods used and a summary of the findings.

1.4.2. Task 2

Task 2 included the identification of the types of bridges from Task 1 with the most problems and identification of the specific problem(s). Two primary methods were used to accomplish this task: 1) consultation with various municipal and county engineers (as well as the advisory panel) and several bridge engineering consultants in Iowa, and 2) questionnaires. Mr. Gordon Burns of Calhoun and Burns, served as a subcontractor for this study and provided extensive information for this task. Findings from this task are also summarized in Chp. 2 of this report.

1.4.3. Task 3

This task determined the methods of strengthening and/or rehabilitation that are most applicable to the types of bridges identified in Task 2. In addition, methods of replacement deemed to be most applicable for short spans were identified. This included both proprietary and nonproprietary replacement methods. To accomplish this task, an extensive review of existing literature was undertaken, as well as contacting colleagues who work in this technical area. Vendors of proprietary replacement bridges were also contacted to obtain pertinent technical literature. From this information, several types of replacement bridges were selected for inclusion in the design manual. Another important source of information for this task was the results from the questionnaires of Task 2. Respondents provided details on strengthening/rehabilitation methods that they had used effectively. The results of this task are presented in Chp. 3.

1.4.4. Task 4

Task 4 consisted of the development of a procedure for performing bridge strengthening and replacement decisions. The initial literature review indicated that a widely applied method of evaluating cost effectiveness of strengthening versus replacement considered the initial strengthening cost as a percentage of the initial replacement cost. Although this is a very basic approach for measuring cost effectiveness, determining the percentage at which replacement becomes a more cost-effective solution is a difficult procedure. Several different percentages were suggested in the literature review, each with little validation.

The method developed (and included in the manual) for evaluating the cost effectiveness of strengthening bridges is based on determination of Equivalent Uniform Annual Costs (EUACs), which are commonly used in engineering economy studies. The models and equations used to determine the EUACs for the strengthening and replacement alternatives met the requirements of a flexible approach for determining the cost effectiveness which includes life cycle costs and user benefits.

The EUAC models are presented in a generalized form that allows the manual user to introduce individualized cost data into the equations. However, as an aid to the manual user, cost data for most of the variables in the EUAC models are included in the manual. Detail on EUACs are provided in Chp. 3 of this report.

1.4.5. Task 5

For the information collected from the previous tasks to be useful for the practicing engineer, it must be organized and presented in a manual format. The development of such a manual was the objective of Task 5. Chapter 3 of this report is the technical manual on the application portion of this investigation. Section 3.1 contains general information on the scope and use of the manual. Sections 3.2 and 3.3 contain basic information to assist the engineer with inspections and fundamental bridge evaluation calculations. Economic analysis information is provided in Sec. 3.4. Design information for strengthening steel and timber stringer bridges is presented in Secs. 3.5 and 3.6, respectively. Bridge replacement alternatives are summarized in Sec. 3.7.

1.4.6. Task 6

The purpose of Task 6 was to prepare a final report documenting the research undertaken in this study. Since this report in part is the compilation of the research of three graduate students at ISU, the following references are cited for additional background information (11,40,74). As previously noted, Chp. 3 of the final report is the design manual; the other chapters and appendices provide supplementary and background information.

2. FINDINGS

This chapter summarizes the information assimilated in Tasks 1 and 2. To accomplish these tasks, the research team made numerous site inspections, held several meetings with the project advisory panel, reviewed the National Bridge Inventory (NBI) data for Iowa, developed, disseminated, and analyzed the results from a questionnaire, and made a literature review. A summary of the panel meetings is presented in Sec. 2.1. The summaries of the NBI data, questionnaires, and literature are presented in Secs. 2.2, 2.3 and 2.4, respectively.

2.1. Panel Meeting

The advisory panel was proposed and formed to assist the research team in making sure the project had the right direction and that the final results (i.e. the design manual) were practical and in such a format that they would be easy for practicing engineers to use. The advisory panel was comprised of representation from the Iowa DOT, county engineers, and municipal engineers. Listed below are the members of the advisory panel:

Dennis Gannon	Coralville Assistant City Engineer
Moe O. Hanson	Poweshiek County Engineer
Del Jesperson	Story County Engineer
Larry R. Jesse	Office of Local Systems, Iowa DOT
Nick R. Konrady	Lucas County Engineer
Richard Ransom	Cedar Rapids City Engineer
Fred M. Short	Audubon County Engineer

The panel meetings were especially beneficial in the development of the questionnaires and defining the scope of the project. Early in the project, through exchanges with the municipal engineers on the panel, it became apparent that the majority of their problems were beyond the scope of the project. Thus, the project proceeded primarily with the county engineer in mind, however, numerous sections of the design manual are equally applicable to certain municipal bridges.

As noted in the proposal, the input of consultants familiar with low volume bridge problems would also be contacted for input in the project. Upon acceptance by the advisory panel, Gordon E. Burns of the firm, Calhoun-Burns and Associates, Inc. (West Des Moines, Iowa) was contracted and worked closely with the research team in several areas of the project.

2.2. National Bridge Inventory

The NBI (56), now essentially complete, contains records from Structural Inventory and Appraisal (SI & A) sheet on bridges having spans of at least 20 ft, culverts of bridge length, and tunnels. Records are placed on the SI & A sheet in accordance to a FHWA coding guide (25). Based on results from a previous project (36) and spot checks of the Iowa NBI data, it has been determined that the NBI data are relatively free of obvious errors. There are some definite and some probable coding errors, however, those errors did not exceed 5 percent and often were less than 1 percent for the NBI items checked. Records having obvious errors or significant omissions were rejected thus improving the accuracy of conclusions based on NBI data.

The Iowa DOT's Secondary Structures and Municipal Structures Computer Tapes for January 1989 were reviewed to compile statistical information on Iowa's county and municipal bridge structural problems. This information is provided to the FHWA for inclusion in the NBI.

By reviewing Iowa NBI data, the number and type of bridges found on the county and municipal systems were determined. Figure 2.1 shows the ten most frequently occurring bridges on the secondary system by number and percentage. These ten FHWA types represent 90 percent of the 20,882 bridges on the secondary system. Note that close to 50 percent of the bridges are in two categories -- 28.0 percent steel stringer/multi-beam or girder [FHWA 302] and 20.8 percent timber stringer/multi-beam or girder [FHWA 702]. Approximately two-thirds of the bridges are in the first four categories. Table 2.1 provides the FHWA number key for identifying the bridge types identified in Fig. 2.1 and subsequent figures. Figure 2.2 shows similar information for the municipal system; the top ten bridge types represent 86 percent of the 1,308 bridges found on the municipal system. Approximately 30 percent of the bridges are in two categories -- 17.6 percent steel/multi-beam or girder [FHWA 302] and 12.2 percent concrete continuous slab [FHWA 201]. Slightly over 44 percent of the bridges are in the first four categories. By comparing Figs. 2.1 and 2.2 one observes that three of the top four categories on the two systems are the same.

Deficient bridges in the state of Iowa are characterized as either structurally deficient or functionally obsolete. These designations are based on data found in the NBI. Sufficiency ratings range between 0 and 100 percent. The three main variables used in the sufficiency ratings are structural adequacy and safety, serviceability and functional obsolescence, and essentially for public use. A bridge classified as structurally deficient and functionally obsolete with a SI&A sufficiency rating less than 50 percent is eligible for replacement with Federal bridge funds. While one classified as structurally deficient and functionally obsolete with a SI&A sufficiency rating between 50 percent and 80 percent, inclusive, is eligible for Federal rehabilitation funds.

Bridges were also reviewed according to the SI & A sufficiency rating. Of particular interest were those bridges with values below 50 percent, which are frequently considered structurally deficient. Figure 2.3 shows the top ten structurally deficient bridge types on the secondary system. Of the total of 5,372 structurally deficient bridges on the secondary system, the first four types [FHWA 702, 302, 380, and 310], account for 92 percent of all structurally deficient bridges. Figure 2.4 shows the top ten structurally deficient bridges on the municipal system. Note that the top four bridge types on the municipal system are the same bridge types as those on the county system, and make up 69 percent of 306 structurally deficient bridges. Based on this review, strengthening and/or rehabilitation procedures which apply to these four bridge types would be the most beneficial.

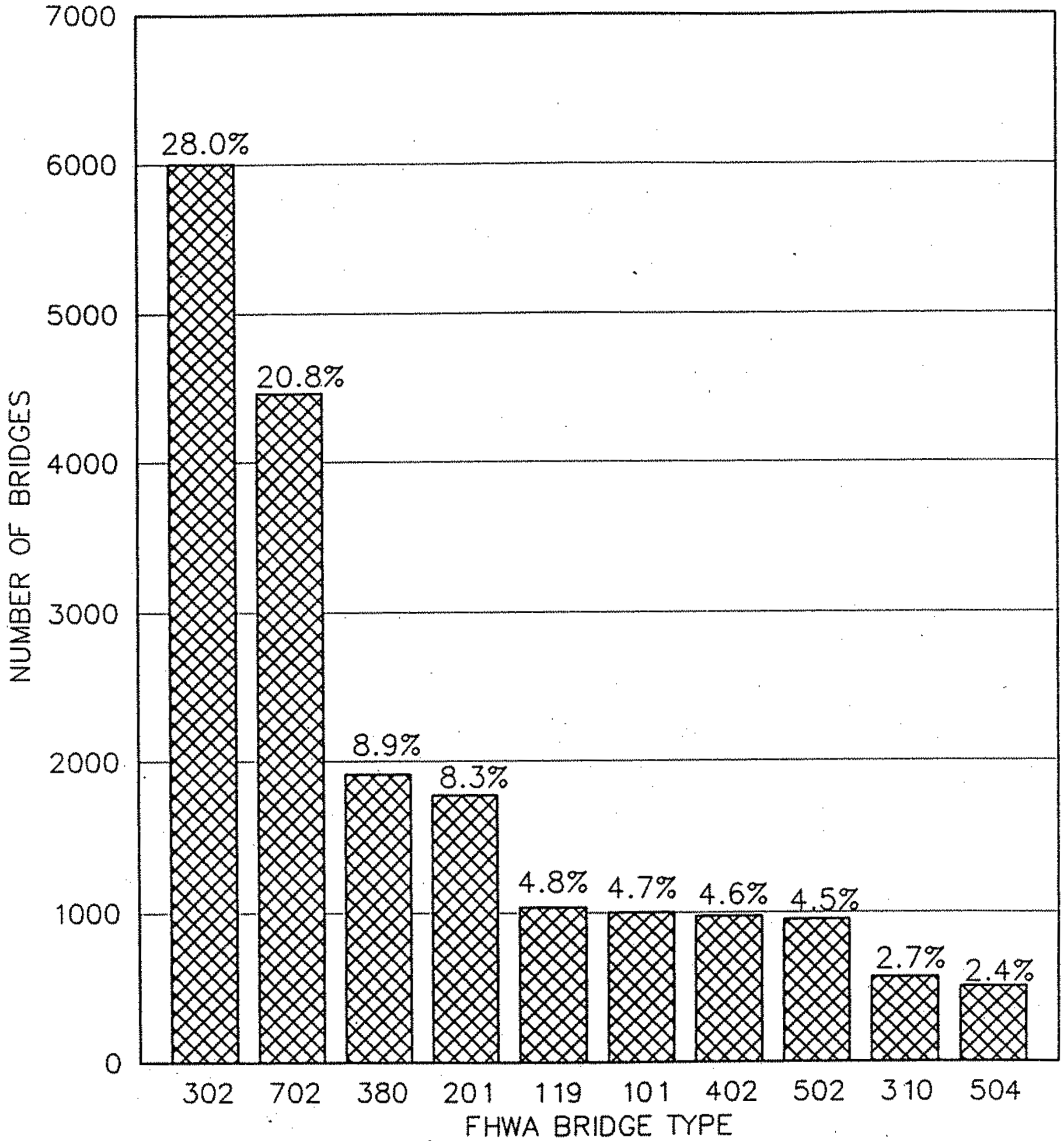


Fig. 2.1. Iowa county bridges by structure type.

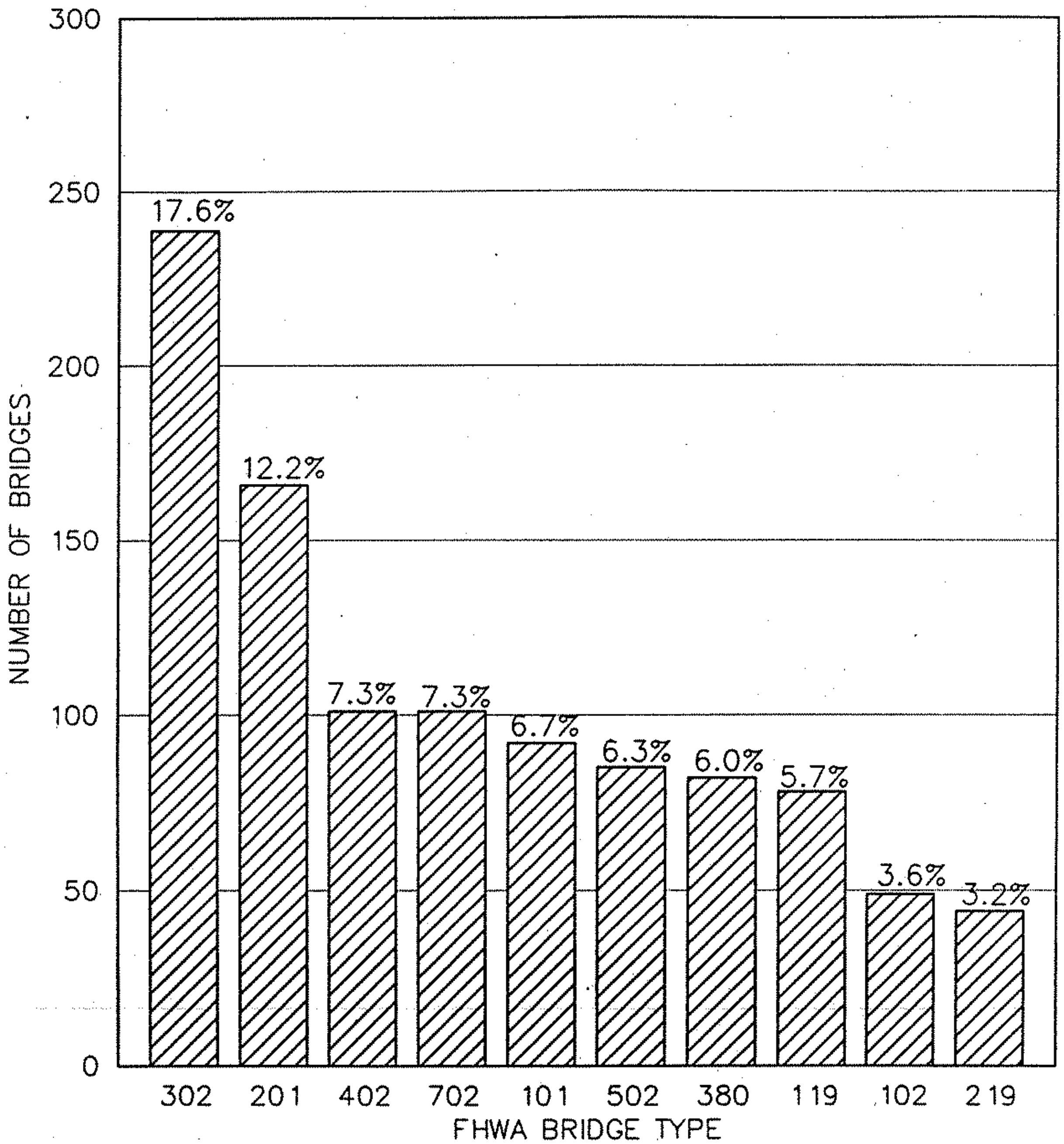


Fig. 2.2. Iowa municipal bridges by structure type.

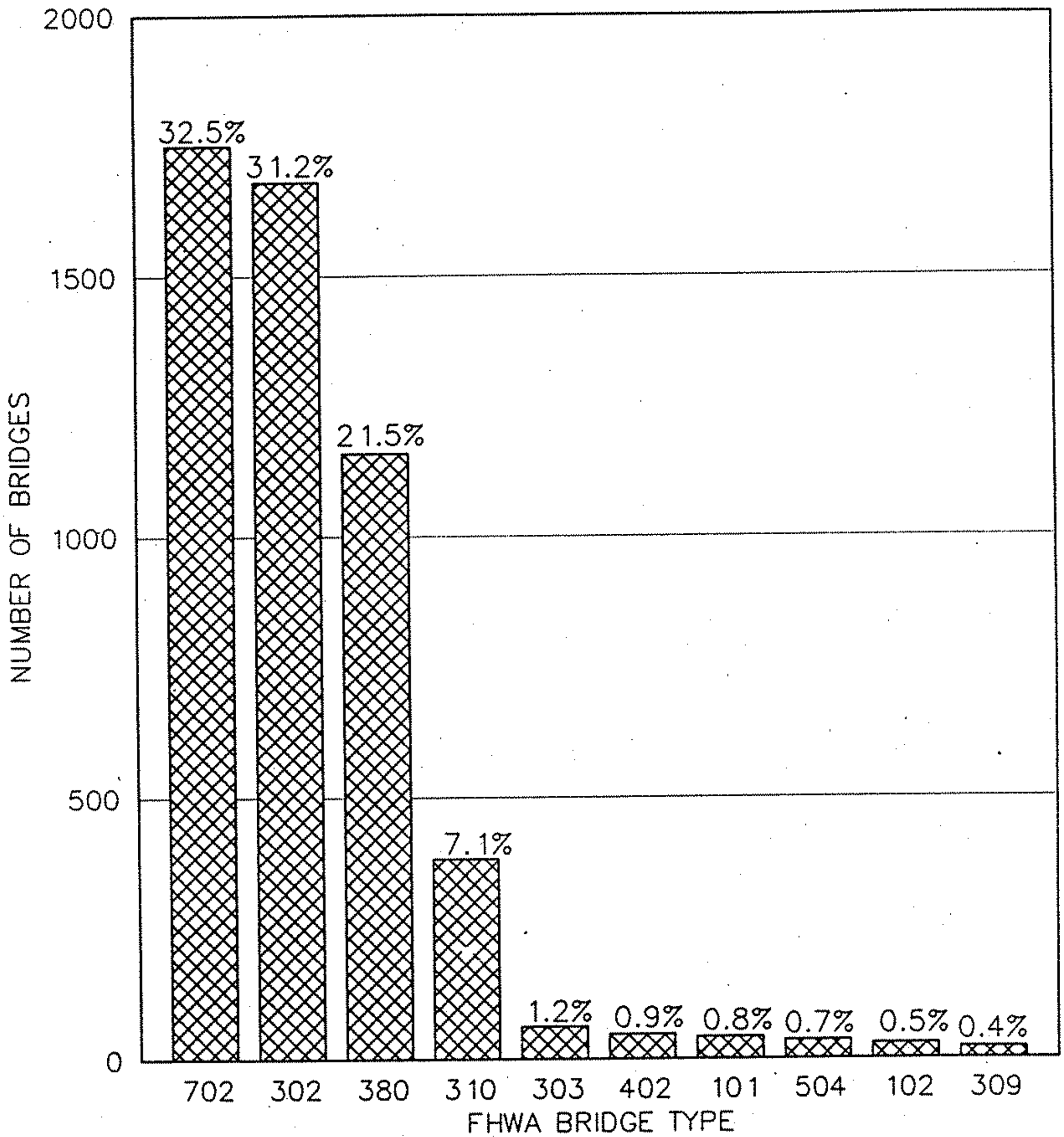


Fig. 2.3. Structurally deficient county bridges by structure type.

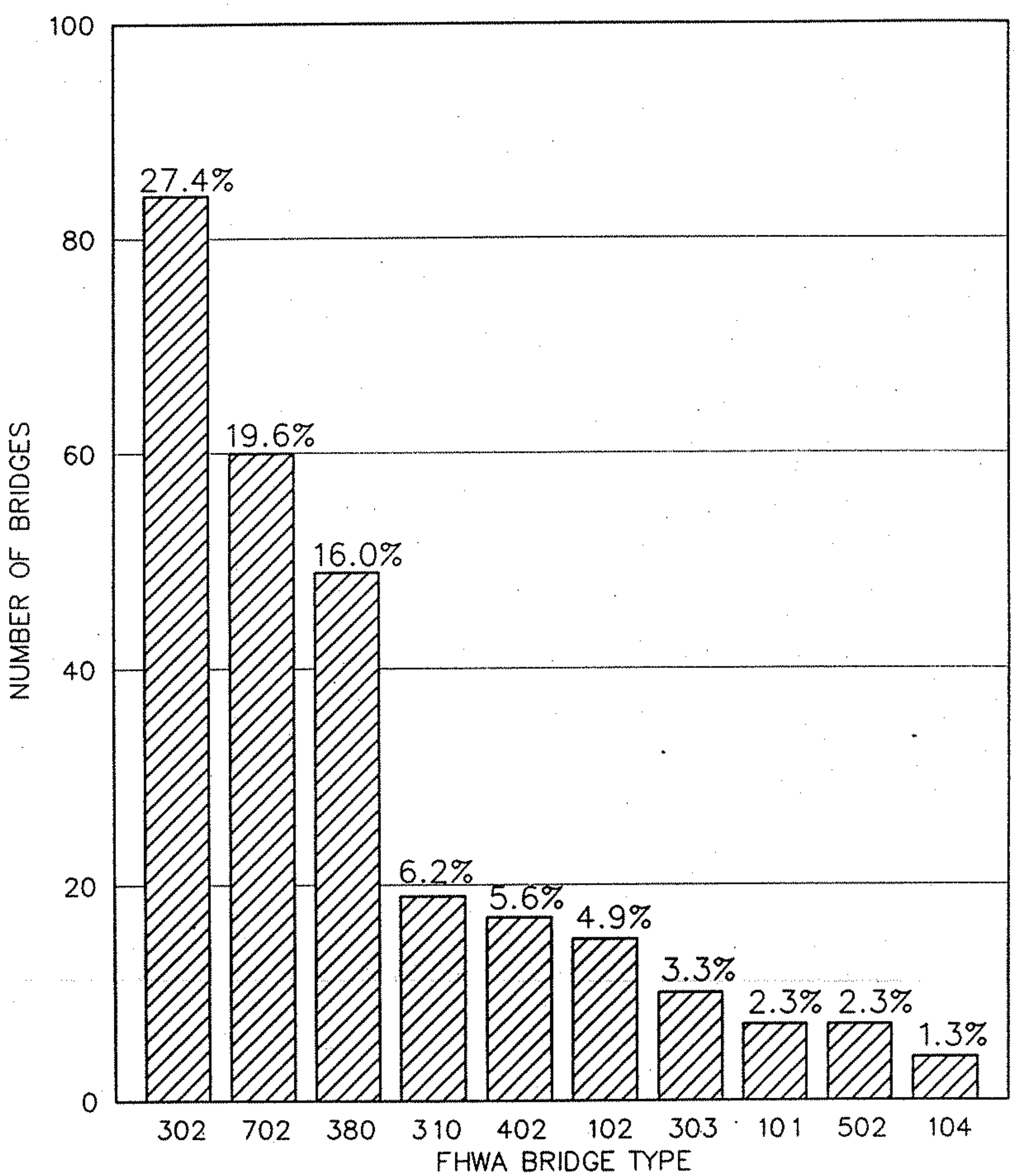


Fig. 2.4. Structurally deficient municipal bridges by structure type.

Table 2.1. FHWA bridge codes.

FHWA Designation	Descriptive FHWA Bridge Type Name
101	Concrete slab
102	Concrete stringer/multi-beam or girder
104	Concrete tee beam
119	Concrete culvert
201	Concrete continuous slab
219	Concrete continuous culvert
302	Steel stringer/multi-beam or girder
303	Steel girder and floor beam system
309	Steel truss-deck
310	Steel thru-truss
380	Steel pony truss
402	Steel continuous stringer/multi-beam or girder
502	Prestressed concrete stringer/multi-beam or girder
504	Prestressed concrete tee beam
702	Timber stringer/multi-beam or girder

Also reviewed for comparison were the functionally obsolete bridges. As Fig. 2.5 illustrates, FHWA bridge types 302 and 702 were the top two functionally obsolete bridges on the secondary system, representing 69 percent of all functionally obsolete bridges. On the municipal system, more FHWA 302 bridges were found to be functionally obsolete than any other type of bridge (see Fig. 2.6). Steel stringer and timber stringer bridges account for over 47 percent of the bridges found on the two systems and also make up the highest percentage of structurally deficient and functionally obsolete bridges. Thus, the greatest percentage of bridges which will most likely become structurally deficient and functionally obsolete in the future will be these two bridge types. Unique serviceability requirements, high ADT for example, may require replacement of some of these bridges; however a large percentage would benefit from strengthening. In other words, possibly 63.7 percent of all structurally deficient and 59.0 percent of all functionally obsolete bridges in the state of Iowa are potential candidates for rehabilitation.

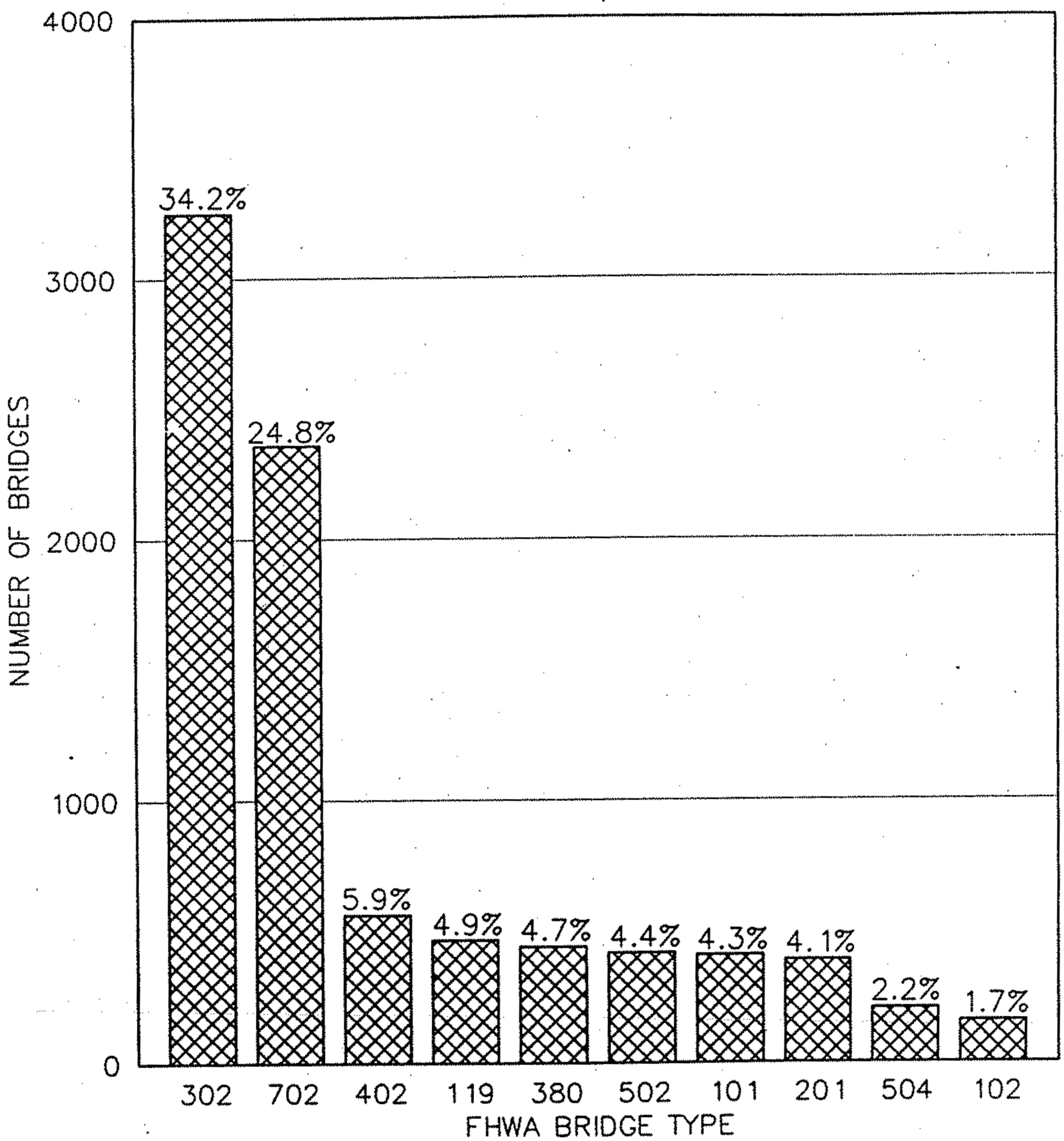


Fig. 2.5. Functionally obsolete county bridges by structure type.

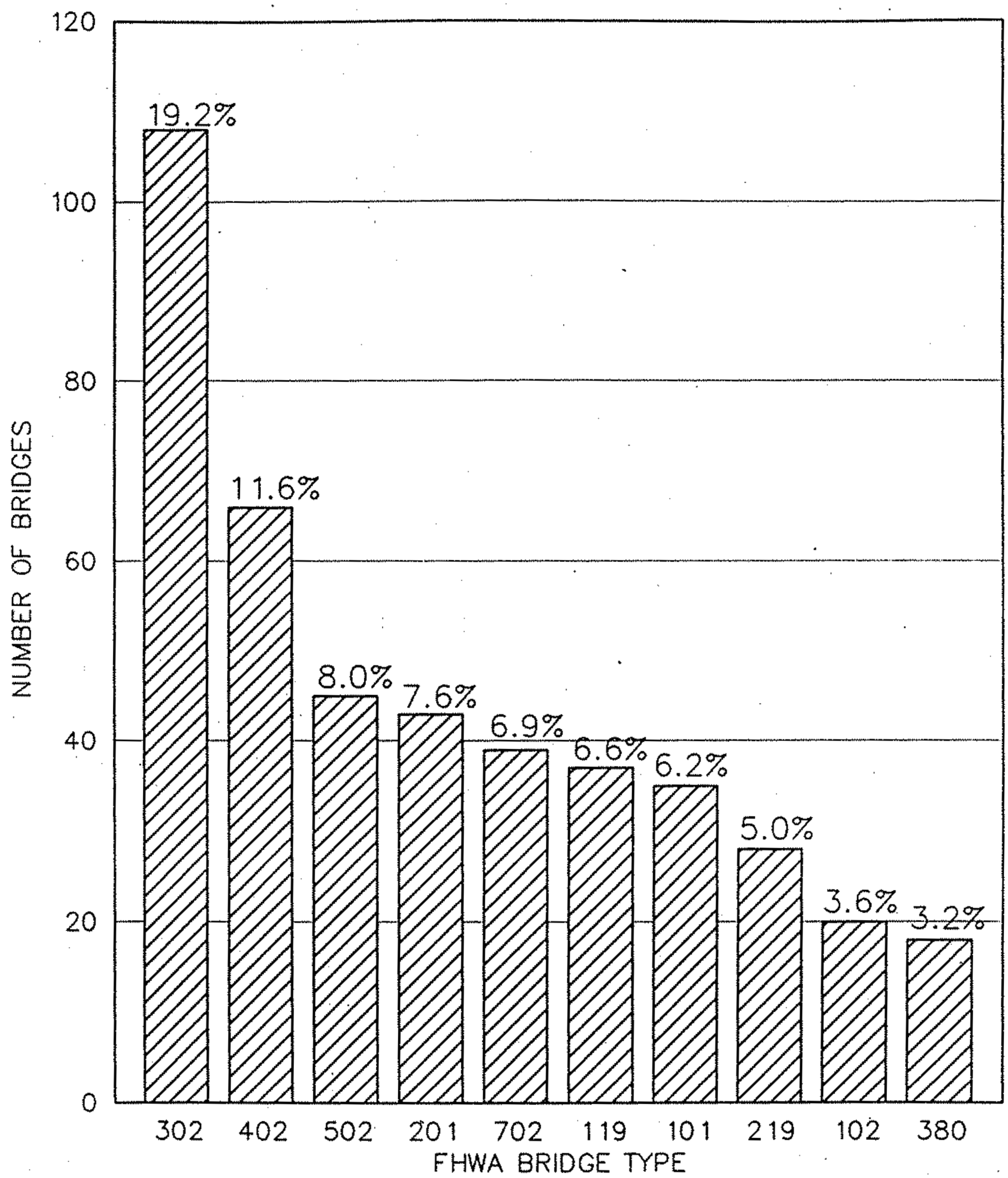


Fig. 2.6. Functionally obsolete municipal bridges by structure type.

2.3. Questionnaire Results

With the assistance of the advisory panel, questionnaires were developed to determine the strengthening and rehabilitation needs of Iowa's counties and municipalities. Although two questionnaires were prepared -- one for each group -- the questionnaires were essentially the same except for some of the wording; an example of the questionnaire sent to counties may be found in Appendix C.

Each of Iowa's 99 counties and 77 of Iowa's municipalities, those with populations greater than 5000, were sent questionnaires. The county response rate was 88 percent; while the municipal response rate was 75 percent.

In the questionnaires, low volume bridges were defined as those bridges with an ADT of 400 or less. The questionnaires encouraged the inclusion of supplemental information, comments and/or suggestions. Since responsibilities of the counties are different from those of the municipalities, responses from each were compiled separately. The questionnaires were divided into two sections. Completion of Section 2 of the questionnaire was required only if the responding agency had strengthening or rehabilitation experience.

The purpose of Section 1 of the questionnaire was to determine the Iowa county's/municipality's experience with bridge strengthening and bridge rehabilitation. The questionnaire defined rehabilitation as including bridge replacement. As Fig. 2.7 indicates of the counties responding, 43.6 percent had implemented at least one strengthening method; 81.4 percent had rehabilitated/replaced a bridge.

Fewer municipalities had attempted to strengthen bridges than counties. Of all municipalities responding, 14.3 percent had strengthened bridges, and 52.6 percent had employed a rehabilitation/replacement method. It should be noted that 40 percent of all the municipalities either had no bridges, did not have any bridges with ADT's less than 400, or lacked a situation which could benefit from strengthening. The primary reason given by counties for not strengthening a bridge was that the deck geometries still would not meet state width specifications.

Figure 2.8 illustrates those reasons given by the various agencies for not strengthening and/or rehabilitating/replacing bridges. The indication is that counties, which are responsible for approximately 16 times as many bridges as municipalities, would benefit more from useful guidelines for bridge strengthening and replacement. Several respondents indicated that strengthening/rehabilitation had not been used because of the lack of appropriate expertise.

Questions in Section 2 of the questionnaire were designed to identify the current bridge strengthening and replacement procedures most often used by county/municipal engineers. Table 2.2 summarizes the responses to the questions in Section 2 which required a yes/no answer. When asked if any type of economic analysis was performed in making decisions, respondents noted that decisions were controlled by budget constraints, structural deficiency priority systems, and the needs of the public, thus making an economic analysis less effective.

Responses to Question 2 of Section 2 of the questionnaire indicated five counties have developed their own bridge rehabilitation decision tools which included:

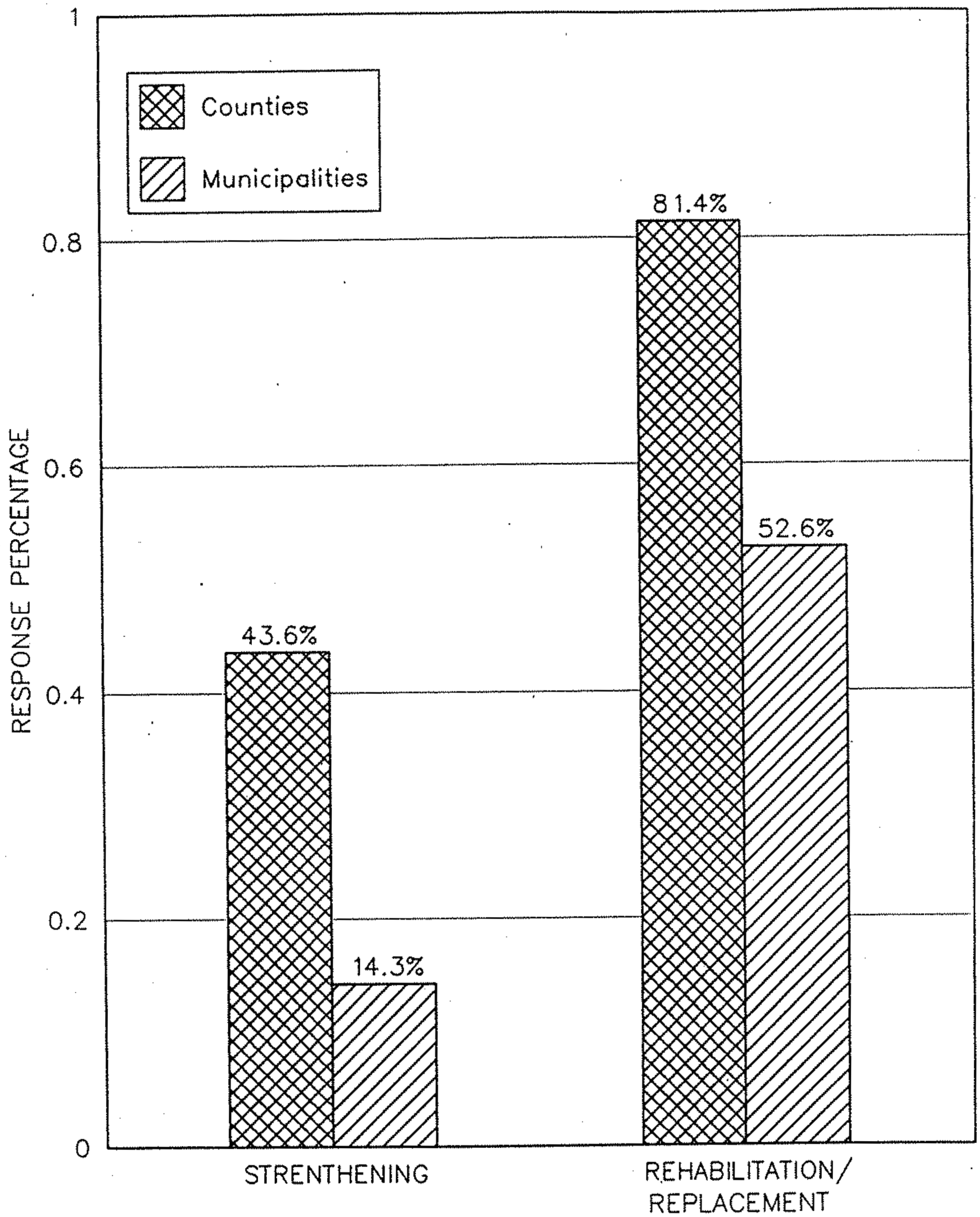


Fig. 2.7. Experience of local agencies with bridge strengthening and replacement.

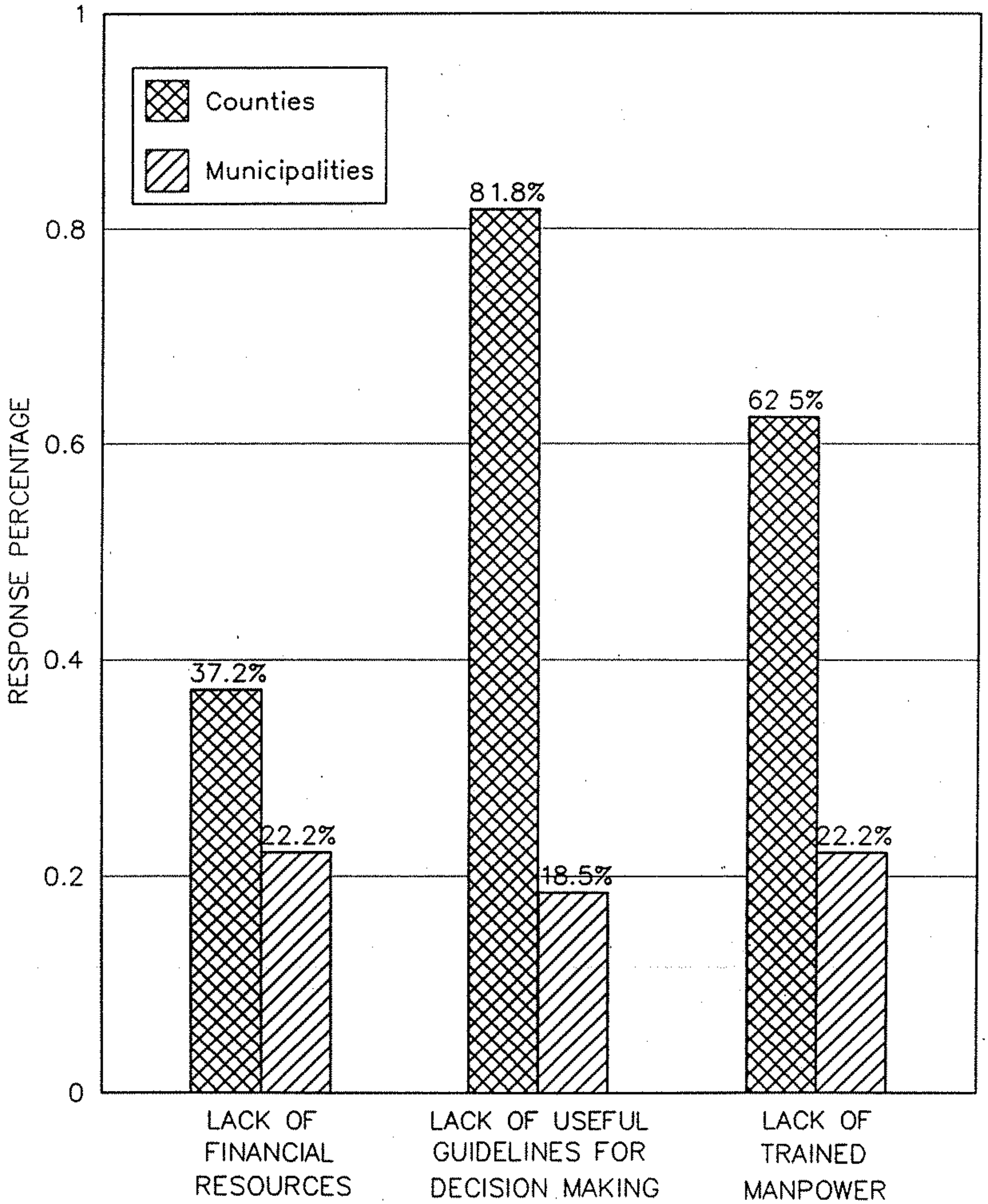


Fig. 2.8. Reasons for not implementing strengthening.

- a bridge rating sheet,
- graphs for determining beam spacing,
- tables for determining maximum spacing for various sized timber stringers to meet current legal load capacities for all wood bridges,
- a simple span bridge rating program used to assist in rehabilitation decisions, and
- charts indicating span lengths and stringer requirements for carrying fully legal loads.

Table 2.2. Summary of Section 2 questions.

Questions	County		Municipalities	
	Yes	No	Yes	No
1. Do you use formal methodologies (e.g. benefit/cost analysis, equivalent annual cost method, etc) when making management decisions?	21	55	10	23
2. Have you developed any design aids, nomographs, software, etc. that are useful in making bridge rehabilitation choices?	5	13	10	32
3. Does your agency hire any structural engineering consultants?	68	10	30	3
4. Would your county/municipality benefit from a design aid or decision making tool?	62	9	21	9
5. Are you familiar with the National Cooperative Highway Research Program Report #293, Methods of Strengthening Existing Highway Bridges?	20	56	2	31

A tabulation of the number of counties and municipalities which used the services of structural engineering consultants is included in Table 2.2; the specific services performed by the consultants are presented in Table 2.3.

Of those responding to Question 4 of Section 2 (see Table 2.2), county approval was 87 percent and municipal approval was 70 percent in favor of the development of decision making tools or rehabilitation/strengthening design aids. Given a list of "tools" from which the agencies would most likely benefit, 81 percent of the counties listed computer software development, 52 percent requested nomographs; and 23 percent requested flow charts. Other "tools" counties specified as being beneficial were plans, cost comparison documentation of rehabilitation versus replacement, a maintenance manual (similar to the one used in Florida which outlines approved repair practices), and a design manual (similar to the one used in California which outlines design values and techniques).

Of the municipalities responding who favored design "tools", 67 percent requested computer software, 52 percent requested nomographs, and 52 percent requested flow charts. One municipality noted that design aids are not necessary since they are a political entity and the insurance liability would be too great; another municipality noted they will always use a structural engineering consultant for bridge problems.

To determine the strengthening procedures with which counties/municipalities had experience, agencies were asked to identify procedures they had employed on the four most common structurally deficient types of bridges. The number of responses by counties were: addition or replacement of timber stringers -- 50; addition or replacement of steel stringers -- 31; and lightweight deck replacement in timber stringer bridges -- 17.

Table 2.3. Summary of services for which agencies employ consultants.

Consulting Service	Counties	Municipalities
Structural analysis	61	27
Bridge inspection	52	26
Strengthening or rehabilitation	24	20
New or special bridge designs	11	7
Construction inspection	3	17
Load rating	1	0
Culvert design	1	0
Underwater inspection	0	1

Municipality responses were: strengthening of existing members on steel pony and through trusses -- 6; all other methods yielded fewer than 3 responses. Responses to the "other" category were given very infrequently. Agencies which had employed strengthening methods were asked to indicate which of these methods were perceived to be cost effective and structurally effective. Counties noted that the two most cost effective strengthening methods were increasing transverse stiffness and providing composite action; the two methods perceived as the most structurally effective were the addition or replacement of members and the strengthening of existing members. Municipalities noted the most cost effective methods were the addition or replacement of various members, the strengthening of existing members, and the strengthening of critical connections (equal number of responses for each.) The two structurally effective strengthening methods noted were the strengthening of existing members and the strengthening of critical connections. The addition or replacement of various members was also indicated as being very effective. As expected, those methods which were perceived as being very costly or structurally ineffective were the methods which have been employed the least.

It was suggested that if it were not cost effective to increase the capacity of a given bridge to current loading standards, a compromise could be reached where the bridge could be strengthened to a specified increased load. Counties specified in such a case the load they would desire a bridge to carry is 19.1 tons; this value was obtained by averaging all reported values which ranged between 12 tons and 30 tons. The municipalities specified 16.4 tons (obtained by averaging reported values) as the desired capacity; reported values ranged between 10 tons and 20 tons.

The National Cooperative Highway Research Program Report 293, Methods of Strengthening Existing Highway Bridges (36), reviews and describes current strengthening techniques used on existing highway bridges. Only 26 percent and 6 percent of the counties and municipalities, respectively, noted that they were familiar with this report.

Question 12 on the questionnaire asked the respondents to prioritize the top four deficient bridges into three categories: 1) the type of bridges which need to be strengthened, 2) those bridges which would most benefit from a combination of strengthening and posted weight/speed restrictions, and 3) those bridge types which are least likely to benefit from strengthening or rehabilitation methods. Responses by both counties and municipalities indicated that steel stringer bridges would benefit the most from either strengthening or a combination of strengthening and posted weight/speed restrictions.

In summary, a significant percentage of counties are currently employing strengthening methods, although a limited number of methods are being utilized. Replacement decisions typically tend to be sound economical and structural decisions based on current information available. It appears that part of the hesitation to strengthen a given bridge is due to lack of adequate information and the bridge's inability to meet required deck geometries. Both counties and municipalities indicated a rehabilitation/strengthening "tool" or design aid is desirable.

The number of bridges per municipality is considerably less than the number of bridges per county. Apparently the reason municipalities tend not to undertake their own strengthening and replacement designs is the high cost of liability insurance. However, while counties also employ a large number of consultants, they are more likely to do some of their own engineering because of the large number of bridges for which they are responsible and budgetary constraints.

Data from the Iowa NBI, questionnaire responses and input from the advisory panel influenced and directed the second portion of this investigation (Tasks 3-6). Based on information obtained and reviewed in the initial tasks of this investigation, it was determined that strengthening procedures and techniques which are applicable to the steel stringer bridges and timber stringer bridges found on low volume roads would be the most beneficial to practicing engineers. A more detailed summary of the findings of Tasks 1 and 2 are presented in Ref. 94.

The manual (Chp. 3) thus provides practical strengthening methods for these two types of bridges and numerous spreadsheets to assist the engineer in designing various strengthening systems.

2.4. Literature Review

2.4.1. General

A literature search was conducted to gather available information on strengthening/rehabilitation of low volume bridges. Computerized literature searches were made using the Highway Research Information Service through the Iowa DOT and the Engineering Literature Index System which is available at the university library. In addition to searching these two sources, the Geodex System -- Structural Information Service was used to locate additional pertinent references.

The literature search revealed that minimal work has been done on the general subject of strengthening, rehabilitating, or replacing low volume bridges. Most of the research which has been reported has been directed toward one specific type of bridge or set of circumstances.

Literature was located on strengthening/rehabilitating of essentially all types of bridges. However in this brief literature review, only information on the two types of bridges previously identified -- steel stringer bridges and timber stringer bridges -- as having the greatest potential for being strengthened/rehabilitated will be presented.

It should be noted at the outset that much of the information related to replacement of low volume bridges is not located in the published literature, but rather in the form of proprietary publications by private companies. In most cases, these replacement designs are developed on a case-by-case basis. The engineer submits his site requirements (span length, bridge width, load capacity, and aesthetic considerations) and the predesigned bridge is shipped to the site essentially complete. These proprietary designs will be discussed in more detail in Chp. 3.

The current AASHTO design specifications (2) do not distinguish between low volume rural bridges and high volume urban bridges. Gangarao and Zelina (28) have suggested that a set of design specifications and procedures be developed specifically for low volume bridges. They note that it is highly unlikely that efficient and economical low-volume bridges can be designed using specifications that were compiled primarily for highway bridges. Similar thoughts have been expressed by Galambos (27) who suggested specifying rules for a lower level of service for non-Federal aid bridges. Alternatives which allow flexibility for site conditions as well as a proposed fatigue model (both of which would allow less structural loading) have been proposed by Moses (48).

In Chp. 1, various FHWA and NCHRP final reports related to bridge strengthening, repair, rehabilitation, and retrofitting were noted. One of these, NCHRP Report 293 (36) is particularly pertinent to this study in that it pertains to strengthening highway bridges. This report reviews strengthening techniques used in the United States as well as in several foreign countries and contains a bibliography with 379 references which review the strengthening of all types of bridges. Strengthening information in this report is organized by strengthening procedure rather than by bridge type as some strengthening procedures are applicable to several bridge types. Strengthening techniques/procedures in this report were classified into eight categories:

- Lightweight deck replacement
- Addition of composite action
- Increasing transverse stiffness of the bridge
- Improving the strength of various bridge members
- Adding or replacing members
- Post-tensioning of various bridge components
- Strengthening of connections
- Developing additional bridge continuity

As previously stated, this literature review is intended primarily to review strengthening techniques which are applicable to timber and steel stringer bridges. This investigation also has collected and reviewed information on numerous replacement structures; as previously noted this information will be presented in Chp. 3.

2.4.2. Timber Bridges

The literature review revealed minimal strengthening procedures for timber stringer bridges. Only three procedures were found -- replacing deteriorated or damaged stringers, reducing existing dead load on the structure by replacing the decking with a lightweight deck or reducing the amount of "fill" on the bridges, and reducing the stringer spacing by adding additional stringers to the bridges. In addition to these possibilities, the research team has developed a strengthening procedure in which a limited number of timber stringers are replaced with steel stringers. This technique is presented in Sec. 3.6.2.

In recent years, there has been an increase in the use of timber in the transportation field. Significant interest in the construction of several timber bridges has developed; some of the techniques and procedures used in new construction can also be used to strengthen existing timber bridges in some situations (12).

Throughout the United States, numerous short span timber bridges are in need of deck rehabilitation. The majority of these decks were nail-laminated. Due to traffic loading and the effect of the environment, these fasteners have loosened over the years. Until recently, the United States Department of Agriculture Forest Service has been unsuccessful in attempts to rehabilitate timber bridges. Between 1965 and 1975, the Forest Service attempted to strengthen existing timber bridges with the application of transverse A36 steel rods. This procedure proved unsuccessful because the prestress force could not be maintained with the ordinary steel rods (44).

The use of lateral load distribution devices has generated significant research. These include distributor beams (69) or several methods of compressing longitudinal timber decks perpendicular to the grain. One method which has shown much promise is the use of high strength steel rods positioned perpendicular to the direction of traffic (58,79). These rods are tensioned against steel bearing plates along the outside edges of the bridge. The friction between the deck timbers induced by this tensioning eliminates inter-laminar slippage and provides substantial lateral load distribution.

2.4.3. Steel Bridges

Steel girder bridges, which have a relatively small ratio of dead to live load, are especially affected by an increase in live load. The strengthening techniques found in the literature for steel stringer bridges essentially all fall in the following six categories:

- Lightweight deck replacement.
- Improving the strength of the stringers.
- Increasing transverse stiffness.
- Adding or replacing members.
- Providing composite action.
- Post-tensioning.

The techniques will only briefly be discussed here as there is a very comprehensive literature review of these strengthening procedures in Ref. 36. To assist the reader in locating reference material on the various strengthening procedures, section numbers and page numbers for Ref. 36 have been provided.

Lightweight deck replacement [Ref. 36; Sec. 2.3.1; p 18]: the live-load capacity of a bridge can be improved by replacing an existing heavyweight deck with a new lightweight deck. A review of the literature reveals that several structurally adequate lightweight decks are available, including steel grid, exodermic, timber, lightweight concrete, aluminum orthotropic plate, and steel orthotropic plates. Each of these will be briefly discussed in the following paragraphs.

Steel grid deck is a lightweight floor system manufactured by several firms. It consists of fabricated, steel-grid panels that are field welded or bolted to the bridge superstructure. In application, the steel grids may be filled with concrete, partially filled with concrete, or left open.

Exodermic deck is a newly developed, prefabricated modular deck system that is being marketed by major steel-grid-deck manufacturers. The bridge deck system consists of a relatively thin upper layer (3 in. minimum) of prefabricated concrete jointed to a lower layer of steel gratings.

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 and are secured to each other with steel dowels or stiffener beams to allow for load transfer and to provide continuity between panels.

Structural lightweight concrete can be used to strengthen steel bridges that have normal-weight, noncomposite concrete decks (43). Lightweight concrete (unit weight of 115 lb/cu ft or less) can be either cast in place or installed in the form of precast panels. Cast-in-place lightweight concrete decks can be made to act compositely with the stringers.

Aluminum orthotropic deck is structurally strong, lightweight deck weighing between 20 and 25 lb/sq ft. This proprietary decking system is fabricated from highly corrosion-resistant aluminum alloy plates and extrusions that are shop coated with a durable, skid-resistant, polymer wearing surface. Connections between the aluminum orthotropic deck and the steel stringers should not be considered to provide composite action.

Steel orthotropic plate decks are an alternative for lightweight deck replacement, however they are usually designed on a case-by-case basis with essentially no standardization. Although steel orthotropic deck is applicable for short spans, it is unlikely that there would be sufficient weight savings to make it economical.

Improving the strength of the stringers [Ref. 36; Sec. 2.3.4; p 21]: One of the most common procedures used to strengthen existing bridges is the addition of steel cover plates to the existing stringers. Steel cover plates, angles, or other sections may be attached to the stringers by means of bolts or welds. 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. When angles are employed to strengthen stringers, they are usually attached to the webs of the stringers with high strength bolts. In most cases, the member is jacked up during the strengthening process, relieving dead-load stresses on the existing stringers. The resulting cover-plated section will resist both live-load and dead-load stresses when the jacks are removed. If the stringers are not jacked, the added cover plates will carry only live-load stresses.

Increasing transverse stiffness of a bridge [Ref. 36; Sec. 2.3.3; p 20]: Much of the literature on transverse stiffness of a bridge deals with the effects of diaphragms and cross frames on transverse stiffness rather than strengthening of a bridge by increasing the transverse stiffness. Literature indicates that increasing transverse stiffness will be most effective for interior stringers and will have essentially no effect on exterior stringers. Increasing transverse stiffness should be considered a secondary method of strengthening a bridge. In most practical cases, the stress reduction resulting from transverse stiffening is less than 30 percent; in some cases, it may even be negligible.

Adding or replacing members [Ref. 36; Sec. 2.3.5; p 23]: Steel stringer bridges can be strengthened by the addition or replacement of one or more stringers. Adding stringers will increase the deck capacity and reduce the magnitude of the loads distributed to the existing stringers. This method is most practically performed in conjunction with replacement of the deck because this allows respacing of the existing stringers. Stringer replacement is more typically a repair technique that is used when a stringer has been damaged by an overheight vehicle or corrosion. The addition or replacement of a stringer is more difficult when the existing deck is not removed. Installation of the new stringer is usually carried out from below the bridge and is usually a difficult procedure.

Providing composite action [Ref. 36; Sec. 2.3.2; p 20]: Modification of an existing system is a common method of increasing the flexural strength of a bridge. This procedure can be used when a deteriorated concrete deck is removed and replaced with a new deck or when the existing deck is sound by coring holes through the deck, adding shear connectors, and grouting the holes. The composite action of the stringers and deck not only reduces live-load stresses but also reduces deflections as a result of the increase in the moment of inertia resulting from the stringers and deck acting together.

Post-tensioning [Ref. 36; Sec. 2.3.6; p 24]: Prestressing or post-tensioning in various configurations has been used for more than 30 years to relieve stresses, control displacements and strengthen bridges. Through research sponsored by the Iowa DOT, Iowa State University (ISU) has developed post-tensioning procedures for strengthening simple-span as well as continuous-span steel-stringer bridges. As has been earlier documented, the majority of the steel-stringer bridge problems on the low volume roads are with single-span bridges; thus the strengthening of continuous span bridges will not be discussed in this report.

For the practicing engineer, ISU has developed a post-tensioning strengthening manual (24) that will assist the engineer in determining the post-tensioning force a given bridge needs to reduce flexural stresses the desired amount. In most instances, only the exterior beams need post-tensioning. Lateral distribution of the post-tensioning forces inmost situations also reduces the stresses in the interior stringers.

2.4.4 Cost Effectiveness Studies

Several studies have been performed on the cost effectiveness of various bridge strengthening or rehabilitation methods. In 1985, Cady (13) developed a policy for the decision making process in bridge deck rehabilitation. An economic model, based on the present worth of perpetual service or the capitalized cost of each alternative was developed. This analysis may be extended to apply to essentially any rehabilitation project.

A study at Pennsylvania State University (90) developed a flow chart of rehabilitation methods for highway bridges. A survey of state bridge and maintenance engineers determined the type and effectiveness of various maintenance and rehabilitation procedures. These procedures were subjected to a life cycle cost analysis to determine a range of expected unit costs for the various methods. A flow chart was then developed which would allow a maintenance engineer to select the most cost effective rehabilitation method based on the amount of deterioration, etc. This type of flow chart was found to be very useful to local engineers.

The use of incremental benefit-cost analysis has been used by many agencies to aid in the decision making process. This method identifies the optimum alternatives and also prioritizes them. In public projects, measuring benefits in monetary terms poses a problem. One has to estimate the value of benefits, both for the agency and for the traveling public from available sources (64).

The use of the benefit-cost ratio method to compare possible bridge rehabilitation or replacement alternatives has traditionally been avoided due to the difficulty in quantifying the benefit of the proposed improvement. One study which attempted to alleviate this difficulty was performed by the New York DOT Planning Division. To use this procedure, one needs only the posted speed, the average running speed, the traffic count with some estimate of vehicle mix, and highway section length for both the before and after conditions (41); with this data, the operating and travel time costs of the alternatives are calculated. Maintenance and accident costs of the alternatives are not considered in this model.

There has been a significant amount of research on the use of value engineering (50) in the design and construction of low volume road bridges. Gangarao, et. al. (29) used value engineering, – that is, the systematic application of recognized techniques which identify the function of a product or service, establish a monetary value for that function, and provide the necessary function reliably at the lowest overall cost -- to develop value graphs, which relate the importance of various bridge components to their costs.

One major problem with this procedure noted in the literature was the difficulty in predicting the service life of the various strengthening methods. This obstacle is somewhat mitigated by two factors: 1) as service life increases, variation in service life has a diminishing effect on calculated equivalent cost and 2) if the average service lives of relatively short-lived procedures are reasonably well known, rather large variations will have relatively little effect over the long run.

Engineering economic analyses have historically ignored the effects of inflation. It has been thought that inflation affects all aspects of cash flow in the same manner, thus its net effect on the decision making process is negligible. The 1973 oil embargo produced a significant change in the effect of inflation. A marked reduction in fuel consumption caused a drastic reduction in gasoline tax revenue. At the same time, the cost of construction increased dramatically due to the rising rate of inflation. Inflation has affected income and disbursements in opposite directions, creating a situation where engineering economic analysis must take the effects of inflation into account (14).

MANUAL for EVALUATION, REHABILITATION and STRENGTHENING OF LOW VOLUME BRIDGES

(Chapter 3)

3. APPLICATIONS

As has been previously noted, Chp. 3 of this report is a strengthening manual. The other chapters provide supplementary and background information. Section 3.1. provides general information about the manual. Secs. 3.2 and 3.3 provide basic information about bridge inspection and fundamentals of analysis, respectively. Section 3.3 is particulary informative because ten bridge evaluation examples have been provided. Economic analyses to assist in the decision to strengthen or replace a given bridge is presented in Sec. 3.4. The remaining three sections of this chapter (3.5 to 3.7) present various strengthening methods and replacement alternatives.

3.1. General Information

3.1.1. Background

As previously noted, the statistical information from the Secondary System Section of the NBI served as the basis for determining the type of bridges that require strengthening. For additional clarification of NBI data, various categories of steel stringer bridges found on the Iowa DOT's county system base inventory record were reviewed. The program Syncsort was used to sort, extract and summarize information. The year each bridge was built was reviewed to determine if a correlation existed between the year a particular Iowa DOT V-series standard bridge was issued and the year a given FHWA 302 bridge was constructed; no clear correlation was found. Site inspections of several Iowa steel stringer bridges revealed that the majority of these bridges actually were not constructed in accordance with any of the Iowa DOT V-series.

Lengths and widths of existing FHWA 302 bridges were also reviewed. The average length of the FHWA 302 bridges found on the Iowa county system is 50.6 ft. Figure 3.1 illustrates that a majority of the bridges have lengths between 20 ft and 45 ft. Although, as shown, a number of bridges had lengths over 100 ft, more than likely these bridges are the result of coding errors. As may be seen in Fig. 3.2, the majority of the bridges have widths of 20 ft and 24 ft. However, a significant number of bridges (over 200 in each case) have widths of 16 ft and 18 ft.

Two other parameters of interest in the Iowa DOT's county system base inventory record were the design loading and the design H-loading. The design loading refers to Item 31 in the SI&A data (see Sec. 2.2.); this one digit code represents the design live load for the structure. Instructions for coding this item require classifying the loading, if it is other than standard loading, as the nearest equivalent standard loading. As shown in Fig. 3.3, in the majority of cases (72.1 percent) the design load is not known; in 20.6 percent of the cases the design load was classified as H15. Over 93 percent of the bridges fall into these two categories of loading (H15 or unknown). The design H-loading of the bridge was obtained from maintenance records. The number of bridges designed for each of the five standard design H-loadings is shown in Fig. 3.4. Note that the design loading for the majority (71.0 percent) of bridges is unknown (NC).

As noted in Chp. 2, it was determined that strengthening techniques are primarily required for noncomposite, simply supported steel stringer bridges [FHWA 302] and timber stringer bridges [FHWA 702].

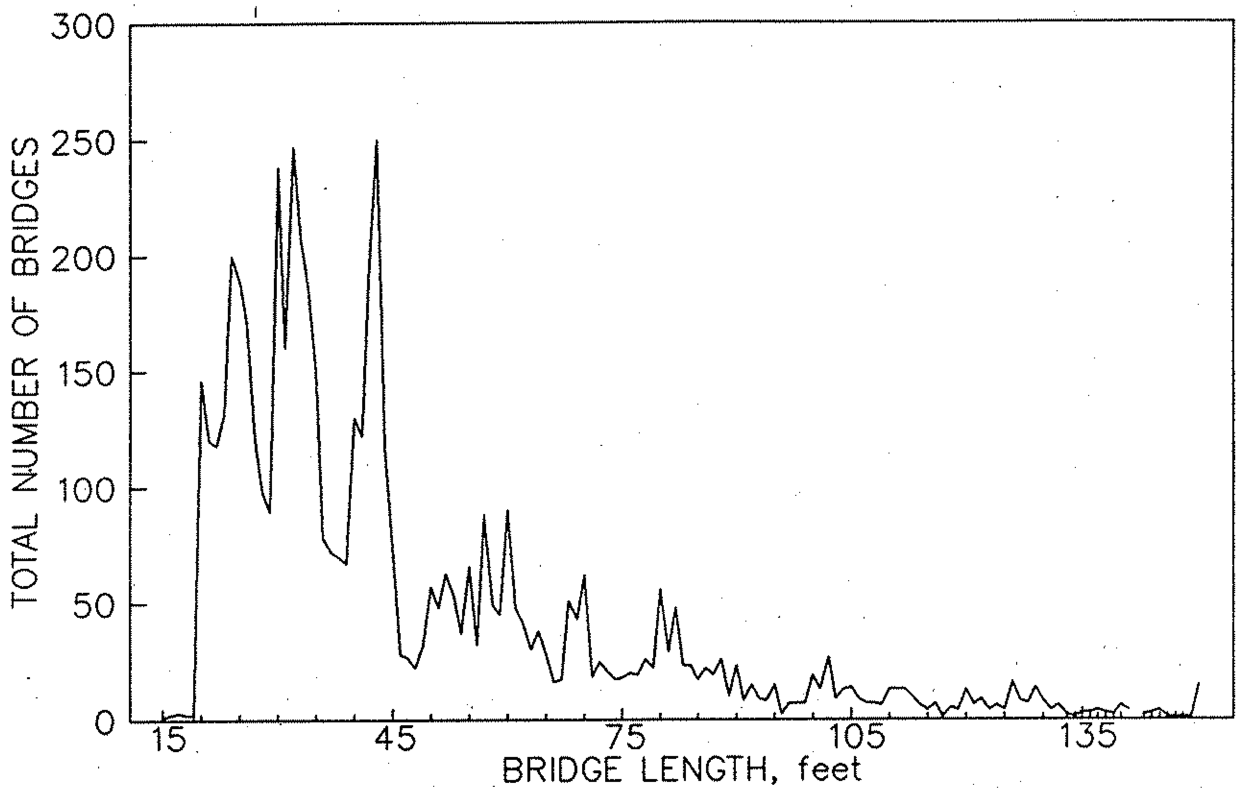


Fig. 3.1. Lengths of steel stringer bridges: FHWA 302.

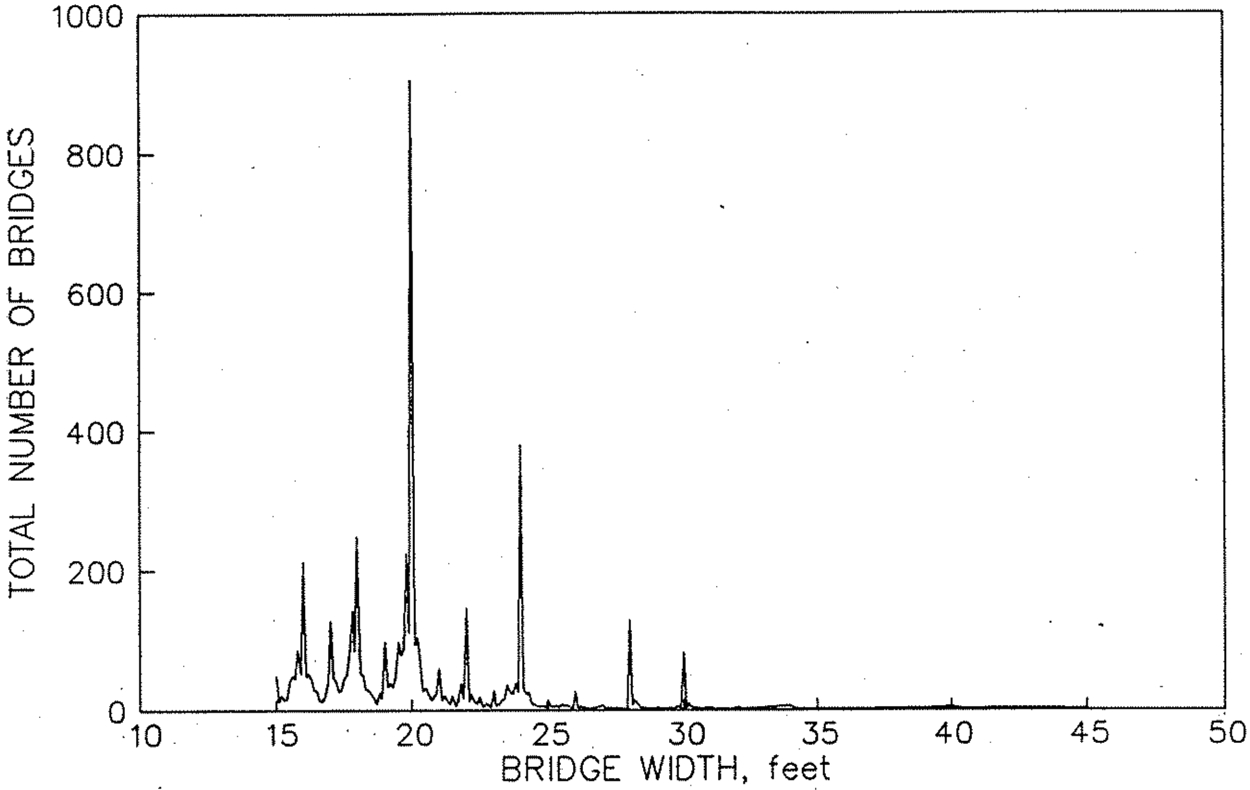


Fig. 3.2. Widths of steel stringer bridges: FHWA 302.

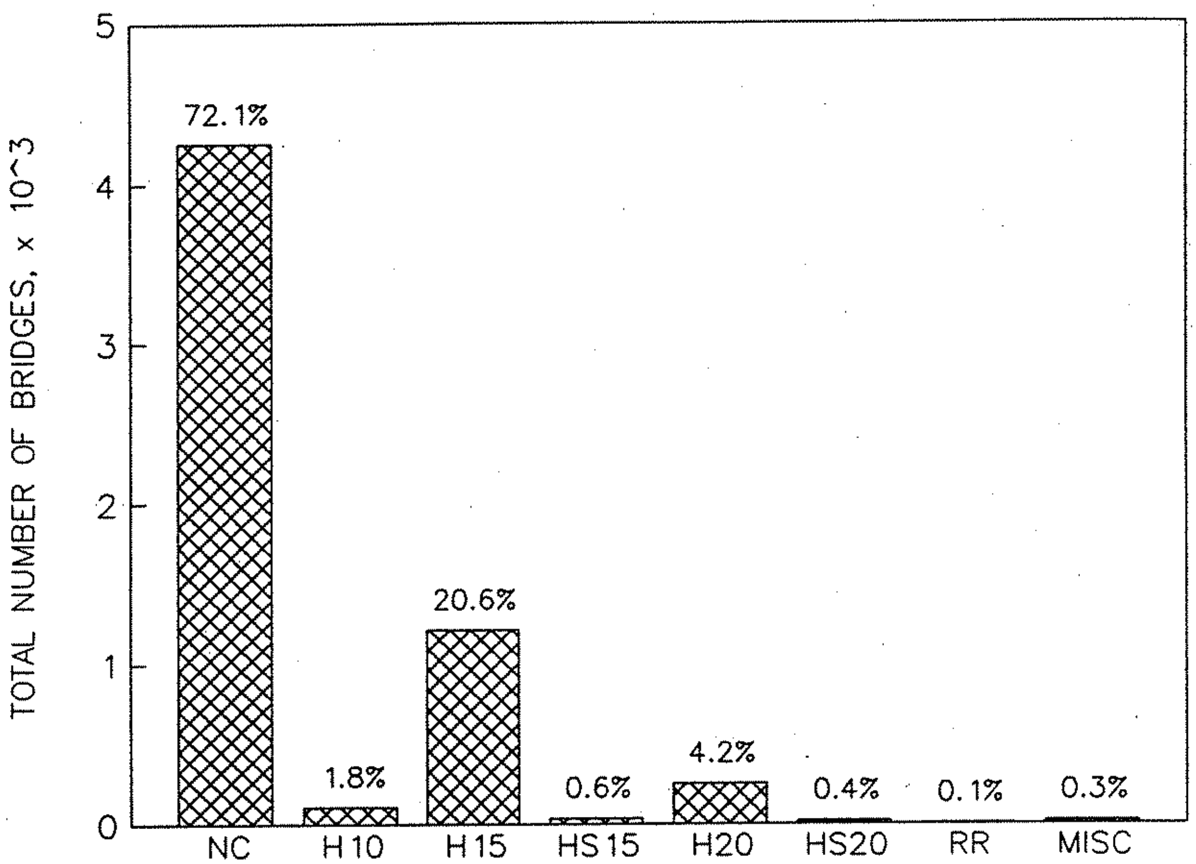


Fig. 3.3. Design load cases: FHWA 302.

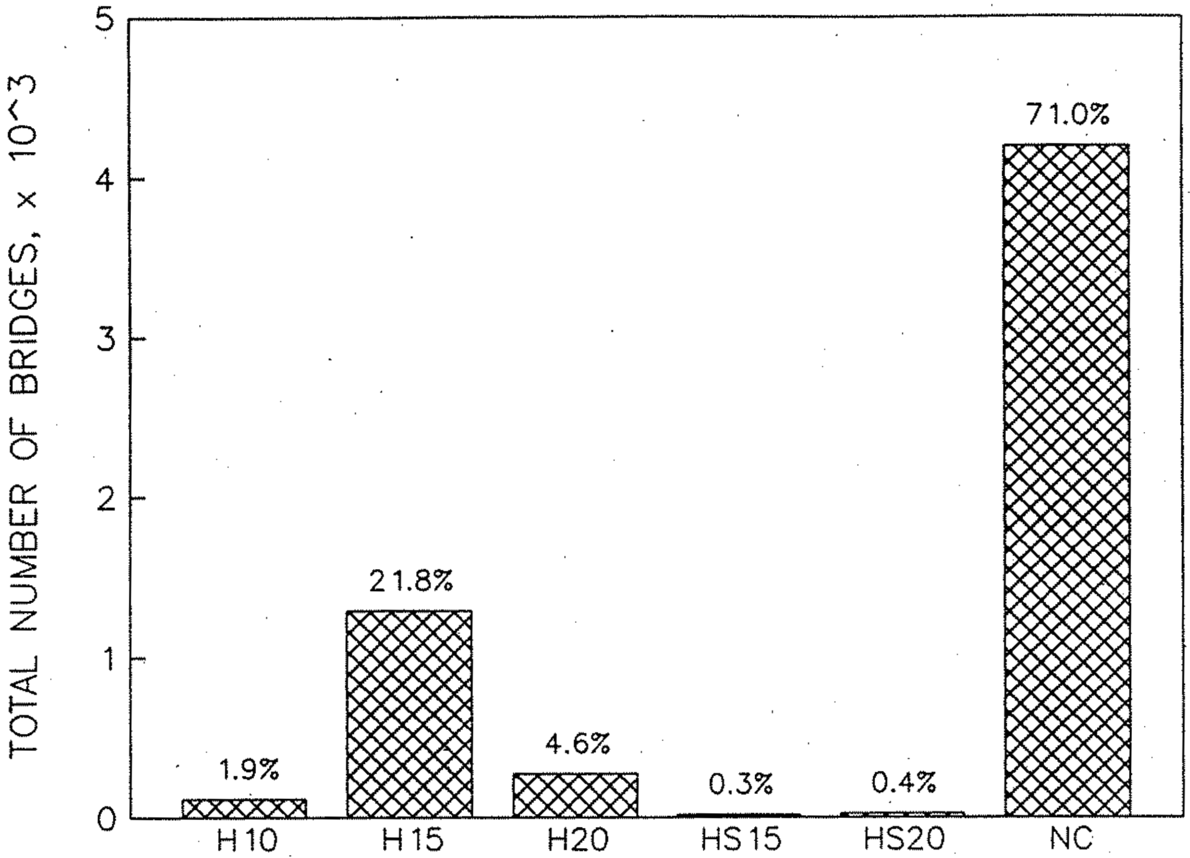


Fig. 3.4. Design H load cases: FHWA 302.

Thus, this manual is limited to these two bridge types. One exception to this limitation is the economic analysis presented in Sec. 3.4; this analysis is applicable to any type of structure.

The economic analysis procedure assists the user in determining if it is more cost effective to replace a given bridge or to strengthen it. Assuming it is more cost effective to strengthen it, the same analysis procedure can also be used to determine which strengthening procedures are the most cost effective. The economic analysis procedure developed makes use of an equivalent uniform annual cost analysis approach and considers factors such as annual maintenance costs, initial construction costs, service lives, interest rates etc. The various strengthening techniques and procedures have been organized and presented in this chapter according to bridge type. Only strengthening and replacement procedures that have been used successfully in the field are included in this chapter. In the next few years, as a result of extensive bridge strengthening research currently in progress, and the development of new materials, several new strengthening procedures most likely will be developed; these could then be easily added to this manual. In addition, as databases become available as part of the development of bridge management systems (BMS), that data will be extremely useful in the economic model developed in this study.

In the replacement section of this chapter, several proprietary products have been presented. Inclusion of such products in this manual does not constitute a recommendation by the research team or by the Highway Division of the Iowa DOT.

3.1.2. Scope of Manual

No special considerations have been accorded skewed bridges. However, the strengthening techniques presented are applicable to bridges with a small degree of skew.

All strengthening methods presented apply to the superstructure of bridges. No information is included about the strengthening of existing foundations, since such information is dependent on soil type and condition, type of foundation, forces involved, etc., and thus is not readily presentable in a manual format. Before initiating the strengthening of the superstructure of a given bridge, however, one must investigate the substructure to determine if it is of adequate strength or it also requires strengthening. Some of the strengthening procedures presented illustrate field welding applications. However, field welding in certain situations is not the best practice. In older bridges, the type of steel is frequently unknown and thus the weldability of the steel is also unknown. In these situations, bolted connections should be used unless laboratory tests are undertaken to determine the steels' weldability. Even when it is determined that the steel involved is weldable, welding should not be used in locations where it would lower the fatigue resistance of the original structure. Since the bridges in question have low ADT, fatigue is rarely a problem. However, strengthening details which may create such problems should obviously be avoided. Potential welding problems are noted several times in the presentation of the various strengthening techniques.

As previously noted, strengthening procedures are presented by bridge type. In the following sections the strengthening procedures presented later in this chapter are briefly described. Replacement bridge types are also listed in the following sections and presented in detail later in this chapter

Care has been taken to provide the background for the design procedures for strengthening and replacement alternatives. Design procedures and calculations are provided, or references are listed, to provide appropriate technical information for a proper analysis and design. The manual user is encouraged to thoroughly understand the analysis and design application before performing the final design.

3.1.3. Use of Manual

The design manual contains information for the bridge engineer ranging from the inspection procedure to the design of strengthening and/or replacement alternatives. Table 3.1 outlines the major sections of the manual. It is recommended that the user become familiar with the manual's contents prior to using it. The step by step progression of the evaluation and design process as outlined in this manual includes: (1) bridge inspection, (2) bridge evaluation (load capacity analysis and, if applicable, desired load capacity goal), (3) selection of strengthening method and/or replacement bridge type, (4) analysis and design of items in step 3, and (5) economic analysis of step 3 alternatives.

Section 3.2 related to bridge inspection is presented to provide important information to perform an accurate evaluation. Inspection is possibly the most important step in the complete evaluation process and observations can effect the calculated load capacity of the bridge.

Section 3.3 provides fundamental information for calculating the bridge rating based on inspection data. The evaluation procedure is outlined and references are provided to assist the engineer in obtaining pertinent information. Sample calculations are provided to clearly illustrate various evaluation procedures.

Table 3.1. Contents of design manual listed by primary section.

3.2.	Inspection
3.3.	Fundamentals
3.4.	Economic Analysis
3.5.	Strengthening Techniques for Steel Stringer Bridges: FHWA 302
3.6.	Strengthening Techniques for Timber Stringer Bridges: FHWA 702
3.7.	Replacement Bridges

Computer spreadsheets are provided in this manual to perform these evaluation calculations and provide a load rating, load posting and a maximum SI&A rating.

Once the load rating os the bridge has been determined, a bridge capacity goal must be selected by the engineer. A strengthening analysis and/or replacement analysis should then be performed once suitable strengthening methods and bridge replacement types are identified. Sections 3.5 and 3.6 of the manual contain various applicable strengthening methods for steel stringer and timber stringer bridges, respectively. Based on the information in this manual, an analysis and design can be performed to achieve the desired bridge capacity goal. Computer spreadsheets have been provided for some of the strengthening techniques to assist this process. Section 3.7 contains various bridge replacement alternatives that can also achieve the desired bridge capacity goal. Both design information and cost data, where applicable, have been provided to assist the engineer. In addition, computer spreadsheets are presented for three timber bridge types to assist this process.

After a strengthening method (or methods) and/or replacement type have been selected and designed to achieve desired load capacity goals, an economic analysis should be performed. This analysis provided in this manual involves determination of the Equivalent Uniform Annual Cost (EUAC) of each alternative. Section 3.4 presents procedures for determining EUAC's. A computer spreadsheet is also presented in this manual to perform an EUAC analysis.

The final step in the decision making process is to compare EUAC's for each alternative. In addition, any unusual problems that may have an influence on this decision making process, and is not quantifiable for calculating the EUAC, should be considered at this time. After all of these factors have been reviewed, a decision should be made as to which replacement or strengthening method, or combination of methods, should be applied.

3.1.3.1. Applicable Strengthening Techniques for FHWA 302 Bridges

As previously noted in Chp. 2, there are six well tested strengthening techniques which have been used to increase the live load carrying capacity of noncomposite simple span steel stringer bridges:

1. Replacement of damaged stringers.
2. Respace existing stringers and adding stringers.
3. Increase section modulus of steel stringers.
4. Develop composite action.
5. Replace existing deck with a lightweight deck.
6. Post-tensioning.

These techniques will be presented in Sec. 3.5. Design examples and spreadsheets are presented for techniques 2, 3, and 6.

3.1.3.2. Applicable Strengthening Techniques for FHWA 702 Bridges

Although the NBI provides quantitative information about bridge systems, it does not contain specific information about the structural properties of bridges. For example, the presence of a concrete deck is noted, but not the spacing of stringers. Thus, to benefit as many bridges as possible, strengthening solutions have been kept general.

While several alternatives for strengthening timber stringers exist, their benefit to cost ratios are low with the exception of two procedures: Respace and add procedure and addition of steel stringers. These two alternatives as well as evaluation techniques will be presented.

3.1.3.3. Replacement Bridges

Eleven replacement bridge types have been selected for presentation in this design manual. They include:

1. Precast culvert/bridge.
2. Air formed arch culvert.
3. Welded steel truss bridge.
4. Prestressed concrete beam bridge.
5. Inverset bridge system.
6. Precast multiple tee beam bridge.
7. Low water stream crossing.
8. Corrugated metal pipe culvert.
9. Stress laminated timber bridge.
10. Glue-laminated timber beam bridge.
11. Glue-laminated panel deck bridge.

Some of these replacement bridges are proprietary. In these cases, general information regarding design criteria and cost information have been provided. Additional references where more detailed design procedures already exist have been included for the manual user. Three timber bridge replacement types have been included. Design examples and spreadsheets are presented for replacement bridge types 9, 10 and 11.

3.1.3.4. Microcomputer Spreadsheets

Responding to the requests of a large percentage of engineers who completed the questionnaires, computer spreadsheets have been developed for performing various evaluation, strengthening, and replacement calculations. The primary advantages of computer spreadsheets for these applications are:

- Most county offices have personal computers which are capable of running spreadsheets.
- Spreadsheet templates can be arranged to follow the normal design process completed by hand.
- When changes are made in any part of the input, the corresponding changes in all the calculations are made automatically.
- Spreadsheet templates can include the necessary tables for beam properties, allowable stresses, etc. which reduces the calculation time. The software developed in this report is Lotus 1-2-3 release 2.3 (42), which includes the @VLOOKUP function (i.e. Vertical Lookup). This allows easy extraction of desired values from stored tables.
- Design and revision of spreadsheets is much quicker than conventional programming. Thus, engineers may take an existing spreadsheet and expand or modify it to better suit their specific needs.

One disadvantage of spreadsheets is that regeneration time increases as the number of equations increases and the spreadsheet size expands. The number of calculations in the spreadsheets developed in this report are small so that is not a problem.

Spreadsheets are comprised of labeled rows and columns. Numbers represent rows and columns are represented by letters. The intersection of a row and column is referred to as a cell; see example spreadsheet in Fig. 3.5. Cells may include either numerical information, text or commands.

	A	B	C	D
1	STEEL STRINGER EVALUATION SPREADSHEET			
2	INPUT			
3	YEAR BUILT	A		
4	BRIDGE LENGTH (FEET)	B		
5	BEAM SPACING (FEET)	C		
6	SECTION MODULUS (INCH^3)	D		
7	DEAD LOAD (KLF)	E		
8	FLOOR TYPE FROM BELOW	F		
9	1 = 4" THICK TIMBER			
10	2 = 6" OR MORE THICK TIMBER			
11	3 = CONCRETE			
12				
13	OUTPUT			
14	LOAD TO POST ON LOAD LIMIT SIGN			
15	TRUCK	2 LANES	1 LANE	
16	HS20			
17	TYPE 3			
18	TYPE 3S2(a)			
19	TYPE 3S2(b)			
20	TYPE 4			
21	TYPE 3S3			
22	TYPE 3-3			
23				
24				
25	INTERMEDIATE VALUES			
26	ALLOWABLE STRESS (KSI)		G	
27	OPERATING STRESS (KSI)		H	
28				
29	MOMENT CAPACITY (KIP-FT)	INVENTORY	OPERATING	
30	TOTAL STRUCTURE			I
31	DEAD LOAD			J
32	LIVE LOAD			K
33				
34	LIVE LOAD DISTRIBUTION FACTORS			
35	TWO LANES OR MORE		L	
36	ONE LANE		M	
37				
38	IMPACT FACTOR		N	
39				
40	LIVE LOAD + IMPACT MOMENT		O	
41	TRUCK	2 LANES	1 LANE	
42	HS20			
43	TYPE 3			
44	TYPE 3S2(a)			
45	TYPE 3S2(b)			
46	TYPE 4			
47	TYPE 3S3			
48	TYPE 3-3			
49				
50	BRIDGE LOAD IN TONS			P
51	TRUCK	2 LANES	1 LANE	MAX LOAD
52	HS20			20.00
53	TYPE 3			25.00
54	TYPE 3S2(a)			36.50
55	TYPE 3S2(b)			40.00
56	TYPE 4			27.25
57	TYPE 3S3			40.00
58	TYPE 3-3			40.00
59				
60	MAX S.I. & A. RATING		Q	
61	TRUCK	2 LANES	1 LANE	
62	HS20			
63	BEST S.I. & A. SUFFICENCY RATING			

Fig. 3.5. Example spreadsheet.

In Fig. 3.5, cells B3 through B8 represent the input for a specific bridge; these cells have been highlighted in the spreadsheet. Cells B16 through B22 and C16 through C22, also highlighted on the spreadsheet, represent the posting values associated with Iowa legal trucks.

A list of the spreadsheets presented in the manual, and their location for various strengthening and replacement methods and economic analysis is presented in Table 3.2.

Table 3.2. Spreadsheets in design manual.

Spreadsheet Application	Primary Function	Location in Design Manual (Section No.)	Spreadsheet Identification
Economic Analysis	Analysis	3.4.4	1
Steel Stringer Evaluation	Analysis	3.5.2	2
Post-tensioning	Strengthening	3.5.6	3
Timber Stringer Evaluation	Analysis	3.6.1	4
Stress Laminated Timber Bridge	Replacement	3.7.9	5
Glue-Laminated Timber Beam Bridge	Replacement	3.7.10	6
Glue-Laminated Panel Deck Bridge	Replacement	3.7.11	7

3.2. Inspection

Prior to an engineer's determination of whether a bridge is a suitable candidate for rehabilitation is the performance of a thorough inspection of the key elements of the structure. This inspection should be made by a person with at least a general understanding of how loads are distributed through a bridge, a knowledge of what members are main load-carrying elements and what constitutes capacity-reducing damage and deterioration.

Since 1973, bridges have been routinely inspected on a two-year cycle. Therefore, the SI&A reports on file should indicate which elements of a given bridge warrant closer examination. The information provided on an SI&A report is not always in a narrative form, but rather is numerically coded with frequently sketchy comments. A complete report will include structural calculations with a summary of operating, inventory and posting loads, if they are required.

The most current SI&A report, as well as all previous reports and plans of the bridge in question (if available), should be reviewed in the office prior to the field inspection. The inspector should have this same information available during the actual field inspection.

If the previous inspections have been carefully performed and the data properly and correctly recorded, the measuring of members and the determination of general dimensions is not necessary each time a given bridge is inspected. However, it is recommended that the inspector carry a scraper and/or wire brush and calipers to determine the current status of corrosion and section loss in the case of steel members.

With the most recent SI&A report in hand, the inspector should verify the current condition of previously recorded damage or deterioration. Since this manual deals primarily with simple-span superstructures, it is assumed that the substructures are sound or can be repaired by practicable, cost-effective methods pursuant to repairing and rehabilitating the superstructure.

The principal elements for review are the deck, stringers, bearings, and their ability to interact. The inspector should keep in mind that the primary purpose of this inspection is to view the bridge from the aspect of determining its general condition and suitability for repairs and rehabilitation. That is, can the life of the bridge be extended for an appreciable (cost-effective) length of time? Can the load posting be eliminated or increased to carry school buses, farm equipment, etc.?

3.2.1. Decks

A large number of low volume bridges in Iowa are of timber construction (see Sec. 2.2), having stringer elements ranging generally from 3 in. x 12 in. to 6 in. x 18 in. spaced for the utilization of 3 in. x 12 in. or 4 in. x 12 in. decking laid transverse to the stringer alignment. Another popular timber deck system is to position 3 in. and 4 in. thick by generally 6 in. to 12 in. wide planks transversely on the stringers on 12 in. to 18 in. centers and then provide appropriate sized planks longitudinally for what is often called a "deck and a half".

Decks on timber-stringer bridges are usually well spiked to the stringers which, in turn, should have spacer blocks or X-bracing at bearings and at mid-span to resist transverse "rolling". A timber bridge constructed in this manner is of little concern regarding the stability of the stringer compression zone.

The method of attaching timber decks to steel stringers/beams is a common area of neglect, which often leads to the need for load posting. The inspector should review closely the connection between the deck and stringers. ***The deck functions as a lateral support for the top flange of beams and stringers*** and thus must be appropriately attached.

The allowable stress used by the engineer in computing load capacity is inversely proportional to the spacing of top-flange points of lateral support; which is provided by adequate-depth transverse diaphragms, deck clips or clamps measured along one side but staggered back and forth along both sides of the flange, power-driven fasteners, etc. In some instances, a bridge can have its posting removed or significantly reduced very simply and economically by reducing the spacing between points of lateral support on the top flange.

The load capacity is also inversely proportional to the width of the compression flange of the stringer or beam. It is important that the inspector determines the current area and configuration of top-flange as well as general corrosion and section loss. Top flanges of stringers supporting timber decks are naturally subject to section loss due to deck leakage. As a general concern, the inspector should determine whether the existing condition (decay, insect infestation, dry rot, etc.) of the timber will allow it to act as a sound structural element and whether it can support the fasteners previously described which provide lateral support to the compression flange of the steel stringers/beams.

Timber decks often have rock or earth fill ranging from light to very substantial. This naturally reduces the live load capacity of the bridge and inhibits determination of the condition of the deck. Before a rehabilitation scheme is developed for a given bridge, consideration should be given to adjusting the gradeline which could remove or reduce the deck fill, which in turn would influence the method of rehabilitation and/or strengthening.

The rehabilitation of concrete decks can often be accomplished using a high-density concrete or latex-modified concrete overlay. Such overlays usually prolong the life of the deck for a substantial period of time, but also reduce the live load capacity of the bridge. For bridges that are marginally "legal" this extra weight may require the bridge to be posted if provisions were not made for a "future wearing surface" in the original design. Whether a concrete deck is removed and replaced as part of the overall rehabilitation of the bridge will influence the approach taken to improve the load capacity of the stringers. For example, if the deck is removed from a steel stringer bridge which has no composite action between the deck and the stringers, the load capacity of the bridge can be easily and economically improved by designing and installing shear studs to the stringers for composite action before replacing the decks and/or respacing and adding stringers. The emphasis here is that the inspector should provide enough detailed data (surface spalls, map cracks, delamination, rebar exposures, etc.) regarding the deck condition to determine if it is cost effective to salvage the deck, repair it by patching, overlay it, or remove it to facilitate a more practical method of strengthening the stringers.

Full-depth deck cracks can often be detected by the presence of efflorescence on the bottom of the deck and by rust on the edges of the top stringer flanges. If plans are not available indicating the presence of shear connectors for composite actions, coring of the deck over the stringers should be considered to determine if shear connectors are present.

Another situation for overall strengthening is that in which it is determined that the non-composite concrete deck needs replacing and the stringers are slightly sub-legal. If the concrete deck is removed and replaced with a properly designed and installed timber deck, the reduction in dead load and the resulting increase in live load capacity could be substantial. Naturally, existing stringer spacing and the economy of respacing is important here. A concrete deck can provide lateral support for the compression flange of steel stringers if that flange is embedded in the bottom of the deck.

The use of metal decks is not too common, but their inspection should include the spacing and quality of fasteners (usually welds) which attach the deck to the stringers. Most metal decks are a commercially designed product. The manufacturer's specifications, unit weights and recommended installation methods should be obtained and reviewed. The inspector should also be concerned about the presence of cracks in the portland cement or asphaltic concrete topping reflecting from the corrugations in the metal pans. This is quite common and leads to moisture reaching and corroding the metal decks.

Metal decks -- as an element considered for strengthening and rehabilitation -- can offer a durable, relatively light-weight deck and may have, in some situations, advantages relative to reinforced concrete or timber. The closely spaced welds usually specified provide good lateral support of the steel stringer compression flange.

As a final emphasis, inspection and evaluation of decks should consider not only their condition and load carrying capacity as an isolated structural element, but also the effect the deck has, or could have, on the stringer system in terms of lateral support, composite action and reduction of dead load.

3.2.2. Stringers/Beams

Inspectors should recognize signs of distress in stringer systems. In timber bridges, the top fibers of the stringers are subject to high moisture content and rot caused by deck leakage. The inspector should probe this area and other suspicious areas, such as around bearings, with an ice pick to determine the reduction in useful section. It is a practice in some places to deal with top fiber rot by removing the deck and turning the stringers over. This provides better material for re-spiking the deck to the stringers, but doesn't improve the stringer capacity because it still has the same effective depth and now the decayed fibers are in tension. The area around knots, especially in the middle-third of the span near the bottom tension fibers, should be inspected closely for signs of horizontal splits or fiber failure.

Horizontal splits is a serious form of failure, although some residual capacity remains. For example, a 4 in. x 16 in. stringer with a horizontal split at mid-depth yields two 4 in. x 8 in. pieces. The resulting section modulus is then $2(4 \times 8^2/6) = 85.3$ in^3 which is half of that of the unsplit 4 in. x 16 in. which is $(4 \times 16^2/6) = 170.7$ in^3. The presence of short horizontal splits at the stringer ends effects shear capacity.

The inspector should be alert for excessive twist (out-of-plane bending), sweep (horizontal deflection) and vertical deflection (usually caused by under design or excessive fill on deck). Twist, sweep and the sudden lateral "rolling" of the stringer system (domino effect) can be effectively controlled by the use of spacer blocks or sturdy x-bracing at the bearings and at mid-span.

Insects, usually termites or carpenter ants, can be very destructive. Often a stringer will appear normal in size and surface texture, but will have its interior fibers eaten away. Tapping and probing of the surfaces and observation of the sawed cross-section at the stringer ends should be a routine part of any timber bridge inspection.

The rehabilitation and strengthening of timber stringer bridges (see Sec. 3.6) is routinely accomplished in several ways, such as by simply substituting sound members for individual damaged or undersized members, by dismantling and re-erecting with sound members of the same or larger size, by using a mix of sound and salvageable members either at the same stringer spacing or at a smaller spacing, determined by calculation, or by inserting sound members into the spacing between existing stringers, which arbitrarily cuts the spacing in half. In any case, the spacer blocks or X-bracing will have to be removed, modified and reinstalled. Naturally, calculations should be prepared to provide comparison data on cost, practicability, and benefits gained by eliminating or reducing the load posting requirement.

Again, the inspector should keep in mind that the data to be gathered are to be used in the rehabilitation and/or strengthening of the bridge in question, not merely to determine its present condition and load capacity.

Any observations or suggestions thought to be helpful should be recorded.

The inspection of steel beams or girders should take the same general approach as with timber. Instead of decaying timber, there is corroding metal. Instead of timber decks which don't interact with timber stringers to produce a greater load capacity, there may be concrete decks which may be composite with the beams or girders through the use of shear connectors.

Whereas decaying timber is irreversible, the life of a steel member can be extended by thorough cleaning and painting. The inspector should note, measure and photograph heavily corroded areas. The extend and location of corrosion will influence a decision on whether to incorporate the damaged member into the rehabilitation plan by determining, with calculations, whether the loss of section will reduce the load capacity.

The inspector will often find that a steel stringer bridge with a timber deck will have that deck affixed to the top (compression) stringer flange with clips located only along one edge of the flange. This practice offers no restraint to lateral movement in the direction opposite the clips. If the member was properly analyzed for this condition and it was determined to govern the load posting, strengthening can be achieved by removing the deck and resetting it with clips placed alternately along each side of the flange. If the required equipment is available, the same results can be achieved with power-driven fasteners, usually spaced at about 2 ft on centers.

The inspector should verify the size and location of bolt holes often found in webs and flanges of re-erected beams and stringers taken from stockpiles of dismantled bridges. The size and location of any bottom-flange cover plates should also be noted. Strengthening may be achieved by attaching cover plates if none exist or additional plates if practicable, to the existing beams.

As with timber, the inspector should note any twists, sweep or excessive vertical deflection. Fatigue cracks are not too common in simple-span bridges on low-volume roads, however, they may result from out-of-plane bending, base metal "notching" caused by careless welding, a poorly designed structural attachment, etc. Therefore, beam and girder webs should be carefully examined at the top and bottom of diaphragm or floor beam connections; flanges at the ends of cover plates should also be carefully examined. Fatigue cracking can usually be arrested if detected in the early stages.

Leaf rust, that is the lamination of metal in the corrosion process, is often found near the ends of beams at leaking expansion joints and at the interface of pieces such as angle legs and webs of built-up girders, diaphragm-to-stiffener connections, or under cover plates or gusset plates.

The location and severity of fatigue cracks and corrosion and the practicability of making repairs must, of course, be included in the rehabilitation decision process to minimize the possibility of "putting good money after bad".

3.2.3. Drains, Bearings and Expansion Joints

Leaking decks, floor drains that do not extend below the bottom of the bottom flange of the beams or girders and expansion joints that leak, such as the once common plate-on-plate device, are sources of concentrated corrosion-producing drainage. The inspector should note any observations in this regard so that necessary changes (such as extending or plugging floor drains, replacing leaking expansion joints with neoprene gland or closed-cell joints, etc.) can be included in the rehabilitation plan. Where a new concrete deck is planned, expansion joints and deteriorating abutment backwalls can be eliminated by pouring an abutment diaphragm integral with the slab and flush against the cap or footing bridge seat using doweled-in reinforcing. The inspector should note the condition or repairability of the cap or footing.

Bearing material in steel bridges is very vulnerable to corrosion caused by drainage through timber decks or the aforementioned leaking expansion devices. This drainage often ponds on the bridge seat and inundates the masonry plates. Over time, the corrosion will cause the bearing plates to "freeze-up". The immobility of joints and bearings is also often caused by movement of the abutments toward each other causing the backwalls to jam against the ends of the beams or girders. The bridge, expanding in the summer months, but having "frozen" bearings, could impart forces of damaging magnitude.

The inspector should note if the expansion device is closed, the severity of rocker tipping, the distance between the beam ends and front face of backwall, etc. These problems may require some rehabilitation of the substructures to allow the superstructure to be more functional and unrestricted by secondary loads and forces.

Thorough inspection is important! Plans for rehabilitation and strengthening should not proceed until the data gathered has been reviewed and a feasibility study, including preliminary pricing, bridge life, and load capacity benefits has been prepared. Good inspection and reporting may expose problems of such a magnitude that rehabilitation and/or strengthening are not cost effective.

3.3. Fundamentals

Pursuant to preparing a load capacity analysis for a simple-span steel beam or timber stringer bridge is the assembly of dimensional, material, loading and condition data.

3.3.1. Dimensions

If plans (preferably as-built) are not available, field measurements will be required to determine:

- Beam/stringer sizes
- Beam/stringer lateral spacing
- Center to center of bearings
- Type, size and spacing of diaphragms (steel) or blocking (timber)
- Width, thickness and type of deck
- Depth and type of any wearing surface present (concrete or asphalt overlay, rock or dirt fill)

3.3.2. Materials (Allowable Stresses)

3.3.2.1. Steel

Plans prepared within approximately the past twenty-five years will show material strengths (allowable stresses). If this information is not available on the plans, but the year of construction is known, the yield strength (F_y) for steel components can be assumed as follows:

- Prior to 1905 - 26,000 psi
- 1905 to 1936 - 30,000 psi
- 1936 to 1963 - 33,000 psi
- 1963 to Present - 36,000 psi

If neither material strength or year of construction is available, the engineer is left to his/her best judgment; one shouldn't, however, assume a yield strength greater than 30,000 psi. The AASHTO manuals (2) includes a table of allowable stresses for steel in Sec. 10; allowable steel stresses may also be found in Sec. 5 of Ref. 3.

The allowable stress to be used in analysis is 0.55 F_y for the "Inventory" capacity and 0.75 F_y for the "Operating" capacity, except as modified when the compression flange is not fully restrained against lateral movement. In these cases, the following is used:

Allowable inventory stress, $f_b = 0.55\ Fy - K(L/b)$

where:

F_y = appropriate yield strength as determine above

K = 3.9 for F_y = 26,000 psi
5.2 for F_y = 30,000 psi
6.3 for F_y = 33,000 psi
7.5 for F_y = 36,000 psi

L = spacing (in inches) of feature that restrains the compression flange from lateral deflection in both directions, (e.g. diaphragms, power driven fasteners, clips, etc.)

b = width (in inches) of compression flange

L/b shall not be greater than 42, 39, 38 and 36 for the respective F_y values listed in the definition of "K". If these limits are exceeded, "L" should be reduced by some physical alteration. The allowable operating stress for a partially restrained compression flange is the inventory stress f_b as calculated above, times a factor of 1.37.

3.3.2.2. Timber

The determination of allowable stress in timber stringers and decks begins with knowing the species and grade of lumber used in the member to be analyzed. The appropriate allowable value (shear, bending, tension, compression, etc.) can then be obtained from tables in Section 13 of the "Standard Specifications for Highway Bridges" (2) based on member size and usage with subjective reductions based on the engineer's judgment, considering whether the timber is treated or untreated, its moisture content, age, condition, etc. This stress may be considered as the allowable "Inventory" stress. The allowable "Operating" stress should not exceed 1.33 times the allowable "Inventory" stress.

3.3.3. Loads

- Dead Load

  Decks -

Timber	=	50 lb/cu.ft
Concrete	=	150 lb/cu.ft
Steel	=	490 lb/cu.ft

  Wearing Surface -

Concrete	=	150 lb/cu.ft
Asphalt	=	144 lb/cu.ft
Rock	=	120 lb/cu.ft
Dirt	=	125 lb/cu.ft

  Rails -

  As determined by measurement and the above unit loads or from tables.

  Curbs -

  Based on measurements and type of material.

  Beams/Stringers -

  If plans or shop drawings are not available, steel beam weights can be read from tables in the *American Institute of Steel Construction* Manual (AISC) (6) once the beam dimensions are carefully determined. Timber stringer weights are based on field measurements and 50 lb/cuft.

  Diaphragms or Blocking -

  Load per foot of beam/stringer is the weight of one diaphragm or blocking unit divided by the longitudinal spacing of the diaphragms or blocking.

  Other Attachments -

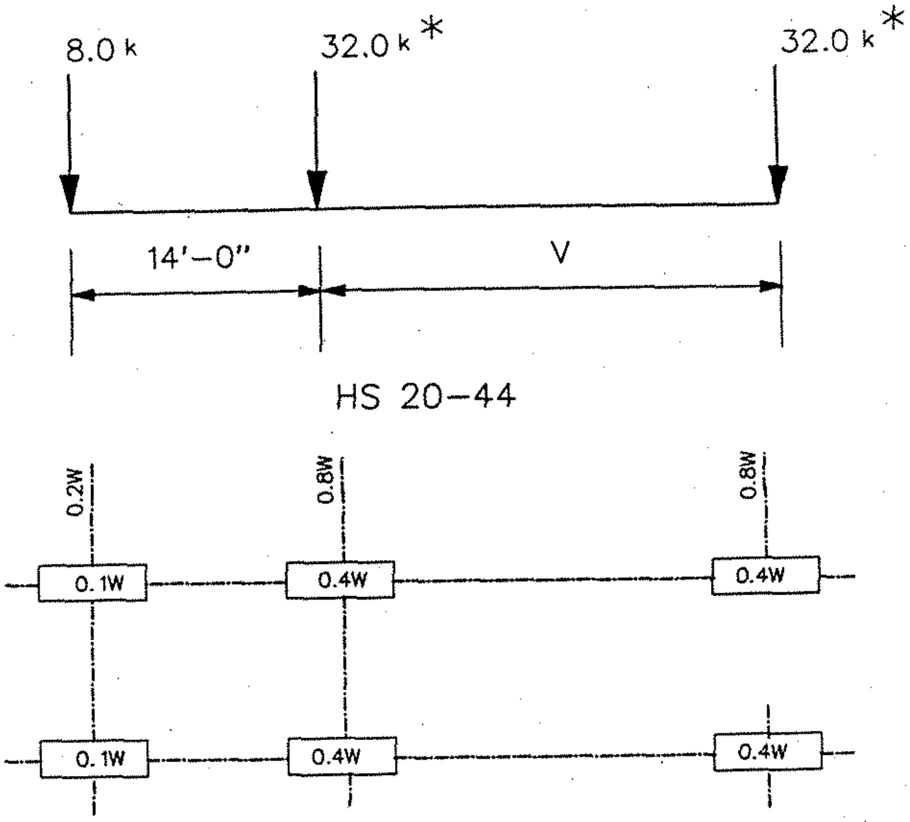
  Pipes, conduits, cattle restraints, etc (as determined by the engineer)

- Live Load

The FHWA requires that the "Operating Rating" and "Inventory Rating" be based on "HS" (AASHTO Loading) as graphically defined in Fig. 3.6. Any analysis involved in rehabilitation and strengthening to determine posting requirements for Iowa legal trucks should include the "HS" capacities as well.

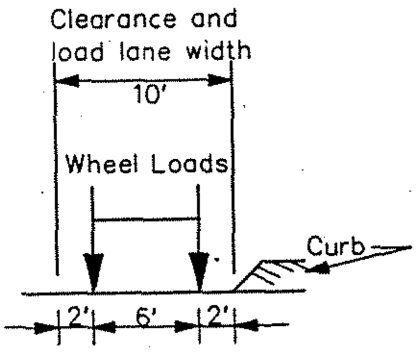
Figure 3.7 shows the axle loads and spacing for the Iowa legal trucks. Bending moments in ft-kips per wheel line for these vehicles are presented in Appendix B. A wheel line is defined as the wheels on one side of the truck.

The bending moments determined from the tables in Appendix B must be distributed laterally to establish a per beam/stringer bending moment. This distribution is determined from Article 3.23 AASHTO (2). "One Traffic Lane" distribution is to be used when the roadway width is 18 ft or less and "Two Traffic



W = Combined weight on the first two axles which is the same as for the corresponding H (M) truck.

V = Variable spacing – 14 feet to 30 feet *inclusive*. Spacing to be used is that which produces maximum stresses.

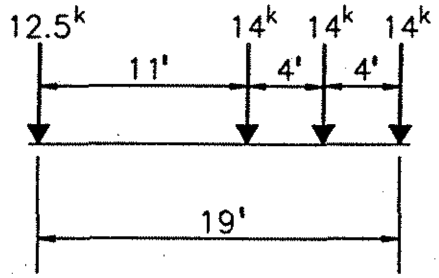


Standard HS Trucks

* In the design of timber floors and orthotropic steel decks (exluding transverse beams) for HS 20 loading: one axle load of 24,000 pounds or two axle loads of 16,000 pounds each, spaced 4 feet apart may be used, whichever produces the greater stress, instead of the 32,000–pound axle shown.

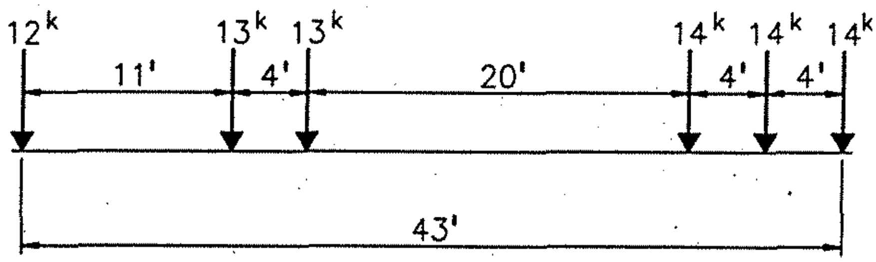
Fig. 3.6. AASHTO standard loads.

RATING VEHICLES (Showing axle loads in kips.)



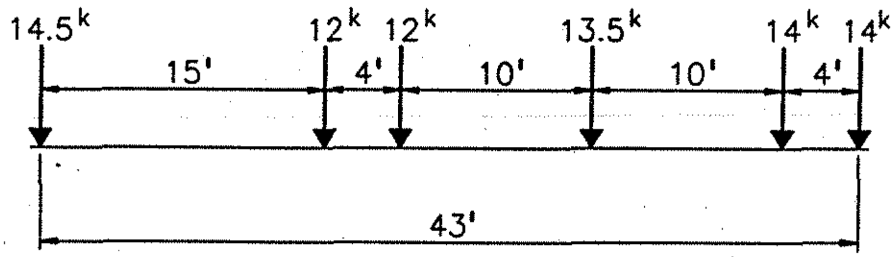
TYPE 4

weight = 54.5 kips



TYPE 3S3

weight = 80 kips



TYPE 3-3

weight = 80 kips

Fig. 3.7. Iowa legal loads.

Lanes" distribution is to be used for bridges with roadways greater than 18 ft. County bridges with roadway widths requiring analysis for more than two lanes (36 ft) are rare and will not be considered here.

Live load distribution for timber decks is addressed in Articles 3.25 and 3.30 of AASHTO (2) and will be demonstrated in example calculations which follow.

- Impact

Impact is included in the total load determination as a fraction of the calculated live load. As noted in Article 3.8.1 of AASHTO (2), impact is not to be considered for timber structures.

The impact fraction is determined by the formula:

$$I = \frac{50}{L+125} \leq 0.30$$

where: I = impact fraction (maximum 0.30)

L = span length, center to center of bearings for simple span bridges in ft.

3.3.4. Condition

Dimensions of stress-carrying members can change over time due to corrosion, decay, wear (decks), damage from external forces (ice and drift flows), etc. This naturally causes a reduction in member cross-section resulting in a smaller section modulus, S. The bending stress, f_b, in a member is equal to the bending moment M divided by the section modulus, thus:

$$f_b = \frac{M}{S}$$

Therefore, the bending stress, f_b, is inversely proportional to the section modulus S. It is not uncommon to find bolt holes in the webs and flanges of steel beams, usually when a member has been re-erected from stockpiled material. The section modulus should be calculated to reflect the presence of these holes. It is important during the gathering of dimensional data that the location of any localized reduction in section be identified along the beam span relative to the centerline of bearing so that the reduced bending moment at that location can be computed.

A uniform loss of section due to light overall rusting of a steel member can be estimated by the inspector. The section modulus may be reduced 1 to 5 percent. When heavy rust and pitting are present, the beam should be spot-cleaned with a wire brush and measurements taken with a caliper and metal tape.

Loss of section in timber stringers is also based on judgement and measurements taken by probing a member with an ice pick or a similar instrument. A reduction in allowable stress as well as in section should be considered by the engineer. The expression for the section modulus of a timber stringer is:

$$S = \frac{bd}{6}$$

where,

b = effective width of member.

d = effective depth of member.

County simple span steel beam bridges, with very rare exceptions in Iowa, can be placed in Case III of Table 10.3.2A of AASHTO (2). In other words, the bridge being considered will not likely be located on "Freeways, Expressways, Major Highways, and Streets" having high average daily truck traffic (ADT).. Therefore, the reduction in allowable stresses due to fatigue caused by large stress ranges and high cyclic loading will not govern and will not be discussed here, other than to say that poor quality and poorly designed welds indiscriminately located can be a source of member failure. Therefore, all welding employed in performing repairs and strengthening procedures should be approved by the engineer and performed by a certified welder in strict adherence to current and applicable codes of the American Welding Society including such considerations as weldability of base metal, selection of proper welding rod and power setting, welder's position (i.e. overhead, vertical, down-hand, horizontal), weather conditions, etc.

3.3.5. Rating

The following ten numerical examples assemble the above data and information into operating and inventory ratings and load posting by means of the following steps:

1. Calculate allowable bending stress (F_a) in beam or stringer based on information described in Materials (Allowable Stresses) (Sec. 3.3.2).
2. Calculate actual dead load bending stress (f_d) in beam or stringer using a uniform load per foot of span (w) as described in "Loads" (Sec. 3.3.3) and applying the following formulas:

   M (Dead Load Moment) = w x $L^2/8$

   where,

   L = span

   f_d = M/S

   where, S is the section modulus as described in Condition (Sec. 3.3.4).

3. Record the gross weight of the applied truck in tons for which rating or posting is being calculated:

HS20	=	36 Tons
Type 3	=	25 Tons (Governs for Deck Analysis)
Type 4	=	27.25 Tons

Type 3S3 = 40 Tons

Type 3-3 = 40 Tons

4. Calculate actual live load bending stress in beam or stringer (including impact except for timber members) using moments from tables in Appendix B which are based on truck dimensions and axle loads shown in Figs. 3.6 and 3.7. These tabular moments, given in ft-kips, should be converted to in.-lbs by multiplying by 12,000 (12 in./ft x 1,000 lb/kip). This moment should then be multiplied by a factor for lateral distribution. The resulting stress when this moment is divided by the section modulus will be designated as (f_l).

The rating of the member can now be calculated by the expression:

$$Rating\ (Tons) = \frac{(F_a - f_d)\ T}{f_l}$$

where,

T = rating vehicle in tons

F_a = allowable bending stress

f_d = dead load bending stress

f_l = live load bending stress

The following examples are included to illustrate the above method. For clarity, larger type size has been used for the examples.

3.3.6. Examples

3.3.6.1. Example 1: Timber stringers (1 ft-6 in. on center); Timber deck with rock fill

GIVEN:

4 in. x 16 in. Timber Stringers @ 1 ft-6 in. c to c
Span = 24 ft
One Lane Bridge (W $\leq$ 18 ft)
3 in. x 12 in. Transverse Plank Deck
Full Lateral Support
4 in. Rock Fill (120 lb/cf)
$F_{b\,INV}$ = 1600 psi
$F_{b\,OPER}$ = 1600 psi(1.33) = 2128 psi

DEAD LOADS:

Deck: $$\left(\frac{3\text{ in}}{12\text{ in/ft}}\right)\left(50\ \frac{\text{lb}}{\text{ft}^3}\right)(1.50\text{ ft}) = 18.75\text{ lb/ft}$$

Stringer: (nominal dimensions) $$\left[\frac{(4\ 1/8\text{in})(16\ 1/2\text{in})}{144\text{ in}^2/\text{ft}^2}\right]\left(50\ \frac{\text{lb}}{\text{ft}^3}\right) = 20.86\text{ lb/ft}$$

Rock: $$\left(\frac{4\text{ in}}{12\text{ in/ft}}\right)\left(120\ \frac{\text{lb}}{\text{ft}^3}\right)(1.50\text{ ft}) = 60.00\text{ lb/ft}$$

Misc: $$= 2.39\text{ lb/ft}$$

$$w_{DL} = 102.00\text{ lb/ft}$$

$$M_{DL} = \frac{wl^2}{8} = \frac{102\text{ lb/ft }(24\text{ ft})^2}{8} = 7344\text{ ft-lb}$$

Section Modulus, $S_b = 1/6(4\ 1/8\text{in})(16\ 1/2\text{in})^2$
$S_b = 155.16\text{ in}^3$

$$f_{bDL} = \frac{M_{DL}}{S_b} = \frac{7344\text{ ft-lb }(12\text{ in/ft})}{155.16\text{ in}^3} = 567.8\text{ psi}$$

LIVE LOADS:

Impact: I = 0 (For Timber Structures)

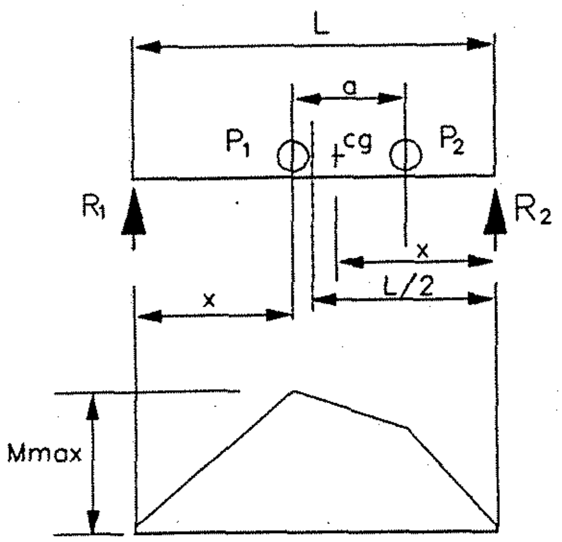
Wheel Factor: (AASHTO TABLE 3.23.1; Timber Plank Floor; Timber Longitudinal Beams; Bridge Designed For One Traffic Lane)

$$W.F. = \frac{S}{4.0} \qquad S = \text{Stringer Spacing}$$

$$W.F. = \frac{1.5}{4.0} = 0.375$$

Live Loads, continued:

M_{LL} = (Moment Due To Live Load Truck) x (1+Impact) x (Wheel Factor)



Maximum moment due to truck loads on a simple span can be determined by use of a general rule. The maximum bending moment occurs under one of the wheel loads when that wheel load is as far from one support as the center of gravity of all the wheel loads is from the other support. The relationship is shown to the left.

The relationship was used to determine the maximum live load moments for different truck types on different lengths of spans (see Appendix B.)

$M_{LL\ (HS20)}$ = (96.00 ft-kips)(1.0)(0.375)(1000 lb/k) = 36,000 ft-lb

$M_{LL\ (TYPE\ 4)}$ = (98.00 ft-kips)(1.0)(0.375)(1000 lb/k) = 36,750 ft-lb

$M_{LL\ (TYPE\ 3S3)}$ = (98.00 ft-kips)(1.0)(0.375)(1000 lb/k) = 36,750 ft-lb

$M_{LL\ (TYPE\ 3\text{-}3)}$ = (77.53 ft-kips)(1.0)(0.375)(1000 lb/k) = 29,074 ft-lb

$$f_{bLL} = \frac{M_{LL}}{S_b}$$

$$f_{bLL\ (HS20)} = \frac{36{,}000 \text{ ft-lb } (12 \text{ in/ft})}{155.16 \text{ in}^3} = 2784.22 \text{ psi}$$

$$f_{bLL\ (TYPE\ 4)} = \frac{36{,}750 \text{ ft-lb } (12 \text{ in/ft})}{155.16 \text{ in}^3} = 2842.23 \text{ psi}$$

$$f_{bLL\ (TYPE\ 3S3)} = \frac{36{,}750 \text{ ft-lb } (12 \text{ in/ft})}{155.16 \text{ in}^3} = 2842.23 \text{ psi}$$

$$f_{bLL\ (TYPE\ 3\text{-}3)} = \frac{29{,}074 \text{ ft-lb } (12 \text{ in/ft})}{155.16 \text{ in}^3} = 2248.57 \text{ psi}$$

HS20 RATING:

$$\text{HS Rating} = \left[\frac{\begin{pmatrix}\text{Allowable}\\ \text{Stress}\end{pmatrix} - \begin{pmatrix}\text{Dead Load}\\ \text{Stress}\end{pmatrix}}{\text{Live Load Stress}}\right] (36T)$$

$$\begin{matrix}\text{Inventory}\\ \text{Rating}\end{matrix} = \left(\frac{1600 \text{ psi} - 567.98 \text{ psi}}{2748.22 \text{ psi}}\right)(36T) = 13.3 \text{ tons}$$

$$\begin{matrix}\text{Operating}\\ \text{Rating}\end{matrix} = \left(\frac{2128 \text{ psi} - 567.98 \text{ psi}}{2784.22 \text{ psi}}\right)(36T) = 20.2 \text{ tons}$$

LOAD POSTINGS:

$$\begin{matrix}\text{Rating/}\\ \text{Posting}\end{matrix} = \left[\frac{\begin{pmatrix}\text{Operating}\\ \text{Stress}\end{pmatrix} - \begin{pmatrix}\text{Dead Load}\\ \text{Stress}\end{pmatrix}}{\text{Live Load Stess}}\right] (W)$$

W = weight of truck causing the live load.

$$\text{Type 4} = \left(\frac{2128 \text{ psi} - 567.98 \text{ psi}}{2842.23 \text{ psi}}\right)(27.25T)$$

Type 4 = 14.96 T - Post for 15 Tons

$$\text{Type 3S3} = \left(\frac{2128 \text{ psi} - 567.98 \text{ psi}}{2842.23 \text{ psi}}\right)(40\ T)$$

Type 3S3 = 21.96 T - Post for 22 Tons

$$\text{Type 3-3} = \left(\frac{2128 \text{ psi} - 567.98 \text{ psi}}{2248.57 \text{ psi}}\right)(40\ T)$$

Type 3-3 = 27.75 T - Post for 28 Tons

3.3.6.2. Example 2: Timber Stringer (1 ft-6 in. on center); Timber Deck

GIVEN:

Same as Example 1, without rock fill.

DEAD LOADS:

Deck: $\left(\frac{3\ \text{in}}{12\ \text{in/ft}}\right)\left(50\ \frac{\text{lb}}{\text{ft}^3}\right)(1.50\ \text{ft}) = 18.75\ \text{lb/ft}$

Stringer: (nominal dimensions) $\left[\frac{(4\ 1/8\text{in})(16\ 1/2\text{in})}{144\ \text{in}^2/\text{ft}^2}\right]\left(50\ \frac{\text{lb}}{\text{ft}^3}\right) = 20.86\ \text{lb/ft}$

Misc: $= \underline{\ \ 2.39\ \text{lb/ft}}$

$w_{DL} = 42.00\ \text{lb/ft}$

$$M_{DL} = \frac{wl^2}{8} = \frac{42\ \text{lb/ft}\ (24\ \text{ft})^2}{8} = 3024\ \text{ft-lb}$$

$$f_{bDL} = \frac{M_{DL}}{S_b} = \frac{3024\ \text{ft-lb}\ (12\ \text{in/ft})}{155.16\ \text{in}^3} = 233.88\ \text{psi}$$

LIVE LOADS:

Live Load Moments (M_{LL}) and Live Load Stresses (f_{bLL}) are the same as those calculated for Example 1.

HS20 RATING:

$$\text{HS Rating} = \left[\frac{\left(\begin{array}{c}\text{Allowable}\\ \text{Stress}\end{array}\right) - \left(\begin{array}{c}\text{Dead Load}\\ \text{Stress}\end{array}\right)}{\text{Live Load Stress}}\right](36\text{T})$$

$$\text{Inventory Rating} = \left(\frac{1600\ \text{psi} - 233.88\ \text{psi}}{2784.22\ \text{psi}}\right)(36\text{T}) = 17.7\ \text{tons}$$

$$\text{Operating Rating} = \left(\frac{2128\ \text{psi} - 233.88\ \text{psi}}{2784.22\ \text{psi}}\right)(36\text{T}) = 24.5\ \text{tons}$$

LOAD POSTINGS:

$$\text{Rating/ Posting} = \left[\frac{\begin{pmatrix}\text{Operating}\\ \text{Stress}\end{pmatrix} - \begin{pmatrix}\text{Dead Load}\\ \text{Stress}\end{pmatrix}}{\text{Live Load Stess}}\right] (W)$$

W = weight of truck causing the live load.

$$\text{Type 4} = \left(\frac{2128 \text{ psi} - 233.88 \text{ psi}}{2842.23 \text{ psi}}\right) (27.25T)$$

Type 4 = 18.16 T - Post for 18 Tons

$$\text{Type 3S3} = \left(\frac{2128 \text{ psi} - 233.88 \text{ psi}}{2842.23 \text{ psi}}\right) (40\ T)$$

Type 3S3 = 26.66 T - Post for 27 Tons

$$\text{Type 3-3} = \left(\frac{2128 \text{ psi} - 233.88 \text{ psi}}{2248.57 \text{ psi}}\right) (40\ T)$$

Type 3-3 = 33.70 T - Post for 34 Tons

3.3.6.3. Example 3: Timber Stringers (1 ft-2 in. on center); Timber deck with rock fill

GIVEN:

Same as Example 1 with 1 ft-2 in. stringer spacing.

DEAD LOADS:

Deck: $\left(\frac{3 \text{ in}}{12 \text{ in/ft}}\right)\left(50 \frac{\text{lb}}{\text{ft}^3}\right)(1.167 \text{ ft}) = 14.58 \text{ lb/ft}$

Stringer: (nominal dimensions) $\left[\frac{(3\ 7/8 \text{ in})(15\ 1/2 \text{ in})}{144 \text{ in}^2/\text{ft}^2}\right]\left(50 \frac{\text{lb}}{\text{ft}^3}\right) = 20.86 \text{ lb/ft}$

Rock: $\left(\frac{4 \text{ in}}{12 \text{ in/ft}}\right)\left(120 \frac{\text{lb}}{\text{ft}^3}\right)(1.167 \text{ ft}) = 46.67 \text{ lb/ft}$

Misc: $= 2.89 \text{ lb/ft}$

$w_{DL} = 85.00 \text{ lb/ft}$

$$M_{DL} = \frac{wl^2}{8} = \frac{85 \text{ lb/ft } (24 \text{ ft})^2}{8} = 6120 \text{ ft-lb}$$

$$f_{bDL} = \frac{M_{DL}}{S_b} = \frac{6120 \text{ ft-lb } (12 \text{ in/ft})}{155.16 \text{ in}^3} = 473.32 \text{ psi}$$

LIVE LOADS:

Impact: I = 0 (For Timber Structures)

Wheel Factor: (AASHTO TABLE 3.23.1; Timber Plank Floor; Timber Longitudinal Beams; Bridge Designed For One Traffic Lane)

$$W.F. = \frac{S}{4.0} \qquad S = \text{Stringer Spacing}$$

$$W.F. = \frac{1.16}{4.0} = 0.292$$

M_{LL} = (Moment Due To Live Load Truck) x (1+Impact) x (Wheel Factor)

$M_{LL\ (HS20)}$ = (96.00 ft-kips)(1.0)(0.292)(1000 lb/k) = 28,032 ft-lb

$M_{LL\ (TYPE\ 4)}$ = (98.00 ft-kips)(1.0)(0.292)(1000 lb/k) = 28,616 ft-lb

$M_{LL\ (TYPE\ 3S3)}$ = (98.00 ft-kips)(1.0)(0.292)(1000 lb/k) = 28,616 ft-lb

$M_{LL\ (TYPE\ 3\text{-}3)}$ = (77.53 ft-kips)(1.0)(0.292)(1000 lb/k) = 22,639 ft-lb

$$f_{bLL} = \frac{M_{LL}}{S_b}$$

$$f_{bLL\ (HS20)} = \frac{28{,}032\ \text{ft-lb}\ (12\ \text{in/ft})}{155.16\ \text{in}^3} = 2167.98\ \text{psi}$$

$$f_{bLL\ (TYPE\ 4)} = \frac{28{,}616\ \text{ft-lb}\ (12\ \text{in/ft})}{155.16\ \text{in}^3} = 2213.15\ \text{psi}$$

$$f_{bLL\ (TYPE\ 3S3)} = \frac{28{,}616\ \text{ft-lb}\ (12\ \text{in/ft})}{155.16\ \text{in}^3} = 2213.15\ \text{psi}$$

$$f_{bLL\ (TYPE\ 3\text{-}3)} = \frac{22{,}639\ \text{ft-lb}\ (12\ \text{in/ft})}{155.16\ \text{in}^3} = 1750.89\ \text{psi}$$

<u>HS20 RATING:</u>

$$\text{HS Rating} = \left[\frac{\left(\text{Allowable Stress}\right) - \left(\text{Dead Load Stress}\right)}{\text{Live Load Stress}}\right] (36T)$$

$$\text{Inventory Rating} = \left(\frac{1600\ \text{psi} - 473.32\ \text{psi}}{2167.98\ \text{psi}}\right)(36T) = 18.7\ \text{tons}$$

$$\text{Operating Rating} = \left(\frac{2128\ \text{psi} - 473.32\ \text{psi}}{2167.98\ \text{psi}}\right)(36T) = 27.5\ \text{tons}$$

LOAD POSTINGS:

$$\text{Rating/ Posting} = \left[\frac{\begin{pmatrix}\text{Operating}\\ \text{Stress}\end{pmatrix} - \begin{pmatrix}\text{Dead Load}\\ \text{Stress}\end{pmatrix}}{\text{Live Load Stess}}\right](W)$$

W = weight of truck causing the live load.

$$\text{Type 4} = \left(\frac{2128\text{ psi} - 473.32\text{ psi}}{2213.15\text{ psi}}\right)(27.25T)$$

Type 4 = 20.37 T - Post for 20 Tons

$$\text{Type 3S3} = \left(\frac{2128\text{ psi} - 473.32\text{ psi}}{2213.15\text{ psi}}\right)(40\ T)$$

Type 3S3 = 29.91 T - Post for 30 Tons

$$\text{Type 3-3} = \left(\frac{2128\text{ psi} - 473.32\text{ psi}}{1750.89\text{ psi}}\right)(40\ T)$$

Type 3-3 = 37.80 T - Post for 38 Tons

3.3.6.4. Example 4: Steel stringers with full lateral support of compression flange

GIVEN:

18 x 54.7 American Standard I-Beam @ 3 ft - 4 in. spacing c to c
Span = 40 ft
Two Lane Bridge (W≥18 ft)
4 in. x 12 in. Transverse Plank Deck
Full Lateral Support of Compression Flange
2 in. Rock Fill (120 lb/cf)
F_y = 33,000 psi (1936 to 1963)

DEAD LOADS:

Deck: $\left(\frac{4 \text{ in}}{12 \text{ in/ft}}\right)\left(50 \frac{\text{lb}}{\text{ft}^3}\right)(3.33 \text{ ft}) = 55.56 \text{ lb/ft}$

Stringer: $= 54.70 \text{ lb/ft}$

Rock: $\left(\frac{2 \text{ in}}{12 \text{ in/ft}}\right)\left(120 \frac{\text{lb}}{\text{ft}^3}\right)(3.33 \text{ ft}) = 66.67 \text{ lb/ft}$

Misc: $= 5.07 \text{ lb/ft}$

$w_{DL} = 182.00 \text{ lb/ft}$

$$M_{DL} = \frac{wl^2}{8} = \frac{182 \text{ lb/ft } (40 \text{ ft})^2}{8} = 36{,}400 \text{ ft-lb}$$

$$S_b = 89.4 \text{ in}^3 \quad \text{(AISC Design Manual)}$$

$$f_{bDL} = \frac{M_{DL}}{S_b} = \frac{36400 \text{ ft-lb } (12 \text{ in/ft})}{89.4 \text{ in}^3} = 4885.91 \text{ psi}$$

LIVE LOADS:

Impact: $I = \frac{50}{L + 125} \leq .30$ (AASHTO 3.8.3.1)

$$I = \frac{50}{40 + 125} = 0.303 > 0.3$$

$$I = 0.3$$

Wheel Factor: (AASHTO TABLE 3.23.1; Timber Plank Floor; Bridge Designed For Two Traffic Lanes)

$$W.F. = \frac{S}{3.75} \qquad S = \text{Stringer Spacing}$$

$$W.F. = \frac{3\text{ ft }-4\text{ in}}{3.75} = 0.889$$

M_{LL} = (Moment Due To Live Load Truck) x (1+Impact) x (Wheel Factor)

$M_{LL\ (HS20)}$ = (224.80 ft-kips)(1.3)(0.889)(1000 lb/k) = 259,801 ft-lb

$M_{LL\ (TYPE\ 4)}$ = (199.59 ft-kips)(1.3)(0.889)(1000 lb/k) = 230,666 ft-lb

$M_{LL\ (TYPE\ 3S3)}$ = (182.00 ft-kips)(1.3)(0.889)(1000 lb/k) = 210,337 ft-lb

$M_{LL\ (TYPE\ 3\text{-}3)}$ = (171.50 ft-kips)(1.3)(0.889)(1000 lb/k) = 198,203 ft-lb

$$f_{bLL} = \frac{M_{LL}}{S_b}$$

$$f_{bLL\ (HS20)} = \frac{259{,}801\text{ ft-lb (12 in/ft)}}{89.4\text{ in}^3} = 34{,}873\text{ psi}$$

$$f_{bLL\ (TYPE\ 4)} = \frac{230{,}666\text{ ft-lb (12 in/ft)}}{89.4\text{ in}^3} = 30{,}962\text{ psi}$$

$$f_{bLL\ (TYPE\ 3S3)} = \frac{210{,}337\text{ ft-lb (12 in/ft)}}{89.4\text{ in}^3} = 28{,}233\text{ psi}$$

$$f_{bLL\ (TYPE\ 3\text{-}3)} = \frac{198{,}203\text{ ft-lb (12 in/ft)}}{89.4\text{ in}^3} = 26{,}604\text{ psi}$$

HS20 RATING:

$$\text{Inventory Rating} = \left(\frac{18{,}000\text{ psi} - 4885.91\text{ psi}}{34{,}873\text{ psi}}\right)(36T) = 13.5\text{ tons}$$

$$\text{Operating Rating} = \left(\frac{24{,}500\text{ psi} - 4885.91\text{ psi}}{34{,}873\text{ psi}}\right)(36T) = 20.2\text{ tons}$$

LOAD POSTINGS:

$$\text{Type 4} = \left(\frac{24{,}500\text{ psi} - 4885.91\text{ psi}}{30{,}962\text{ psi}}\right)(27.25T)$$

Type 4 = 17.3 T - Post for 17 Tons

$$\text{Type 3S3} = \left(\frac{24{,}500\text{ psi} - 4885.91\text{ psi}}{28{,}233\text{ psi}}\right)(40\ T)$$

Type 3S3 = 27.8 T - Post for 28 Tons

$$\text{Type 3-3} = \left(\frac{24{,}500\text{ psi} - 4885.91\text{ psi}}{26{,}604\text{ psi}}\right)(40\ T)$$

Type 3-3 = 29.5 T - Post for 30 Tons

3.3.6.5. Example 5: Steel stringer with lateral support of compression flange at 10 ft intervals.

GIVEN:

Same as Example 4, except for lateral supports of compression flange at 10 ft intervals instead of full support.

DEAD LOADS:

Same as Example 4.

f_{bDL} = 4885.91 psi

LIVE LOADS:

Same as Example 4.

$f_{bLL\ (HS20)}$ = 34,873 psi

$f_{bLL\ (TYPE\ 4)}$ = 30,962 psi

$f_{bLL\ (TYPE\ 3S3)}$ = 28,233 psi

$f_{bLL\ (TYPE\ 3\text{-}3)}$ = 26,604 psi

ALLOWABLE STRESS:

$F_{INV} = 18000\text{ psi} - 6.3(l/b)^2$ $[(l/b) \leq 38]$ [Table 5.4.2A AASHTO 1983, Manual for Maintenance Inspection of Bridges]

l = 10 ft x 12 in/ft = 120 in.
b = Flange Width = 6 in.

$\frac{l}{b} = \frac{120\text{ in}}{6\text{ in}} = 20 < 38$ OK

$F_{INV} = 18{,}000\text{ psi} - 6.3(20)^2$

$F_{INV} = \underline{15{,}480\text{ psi}}$

$F_{OPER} = 1.37F_{INV} = 1.37(15{,}480\text{ psi})$

$F_{OPER} = 21{,}207.6\text{ psi}$

HS20 RATING:

$$\text{Inventory Rating} = \left(\frac{15{,}480\text{ psi} - 4885.91\text{ psi}}{34{,}873\text{ psi}}\right)(36T) = 10.9\text{ tons}$$

$$\text{Operating Rating} = \left(\frac{21{,}207.6\text{ psi} - 4885.91\text{ psi}}{34{,}873\text{ psi}}\right)(36T) = 16.8\text{ tons}$$

LOAD POSTINGS:

$$\text{Type 4} = \left(\frac{21{,}207.6\text{ psi} - 4885.91\text{ psi}}{30{,}962\text{ psi}}\right)(27.25T)$$

Type 4 = 14.4 T - Post for 14 Tons

$$\text{Type 3S3} = \left(\frac{21{,}207.6\text{ psi} - 4885.91\text{ psi}}{28{,}233\text{ psi}}\right)(40\text{ T})$$

Type 3S3 = 23.1 T - Post for 23 Tons

$$\text{Type 3-3} = \left(\frac{21{,}207.6\text{ psi} - 4885.91\text{ psi}}{26{,}604\text{ psi}}\right)(40\text{ T})$$

Type 3-3 = 24.5 T - Post for 25 Tons

3.3.6.6. Example 6: Steel stringers with full lateral support of compression flange and holes in bottom flange

<u>GIVEN:</u>

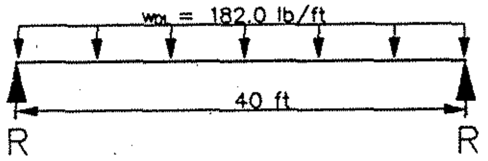
Same as Example 4, with 2 1/2 in. diameter holes in the bottom flange at the third points of the span.

<u>STEPS:</u>

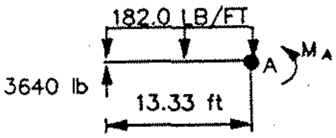
1. Calculate the ratings at the third point, using the section modulus of the cross-section with holes.

2. Compare the ratings calculated at the third point with the ratings calculated in Example 4, to determine which location/cross-section combination will control the rating.

<u>DEAD LOAD @ THIRD POINT:</u>

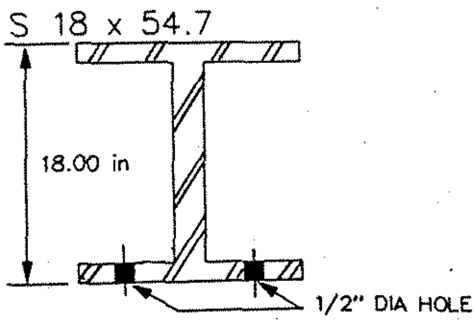


$R = 182$ lb/ft(40 ft)(1/2)
$R = 3640$ lb



$\sum M_A = 0$
$M_A = 3640$ lb(13.33 ft) $- 182$ lb/ft$(13.33 \text{ ft})^2(1/2)$
$M_A = 32{,}351.51$ ft-lb

<u>SECTION MODULUS OF MEMBER AT THIRD POINT:</u>



$A = 16.1$ in^2
$d = 18.00$ in
$b_f = 6.00$ in
$t_f = 0.691$ in
$I_x = 804$ in^4

$$I_{holes} = 2[1/12(1/2\ in)(0.691\ in)^3] = 0.027\ in^4$$
$$A_{holes} = 2(1/2\ in)(0.691\ in) = 0.691\ in^2$$

	A	y	Ay	Ay^2	
I-BEAM	16.100 in^2	9.000 in	144.900 in^3	1304.100 in^4	
				804.000 in^4	I_o
HOLES	-0.691 in^2	0.346 in	-0.239 in^3	-0.083 in^4	
	15.409 in^2		144.661 in^3	-0.027 in^4	I_o
				2107.990 in^4	
				-1358.065 in^4	
				749.925 in^4	I_b

$$y_t = \frac{144.661\ in^3}{15.409\ in^2} = 9.388\ in$$

$$Ay_t^{\,2} = (15.409\ in^2)(9.388\ in)^2 = 1358.065\ in^4$$

$$S_b = \frac{I_b}{y_t} = \frac{749.925\ in^4}{9.388\ in} = 79.881\ in^3$$

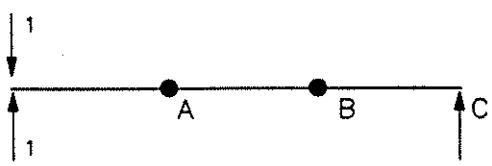
DEAD LOAD STRESS AT THIRD POINT:

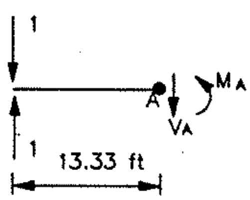
$$f_{bDL} = \frac{M_{DL}}{S_b} = \frac{32351.51\ ft\text{-}lb\ (12\ in/ft)}{79.881\ in^3}$$

$$f_{bDL} = 4859.96\ psi$$

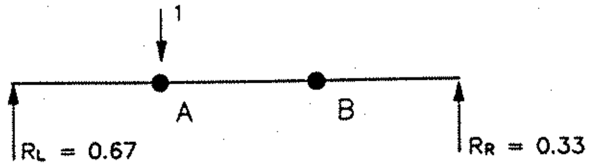
LIVE LOAD @ THIRD POINT

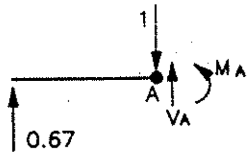
Maximum moment due to a moving live load at a specific point is most easily calculated by use of an influence line. An influence line is developed below to represent the moment produced at the thrid point as a 1 unit force is moved across the beam.





$$\sum M_A = 0$$
$$M_A = 1(13.33\ ft) - 1(13.33\ ft)$$
$$M_A = 0$$

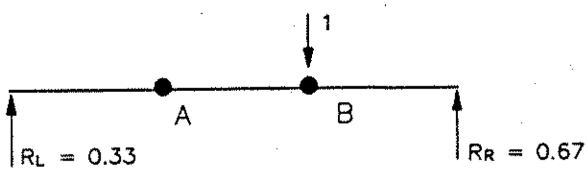


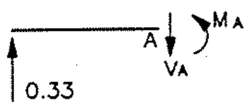


$\sum M_L = 0$
$0 = 1lb(13.33\ ft) - R_R(40\ ft)$
$R_R = 0.33$

$F_v = 0$
$0 = R_L + 0.33\ lb - 1\ lb$
$R_L = 0.67$

$\sum M_A = 0$
$0 = M_A - 0.67(13.33\ ft)$
$M_A = 8.931\ ft$

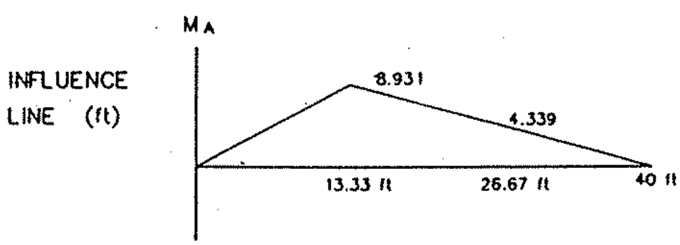




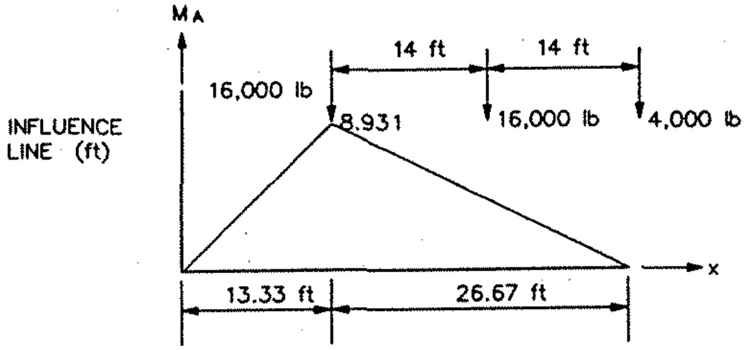
$\sum M_L = 0$
$0 = 1lb(26.67\ ft) - R_R(40\ ft)$
$R_R = 0.67$

$F_v = 0$
$0 = R_L + 0.67\ lb - 1\ lb$
$R_L = 0.33$

$\sum M_A = 0$
$0 = M_A - 0.33lb(13.33\ ft)$
$M_A = 4.399\ ft$



$M_{LL(HS20)}$: Place concentrated wheel load values on influence line to produce maximum moment.



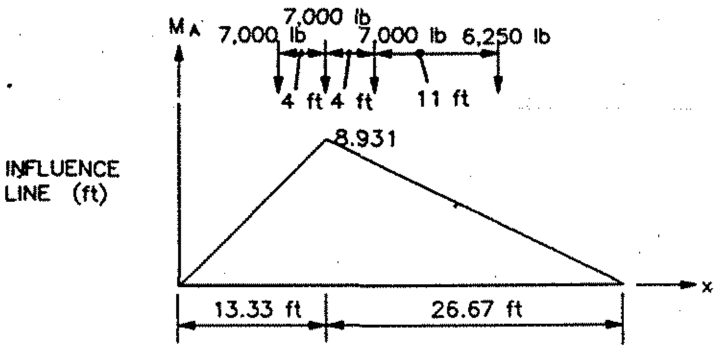
The influence line for a simply supported beam is linear. Values on the influence line at a specific points can be determined by proportioning.

$$M_{LL(HS20)} = 16{,}000\text{lb}(8.931\text{ft}) + 16{,}000\text{lb}\ \frac{26.67\text{ft}-14\text{ft}}{26.67\text{ft}}\ (8.931\text{ft}) +$$

$$4000\text{lb}(0)\quad \underbrace{(1.3)}_{\text{Impact}}\underbrace{(0.889)}_{\text{Lateral Distribution}}$$

$M_{LL(HS20)}$ = 243,599.574 ft-lb
where I = 1.3 and WF = 0.889

$M_{LL(Type\ 4)}$:



$$M_{LL(Type\ 4)} = \Big[(7000\text{ lb})\left(\frac{13.33\text{ft}-4\text{ft}}{13.33\text{ ft}}\right)(8.931\text{ft}) + (7000\text{lb})(8.931\text{ft}) + (7000\text{lb})\left(\frac{26.67\text{ft}-4\text{ft}}{26.67\text{ ft}}\right)(8.931\text{ft}) + (6250\text{lb})\left(\frac{26.67\text{ft}-15\text{ft}}{26.67\text{ ft}}\right)(8.931\text{ft})\Big](1.3)(0.889)$$

$$M_{LL(Type\ 4)} = 212{,}463.264\text{ ft-lb}$$

LIVE LOAD STRESSES AT THIRD POINT

$$f_{bLL(HS20)} = \frac{243{,}599.574\text{ ft-lb (12 in/ft)}}{79.881\text{ in}^3} = 36{,}594.370\text{ psi}$$

$$f_{bLL(Type\ 4)} = \frac{212.463.264\text{ ft-lb (12 in/ft)}}{79.881\text{ in}^3} = 31.916.966\text{ psi}$$

HS RATING AT THIRD POINT

$$\text{INVENTORY RATING} = \left(\frac{18000\text{ psi} - 4859.96\text{ psi}}{36{,}594.370\text{ psi}}\right)(36T) = 12.9\text{ tons}$$

$$\text{OPERATING RATING} = \left(\frac{24500\text{ psi} - 4859.96\text{ psi}}{36{,}594.370\text{ psi}}\right)(36T) = 19.3\text{ tons}$$

LOAD POSTING AT THIRD POINT

$$\text{TYPE 4} = \left(\frac{24500\text{ psi} - 4859.96\text{ psi}}{31{,}916.966\text{ psi}}\right)(27.25T) = 16.8\text{ tons}$$

Compare ratings at third point with ratings at midspan:

	RATING		
	Third Point	Mid-Span	Controlling
HS20 Inventory	12.9 T	13.5 T	12.9 T
HS20 Operating	19.3 T	20.2 T	19.3 T
Type 4	16.8 T	17.3 T	16.8 T*

* post for 17 tons

Third point with reduced cross-section controls rating.

3.3.6.7. Example 7: Rating of timber deck (4 in. x 12 in.)

GIVEN

Same as Example 6 (since Example 6 controls over Example 4)

Check rating for Transverse Timber Deck (4 in. x 12 in.)

Allowable Bending Stress: INVENTORY = 1450 psi
OPERATING = 1450 x 1.33 = 1929 psi

DEAD LOADS

Span Length = Clear distance between stringers + 1/2 width of stringers

(but shall not exceed clear span + floor thickness)
AASHTO 3.25.1.2

Span Length = (3ft-4in) - 6in + 1/2(6in) = 3 ft-1 in. controls
$\leq$ (3ft-4in) - 6in + 4in = 3 ft-2 in.

Span Length = 3 ft-1 in. = 3.083 ft

$$w_{PLANK} = \left(\frac{4in(12in)}{144\ in^2}\right)(50\ lb/ft^3) = 16.67\ lb/ft$$

$$w_{ROCK} = \left(\frac{2in(12in)}{144\ in^2}\right)(120\ lb/ft^3) = 20.00\ lb/ft$$

$$w_{DL} = 36.67\ lb/ft$$

M_{DL}: If flooring is continous over more than two spans, the maximum bending moment shall be assumed as being 80 percent of that obtained for a simple span. (AASHTO 3.25.3)

$$M_{DL} = 1/8(36.67\ lb/ft)(3.083ft)^2(0.8)$$

$$M_{DL} = 34.86\ ft\text{-}lb$$

$$S_b = 1/6(12in)(4in)^2$$
$$S_b = 32\ in^3$$

$$f_{bDL} = \frac{M_{DL}}{S_b} = \frac{(34.86\ ft\text{-}lb)(12\ in/ft)}{32\ in^3}$$

$$f_{bDL} = 13.07\ psi$$

LIVE LOADS

In design of timber floors for HS20 loading, one axle load of 24,000 lb (= two wheel loads of 12,000 lb) may be used instead of the 32,000 lb axle load (16,000 lb wheel loads) generally used. (See Fig. 3.6)

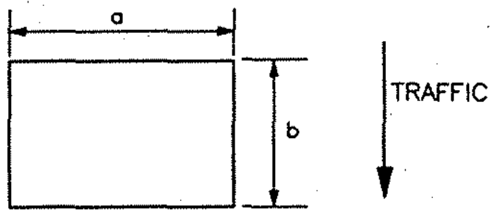
Live Load_{HS20} = 12,000 lb

Live $\text{Load}_{TYPE\ 3}$ = 8,500 lb

<u>HS20</u>

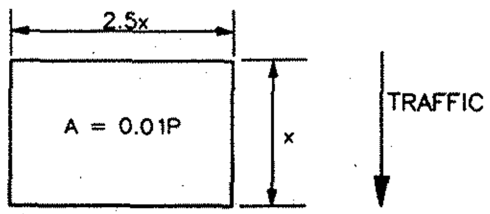
Distribution of wheel load:

Surface plan area over which wheel load is applied.



To find a: In direction of bridge span, the wheel load shall be distributed over the width of the tire as given by the ratio shown below. (AASHTO 3.25.1.1)

(AASHTO 3.30)



P = wheel load in pounds

$A = 0.01P$

$A = 0.01(12{,}000\ \text{lb})$

$A = 120\ \text{in}^2$

$2.5x^2 = A$

$x = \sqrt{\dfrac{120\ \text{in}^2}{2.5}} = 6.93\ \text{in.}$

$a = 2.5x$

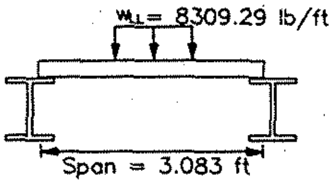
$a = 2.5(6.93\ \text{in.})$

$a = 17.33\ \text{in.}$

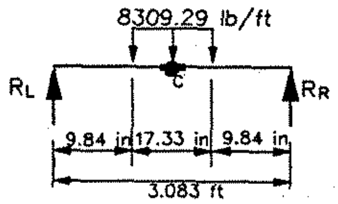
b = width of plank (AASHTO 3.25.1.1)

b = 12 in.

$$w_{LL} = \frac{12000lb(12in/ft)}{17.33\ in.} = 8309.29\ lb/ft\ of\ plank$$



Impact = 0% (Timber)



$\sum F_v = 0$
$0 = 2R - 8309.29lb/ft(17.33in)(1ft/12in)$
$R = 6000\ lb$

$M_c = 6000lb(3.083ft/2)-$
$8309.29lb/ft(17.33in/12in/ft)(1/2)(17.33in/12)(1/4)$
$M_c = 7083.75\ ft\text{-}lb$

$M_{LL\ HS20} = 7083.75\ ft\text{-}lb(0.8) = 5667.00\ ft\text{-}lb$

$$f_{bLL\ (HS20)} = \frac{5667.00ft\text{-}lb(12in/ft)}{32\ in^3} = 2125.13\ psi$$

<u>Type 3</u>

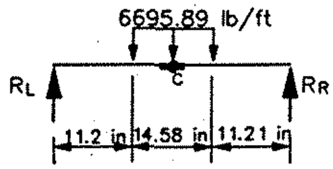
$A = 0.01P$
$A = 0.01(8,500\ lb)$
$A = 85\ in^2$

$2.5x^2 = A$
$$x = \sqrt{\frac{85\ in^2}{2.5}} = 5.83\ in.$$

$a = 2.5x$
$a = 2.5(5.83in)$
$a = 14.58\ in$

$b = 12\ in.$

$$w_{LL} = \frac{8500\text{lb}(12\text{in/ft})}{14.58\text{ in}} = 6995.89\text{ lb/ft of plank}$$



$$\sum F_v = 0$$
$$0 = 2R - 6995.89\text{lb/ft}(14.58\text{in})(1\text{ft}/12\text{in})$$
$$R = 4250\text{ lb}$$

$$M_c = 4250\text{lb}(3.083\text{ft}/2) - 6995.89\text{lb/ft}(14.58\text{in}/12\text{in/ft})(1/2)(14.58\text{in}/12)(1/4)$$
$$M_c = 5261.15\text{ ft-lb}$$

$$M_{LL\ Type\ 3} = 5261.15\text{ ft-lb}(0.8) = 4208.92\text{ ft-lb}$$

$$f_{bLL\ (Type\ 3)} = \frac{4208.92\text{ft-lb}(12\text{in/ft})}{32\text{ in}^3} = 1578.35\text{ psi}$$

HS20 RATING

$$\text{INVENTORY RATING} = \left(\frac{1450\text{ psi} - 13.07\text{ psi}}{2125.13\text{ psi}}\right)(36\text{T}) = \underline{24.3\text{ tons}}$$

$$\text{OPERATING RATING} = \left(\frac{1929\text{ psi} - 13.07\text{ psi}}{2125.13\text{ psi}}\right)(36\text{T}) = \underline{32.5\text{ tons}}$$

Values do not control rating; rating controlled by stringer (See Example 6)

LOAD POSTING

$$\text{TYPE 3} = \frac{1929\text{ psi} - 13.07\text{ psi}}{1578.35\text{ psi}}(25\text{T}) = \underline{30.3\text{ tons}}$$

Values do not control rating; rating controlled by stringer (See Example 6)

3.3.6.8. Example 8: Rating of timber deck (3 in. x 12 in.)

GIVEN

Same as Example 6 (since Example 6 controls over Example 4) with 3 in. x 12 in. transverse plank deck.

Check Rating for deck.

DEAD LOADS

$$\text{Span Length} = (3\text{ft-}4\text{in}) - 6\text{in} + 1/2(6\text{in}) = 3\text{ ft-}1\text{ in. controls}$$
$$\leq (3\text{ft-}4\text{in}) - 6\text{in} + 3\text{in} = 3\text{ ft-}1\text{ in.}$$

$$\text{Span Length} = 3\text{ ft-}1\text{ in.} = 3.083\text{ft}$$

$$w_{PLANK} = \left(\frac{3\text{in}(12\text{in})}{144\text{ in}^2}\right)(50\text{ lb/ft}^3) = 12.50\text{ lb/ft}$$

$$w_{ROCK} = \left(\frac{2\text{in}(12\text{in})}{144\text{ in}^2}\right)(120\text{ lb/ft}^3) = 20.00\text{ lb/ft}$$

$$w_{DL} = 32.50\text{ lb/ft}$$

$$M_{DL} = (1/8)(32.50\text{ lb/ft})(3.083\text{ft})^2(0.8)$$

$$M_{DL} = 30.90\text{ ft-lb}$$

$$S_b = 1/6(12\text{in})(3\text{in})^2$$
$$S_b = 18.00\text{ in}^3$$

$$f_{bDL} = \frac{M_{DL}}{S_b} = \frac{(30.90\text{ ft-lb})(12\text{ in/ft})}{18\text{ in}^3}$$

$$f_{bDL} = 20.60\text{ psi}$$

LIVE LOADS

$$f_{bLL(HS20)} = \frac{5667.00\text{ ft-lb}(12\text{ in/ft})}{18.00\text{ in}^3} = 3778.00\text{ psi}$$

$$f_{bLL(TYPE\ 3)} = \frac{4208.92\text{ ft-lb}(12\text{ in/ft})}{18.00\text{ in}^3} = 2805.95\text{ psi}$$

HS20 RATING

$$\text{INVENTORY RATING} = \left(\frac{1450\text{ psi} - 20.60\text{ psi}}{3778.00\text{ psi}}\right)(36T) = \underline{13.6\text{ tons}}$$

13.6 tons rating for deck > 12.9 tons rating for stringer (Example 6)

HS20 INVENTORY RATING = 12.9 tons

Stringer Controls

$$\text{OPERATING RATING} = \left(\frac{1929\text{ psi} - 20.60\text{ psi}}{3778.00\text{ psi}}\right)(36T) = \underline{18.2\text{ tons}}$$

18.2 tons rating for deck < 19.3 ton rating for stringer (Example 6)

HS20 OPERATING RATING = 18.2 tons

Deck Controls

LOAD POSTING

$$\text{TYPE 3} = \left(\frac{1929\text{ psi} - 20.60\text{ psi}}{2805.95\text{ psi}}\right)(25T) = \underline{17.0\text{ tons}}$$

17.0 ton rating for deck = 17.0 ton rating for stringer (Example 6)

POST FOR 17 tons

Stringers and Deck control equally.

3.3.6.9. Example 9: Rating of laminated timber deck (2 in. x 4 in.)

GIVEN

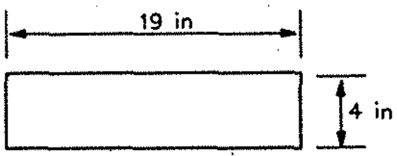
Same as Example 6 (since Example 6 controls over Example 4) with 2 in. x 4 in. continous nail laminated deck.

Check Rating for deck.

DEAD LOADS

Determine member size as described in AASHTO 3.25.1.1

15in + floor thickness = 15 in. + 4 in. = 19 in.



$$S_b = (1/6)(19\text{in})(4\text{in})^2 = 50.67\ \text{in}^3$$

Span Length = 3.083ft (see Example 7)

$$w_{DECK} = \left(\frac{19\text{in}(4\text{in})}{144\ \text{in}^2/\text{ft}^2}\right)(50\ \text{lb/ft}^3) = 26.39\ \text{lb/ft}$$

$$w_{ROCK} = \left(\frac{19\text{in}(2\text{in})}{144\ \text{in}^2/\text{ft}^2}\right)(120\ \text{lb/ft}^3) = 31.67\ \text{lb/ft}$$

$$w_{DL} = 58.06\ \text{lb/ft}$$

$$M_{DL} = (1/8)(58.06\ \text{lb/ft})(3.083\text{ft})^2(0.8)$$

$$M_{DL} = 55.19\ \text{ft-lb}$$

$$f_{bDL} = \frac{M_{DL}}{S_b} = \frac{(55.19\ \text{ft-lb})(12\ \text{in/ft})}{50.67\ \text{in}^3}$$

$$f_{bDL} = 13.07\ \text{psi}$$

LIVE LOADS

$$f_{bLL(HS20)} = \frac{5667.00 \text{ ft-lb}^{*}(12 \text{ in/ft})}{50.67 \text{ in}^3} = 1342.10 \text{ psi}$$

$$f_{bLL(TYPE\ 3)} = \frac{4208.92 \text{ ft-lb}^{*}(12 \text{ in/ft})}{50.67 \text{ in}^3} = 996.78 \text{ psi}$$

*(See Example 7)

HS20 RATING

$$\text{INVENTORY RATING} = \left(\frac{1450 \text{ psi} - 13.07 \text{ psi}}{1342.10 \text{ psi}}\right)(36T) = \underline{38.5 \text{ tons}}$$

$$\text{OPERATING RATING} = \left(\frac{1929 \text{ psi} - 13.07 \text{ psi}}{1342.10 \text{ psi}}\right)(36T) = \underline{51.4 \text{ tons}}$$

Values do not control rating; rating controlled by stringer - (See Example 4)

LOAD POSTING

$$\text{TYPE 3} = \left(\frac{1929 \text{ psi} - 13.07 \text{ psi}}{996.78 \text{ psi}}\right)(25T) = \underline{48.1 \text{ tons}}$$

Value does not control rating; rating controlled by stringer - (See Example 6)

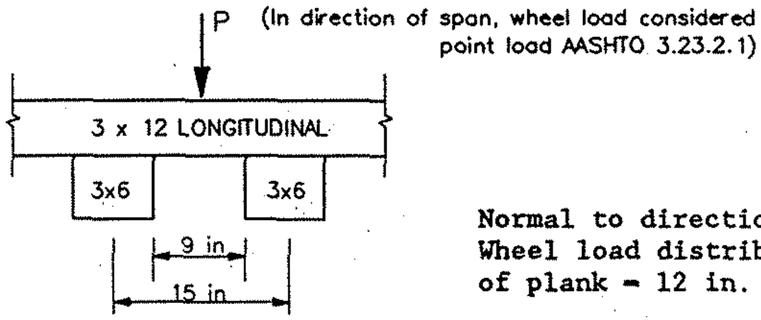
3.3.6.10. Example 10: Rating of layered timber deck

GIVEN

Same as Example 6 with layered deck,
3 in. x 6 in. @ 15 in. transverse planks and
3 in. x 12 in. adjacent longitudinal planks.

Check rating for deck planks.

LONGITUDINAL PLANKS



Normal to direction of span.
Wheel load distributed through width of plank = 12 in. (AASHTO 3.25.2.2)

Span = Clear Distance + 1/2 beam width $\leq$ Clear Span + floor thickness

Span = 9in + 1/2(6in) = 12in $\leq$ 9in + 3in = 12in = 1 ft

Longitudinal Dead Load

$$w_{PLANK} = \left(\frac{3in(12in)}{144\ in^2/ft^2}\right)(50\ lb/ft^3) = 12.50\ lb/ft$$

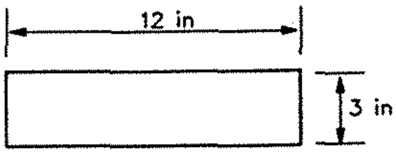
$$w_{ROCK} = \left(\frac{2in(12in)}{144\ in^2/ft^2}\right)(120\ lb/ft^3) = 20.00\ lb/ft$$

$$w_{DL} = 32.50\ lb/ft$$

$$M_{DL} = (80\%)(1/8)(32.50\ lb/ft)(1\ ft)^2$$

$$M_{DL} = 3.25\ ft\text{-}lb$$

Section Modulus; Longitudinal plank



$$S_b = (1/6)(12\text{in})(3\text{in})^2$$
$$S_b = 18.00\ \text{in}^3$$

$$f_{DL} = \frac{3.25\ \text{ft-lb}\ (12\ \text{in/ft})}{18.00\ \text{in}^3} = 2.17\ \text{psi}$$

Longitudinal Live Load

$$M_{LL(HS20)} = 80\%(PL/4) = \frac{0.8(1200)(1\ \text{ft})}{4} = 2400\ \text{lb-ft}$$

$$M_{LL(TYPE\ 3)} = \frac{0.8(8500)(1\ \text{ft})}{4} = 1700\ \text{ft-lb}$$

$$f_{bLL(HS20)} = \frac{2400\ \text{lb-ft}\ (12\ \text{in/ft})}{18.00\ \text{in}^3} = 1600.00\ \text{psi}$$

$$f_{bLL(TYPE\ 3)} = \frac{1700\ \text{ft-lb}\ (12\ \text{in/ft})}{18.00\ \text{in}^3} = 1133.33\ \text{psi}$$

Longitudinal Rating/Posting

HS20:

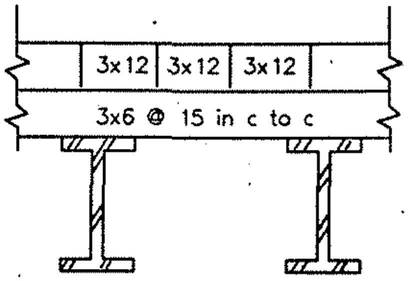
$$\text{INVENTORY RATING} = \left(\frac{1450\ \text{psi} - 2.17\ \text{psi}}{1600\ \text{psi}}\right)(36\text{T}) = \underline{32.6\ \text{tons}}$$

$$\text{OPERATING RATING} = \left(\frac{1929\ \text{psi} - 2.17\ \text{psi}}{1600\ \text{psi}}\right)(36\text{T}) = \underline{43.4\ \text{tons}}$$

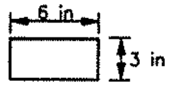
$$\text{TYPE 3} = \left(\frac{1929\ \text{psi} - 2.17\ \text{psi}}{1133.3\ \text{psi}}\right)(25\text{T}) = \underline{42.5\ \text{tons}}$$

TRANSVERSE PLANKS

Span Length = 3.083 ft (See Example 7)



Cross Section 3x6



$S_b = 1/6(6\text{in})(3\text{in})^2$
$S_b = 9\ \text{in}^3$

Transverse Dead Load

$$W_{DL(long\ plank)} = \left(\frac{3\ \text{in}}{12\ \text{in/ft}}\right)\left(\frac{15\ \text{in}}{12\ \text{in/ft}}\right)\left(50\ \frac{\text{lb}}{\text{ft}^3}\right) = 15.625\ \text{lb/ft}$$

$$W_{DL(trans\ plank)} = \left(\frac{3\text{in}(6\text{in})}{144\ \text{in}^2/\text{ft}^2}\right)(50\ \text{lb/ft}^3) = 6.250\ \text{lb/ft}$$

$$W_{DL\ (ROCK)} = \left(\frac{2\text{in}(15\text{in})}{144\ \text{in}^2/\text{ft}^2}\right)(120\ \text{lb/ft}^3) = 25.00\ \text{lb/ft}$$

$$W_{DL} = 46.875\ \text{lb/ft}$$

$M_{DL} = 0.8(1/8)(46.875\ \text{lb/ft})(3.083\ \text{in})^2$
$M_{DL} = 44.55\ \text{ft-lb}$

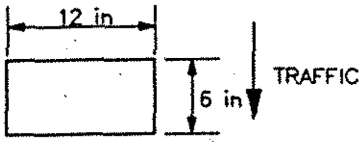
$$f_{bDL} = \frac{M_{DL}}{S_b} = \frac{(44.55\ \text{ft-lb})(12\ \text{in/ft})}{9.00\ \text{in}^3}$$

$f_{bDL} = 59.40\ \text{psi}$

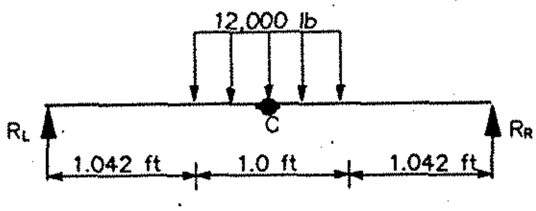
Transverse Live Load

Distribution of wheel load

Surface plan area of wheel load on transverse plank



HS20:



$$W_{LL(HS20)} = \frac{12000lb(12\ in/ft)}{12\ in} = 12000\ lb/ft$$

$$W_{LL(Type\ 3)} = \frac{8500lb(12\ in/ft)}{12\ in} = 8500\ lb/ft$$

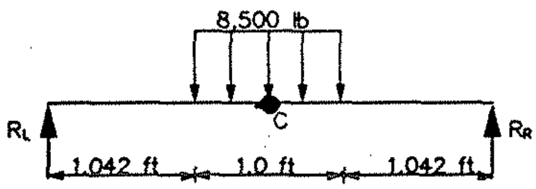
$\sum F_v = 0$
$R = 6000\ lb$

$M_c = 6000lb(3.083ft/2)-12000(1ft/2)(1ft/4)$
$M_c = 7749\ ft\text{-}lb$

$M_{LL(HS20)} = 0.8(7749ft\text{-}lb) = 6199.20\ ft\text{-}lb$

$$f_{bLL(HS20)} = \frac{6199.20ft\text{-}lb(12\ in/ft)}{9\ in^3} = 8265.60\ psi$$

TYPE 3:



$\sum F_v = 0$
$R = 4250\ lb$

$M_c = 4250lb(3.083ft/2)-8500(1ft/2)(1ft/4)$
$M_c = 5488.88\ ft\text{-}lb$

$M_{LL(HS20)} = 0.8(5488.88ft\text{-}lb) = 4391.10\ ft\text{-}lb$

$$f_{bLL(HS20)} = \frac{4391.10ft\text{-}lb(12\ in/ft)}{9\ in^3} = 5854.80\ psi$$

<u>Transverse Rating/Posting</u>

HS20:

$$\text{INVENTORY RATING} = \left(\frac{1450\text{ psi} - 59.40\text{ psi}}{8265.60\text{ psi}}\right)(36T) = \underline{6.1\text{ tons}}$$

$$\text{OPERATING RATING} = \left(\frac{1929\text{ psi} - 59.40\text{ psi}}{8265.60\text{ psi}}\right)(36T) = \underline{8.1\text{ tons}}$$

$$\text{TYPE 3} = \left(\frac{1929\text{ psi} - 59.40\text{ psi}}{5854.80\text{ psi}}\right)(25T) = \underline{8.0\text{ tons}}$$

	RATING			
	Stringer	Longitudinal Plank	Transverse Plank	** Controlling
HS20 Inventory	12.9 T	32.6 T	6.1 T	6.1 T
HS20 Operating	19.3 T	43.4 T	8.1 T	8.1 T
TYPE 3	16.8 T	42.5 T	8.0 T	8.0 T

Post for 8 Tons.

** This indicates that the deck analyzed in this example should be modified to at least the capacity of the stringers or replaced with one of the decks described in the previous examples.

3.4. Economic Analysis

The bridge engineer must compare three alternatives when evaluating a deficient bridge: 1) replace the existing bridge, 2) rehabilitate or strengthen the existing bridge, or 3) leave the bridge in present condition. To make a rational decision among these three alternatives, the engineer must take several factors into account. These factors include: 1) budgetary constraints, 2) potential benefits to the community, and 3) elimination of a safety hazard. Although many of these factors require qualitative analysis, a rational decision based on economic factors, can be made by quantifying the dollar value of each alternative and selecting the one which is the most cost-effective over the life of the structure.

3.4.1. Background

Review of existing literature indicated that several economic evaluation methods have been used in the past. The NBI program established a numerical sufficiency rating for each bridge in the United States to determine its eligibility for federal funding for rehabilitation or replacement. The tremendous amount of information resulting from the inspection of the nation's bridges for the NBI program has necessitated a system for prioritizing bridge rehabilitation and replacement on a national basis. A program has been developed by the North Carolina DOT which ranks bridge replacement and rehabilitation projects on the basis of deficiency points accumulated in the NBI inspection and rating process (33). While this prioritizing is useful, it does not determine the most cost-effective solution to the problems of a particular bridge.

The B/C ratio method has been used in numerous engineering economic analyses to evaluate alternatives. This method was especially useful because it not only determines which alternative is the most cost-effective, it also prioritizes the remaining alternatives. After serious consideration, the B/C ratio method was rejected for this study because of the difficulty in quantifying the potential benefits of bridge rehabilitation or replacement.

A life-cycle cost method has been used in numerous studies to develop cost-effectiveness decision-making tools for bridge engineers (91). In these methods, a series of cash flows is converted to a common reference by the use of the time-value of money equations. According to Winfrey (93), there are four types of life-cycle analysis methods: 1) the equivalent uniform annual cost method, 2) present worth of costs method, 3) equivalent uniform and annual net return method, and 4) net present value method.

One problem with a life-cycle cost analysis is the difficulty in determining the future costs associated with a particular alternative. For example, some bridge strengthening methods may require more and different types of maintenance than other methods performed at the same time. The development of comprehensive bridge management systems by state DOTs should help to alleviate this problem in the future. Bridge management systems are presently being developed and these results will become available in the near future (61).

A significant amount of research has been applied to the use of value engineering in the design and construction of low volume road bridges. Value engineering is defined as "the systematic application of recognized techniques which identify the function of a product or service, establish a monetary value for that function, and provide the necessary function reliably at the lowest overall cost" (90). One product of this value engineering process is a value graph, which is a plot of a bridge component's importance vs. cost.

For this study, a life-cycle test analysis procedure has been used (i.e. Equivalent Uniform Annual Cost method). For additional information on these and other procedures for the comparison of alternatives, the reader is directed to Refs. 10,34,45,59,87.

3.4.2. Description of Analytical Model

The analytical models developed for bridge replacement and/or strengthening alternatives are shown in Figs. 3.8 and 3.9, respectively. The use of a cash-flow diagram is the standard method for describing a series of economic transactions. The horizontal line is a time scale, with the progression of time from left to right. The arrows signify cash flows - downward arrows represent disbursements and upward arrows represent receipts (75). Mathematical expressions can be developed from the cash-flow diagrams based on the time value of money method found in any engineering economy text (53).

The economic model which is presented here was originally developed in a study related to primary system bridges (95). Although general principals of economic analysis will continue to provide the background of the discussion, many of the considerations used in the original model have been modified to adapt the model specifically for low volume road bridges. For the replacement model (Fig. 3.8), the equivalent uniform annual cost, $EUAC_r$ is given by Eqn. (1).

$$EUAC_r = (\frac{A}{P},i,N)(R) - (\frac{A}{P},i,N)(B) - (\frac{A}{F},i,N)(S) + C_{ar} + (\frac{A}{P},i,N)\sum_{j=1}^{m}[F_j \cdot (\frac{P}{F},i,n_j)] \qquad (1)$$

where,

R = replacement structure first cost

B = net salvage value of existing structure

C_{ar} = annual maintenance cost of replacement structure

S = net salvage value of replacement structure

N = service life of replacement structure

i = interest rate

$F_{j,k}$ = single future disbursement

n_j = year of future expenditure (present = year 0)

$(A/P,i,N)$ = capital recovery factor = $\dfrac{i(1+i)^N}{(1+i)^N - 1}$

$(A/F,i,N)$ = sinking fund factor = $\dfrac{i}{(1+i)^N - 1}$

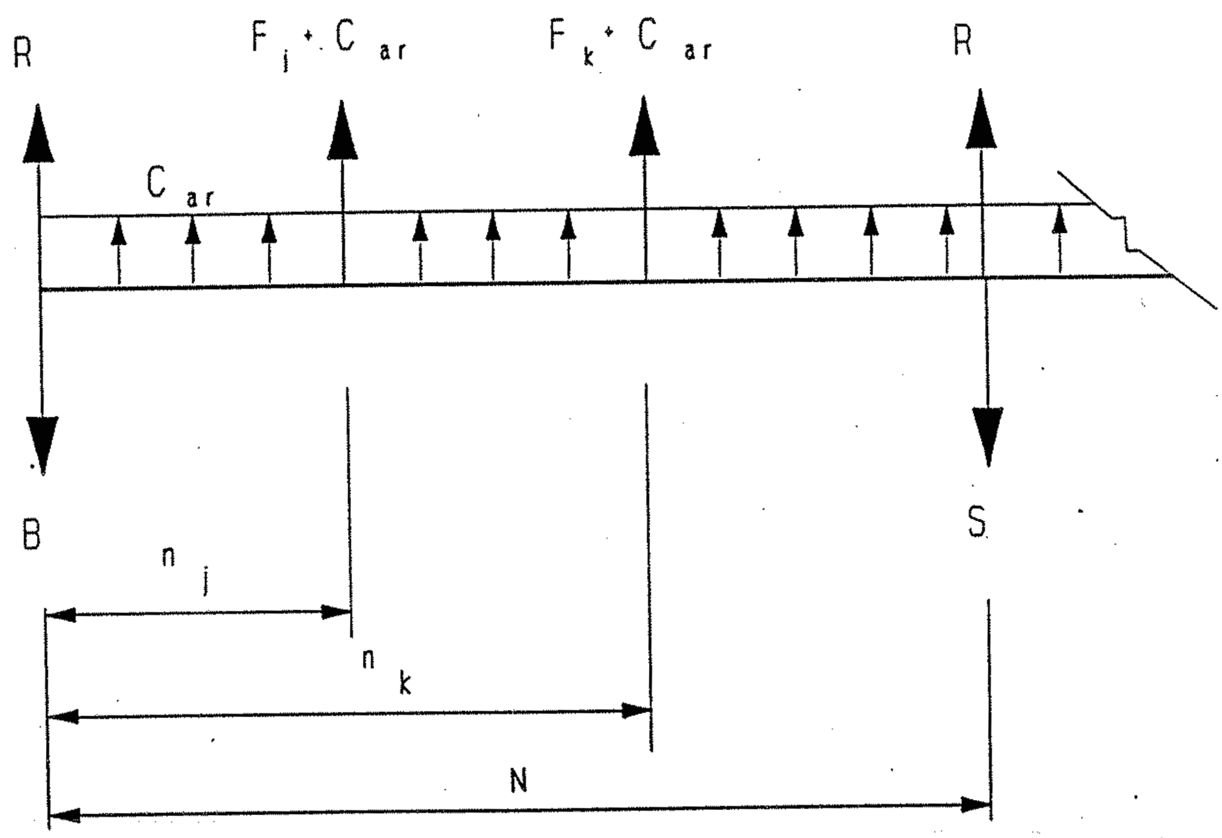


Fig. 3.8. Economic model for replacement alternatives.

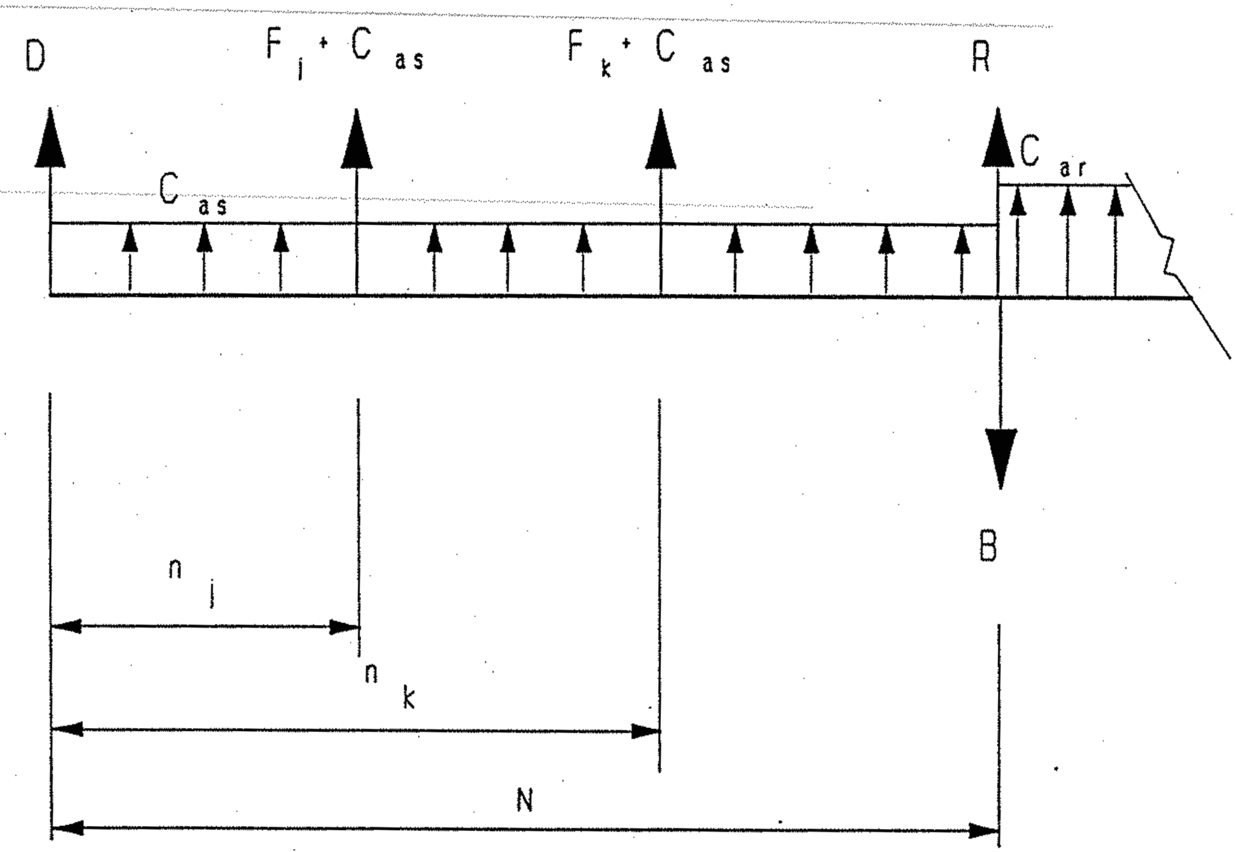


Fig. 3.9. Economic model for strengthening alternatives.

(P/F,i,n_j) = present worth of a future sum = $\dfrac{1}{(1+i)^{n_j}}$

Salvage values can be either positive or negative. In most cases, little salvage value remains in an existing bridge, and the bridge owner must pay for the removal of the bridge. This removal cost must be represented as a negative salvage value in the EUAC equations. If N is assumed to be very large (e.g. > 50 years) and B to be relatively small compared to R, Eqn. (1) can be simplified to Eqn. (2).

$$EUAC_r = (\frac{A}{P},i,N)(R-S) + C_{ar} + (S-B)i + (\frac{A}{P},i,N)\sum_{j=1}^{m}[F_j \cdot (\frac{P}{F},i,n_j)] \qquad (2)$$

There are two significant advantages to using Eqn. (2) rather than Eqn. (1). In most cases, the removal cost of the existing structure, B, will be approximately that of the replacement structure, S. Therefore, the (S-B)i term becomes insignificant and can be ignored. In addition, the bid price of the replacement structure normally includes the removal cost of the existing structure. If this is the case, the (R-S) term in Eqn. (2) will be the total bid price of the replacement structure (95).

To develop an economic model to represent the bridge strengthening procedure, two significant assumptions must be made. First, money spent to strengthen an existing bridge only benefits the existing structure. Therefore, disbursements made to strengthen the existing bridge must be evaluated only over the remaining life of the existing bridge. Secondly, after the existing bridge is eventually replaced, all costs are considered to be common to both the replacement and strengthening model.

The mathematical model to represent bridge strengthening or rehabilitation (Fig. 3.9) is shown in Eqn. (3).

$$EUAC_s = (\frac{A}{P},i,N')(D) + C_{as} + (\frac{A}{P},i,N')\sum_{j=1}^{m}[F_j \cdot (\frac{P}{F},i,n_j)] \qquad (3)$$

where

D	=	initial cost of strengthening existing bridge
C_{as}	=	annual maintenance cost of existing structure after strengthening
N′	=	remaining service life of existing bridge
$F_{j,k}$	=	single future disbursement
n_j	=	year of future disbursement

(A/P,i,N′) = capital recovery factor = $\dfrac{i(1+i)^{N}}{(1+i)^{(N-1)}}$

$(P/F,i,n_j)$ = present worth of a future sum = $\frac{1}{(1+i)^{n_j}}$

A brief description of the variables included in the economic models for bridge strengthening and replacement follows. A database of estimated prices is presented in Sec. 3.7 of this report for various replacement methods.

3.4.2.1. Replacement Structure First Cost (R)

The first cost of a bridge replacement structure has the greatest effect on the EUAC of any of the factors. Fortunately, the first cost of a bridge is the easiest factor to quantify accurately. There are at least four considerations which can significantly effect the first cost of a replacement bridge:

- Span length: In general, the cost per ft^2 of a new bridge tends to increase with longer span lengths. Low volume bridges tend to have much shorter spans, making this variable less significant.
- Roadway realignment: Alignment changes in low volume roads, and thus bridges, are less frequent than in a more highly traveled roadway.
- Environmental studies and potential consequences: The effect of a low volume bridge replacement project on the environment tends to be of a localized nature, which limits the relevance of this variable.
- Construction of temporary detours: The construction of temporary detours would be infrequent on low volume roads. Usually there are a small number of users being inconvenienced, thus most likely the existing road would simply be closed during construction of a new bridge.

3.4.2.2. Structure Service Life (N,N')

It is common practice to assign a service life of 50 years to new bridges in an economic analysis. This common "rule-of-thumb" may be influenced by the geographical and climatological location of the proposed bridge, however. Factors which affect the service life of a new bridge are 1) the quality of the initial design, 2) the quality of materials used in construction, 3) the quality of workmanship used in construction, 4) the level of routine maintenance performed on the structure during the life cycle, and 5) the severity of the climate.

The remaining service life of an existing structure is most often estimated by an engineer after completing an inspection and thus is very difficult to quantify accurately. There are very few detailed guidelines available for this purpose, and it may be difficult to avoid personal bias in the estimate.

3.4.2.3. Interest Rate (i)

Historically, long term economic analyses of public works projects have ignored the effect of inflation on the interest rate used in computing equivalent costs. Cady has determined that, due to the present national economy, inflation must be considered in any life-cycle cost analysis (14).

A relationship which accounts for the difference between the rate of inflation and the nominal interest rate is presented in Eqn. (4):

$$i_0 = \frac{1 + i_p}{1 + y} - 1 \qquad (4)$$

where

i_0 = real or effective interest rate

i_p = nominal interest rate (usually based on high-grade municipal bonds)

y = rate of inflation (usually based on changes in the consumer price index)

In general, a higher nominal interest rate tends to favor future expenditures (e.g. present strengthening with possible future replacement), while a lower nominal interest rate favors the larger immediate capital investment (e.g. immediate bridge replacement) (93).

3.4.2.4. Bridge Maintenance Costs (C_{ar}, C_{as})

The most difficult factor to predict in any economic model is the annual maintenance costs of the bridge, both as it exists today, and after any strengthening or rehabilitation work.

Large, one-time, maintenance expenditures, such as a bridge deck overlay, should not be included in the C_{ar} and C_{as} terms. These types of expenses are represented by the single future disbursement terms, F_j and F_k, in Figs. 3.8 and 3.9 and should be converted to a present worth by the present worth of a future sum equation and then to an EUAC by the capital recovery factor.

3.4.2.5. Bridge Removal Costs (B,S)

Several factors which can significantly affect the cost of bridge removal are: superstructure type, span length, number and type of abutments, depth of removal below ground line, and any environmental precautions which must be taken.

Klaiber, et. al. have shown that bridge width is not a significant factor in estimating removal costs (36). As mentioned earlier, one advantage of using Eqn. 2 rather than Eqn. 1 to model the replacement of a bridge is that both B and S can be ignored.

3.4.2.6. Level of Service Factor (LS)

The Level of Service Factor, LS, was introduced by Wipf, et. al. (95) as a means of quantifying the economic benefits a road user would realize with the construction of a new bridge. A new bridge can be expected to provide reduced accident rates, reduced traffic delays, a reduction in detour mileage for trucks which exceed the posted load limit and other intangible savings over an existing bridge. These reductions are an additional cost of keeping an existing bridge in service and are represented as an additional cost in the strengthening alternative.

In previous studies by McFarland (45), the traffic accident rate at the bridge site is related to the roadway approach width and the bridge width. A reduction in accident rate which could be expected with an increase in bridge width has been calculated.

The present study on low volume road bridges required a modification of the level of service factor previously defined. The three considerations in the level of service factor, reduced accident rate, reduced traffic delay, and reduced detour length do not apply in a low ADT environment. Since there is very little traffic, it is unlikely that a traffic delay will occur due to existing bridge conditions. Similarly, since there is so little traffic, there can be very few accidents which occur on the existing bridge. The reduction of already small accident rates is statistically insignificant. Finally, the concept of a detour length due to an understrength bridge is difficult to define. There are so few users of a particular bridge that, at least in most cases, it would not be a significant inconvenience to require a detour to the next county road (usually located one mile away).

3.4.2.7. Other Considerations

In addition to the variables in the economic model, the practicing engineering must take other considerations into account when making decisions regarding bridge rehabilitation or replacement. These considerations are usually very difficult to quantify. The engineer should not allow personal desires to completely override the decision making process. The most cost-effective solution, as determined by the EUAC method, should not be discarded due to the dislike of the particular procedure. The quantifiable "best" alternative should, however, be reviewed with sound engineering judgement.

3.4.3. Normal Distribution and EUAC Simulation

The economic model presented represents a simulation of possible outcomes. The service life of a particular alternative and the nominal interest rate used for calculations are not certain. To determine the range of possible outcomes, probability theory can be used for these variables.

The normal probability distribution of variables is well known and simple to apply. For this discussion it is assumed that the service life and nominal interest rate are normally distributed. There are several tests available which allow the user to determine whether a particular data set is normally distributed (18).

3.4.3.1. Normal Probability Distribution

The normal probability distribution, also known as the Gaussian distribution, is the most frequently encountered probability distribution. The normal distribution is recognizable as the familiar "bell-shaped curve" (see Fig. 3.10). Two parameters completely describe a particular normal curve: the mean, μ, which locates the center of the curve, and the standard deviation, σ, which provides a measure of the degree of dispersion. The height of a normal curve above any point along the horizontal axis is the relative frequency. This height can be described by:

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

Every normal frequency curve is centered on the mean. The tails of the curve taper off rapidly for values of x very far from the mean. The area under the normal curve between μ and any point depends only on the

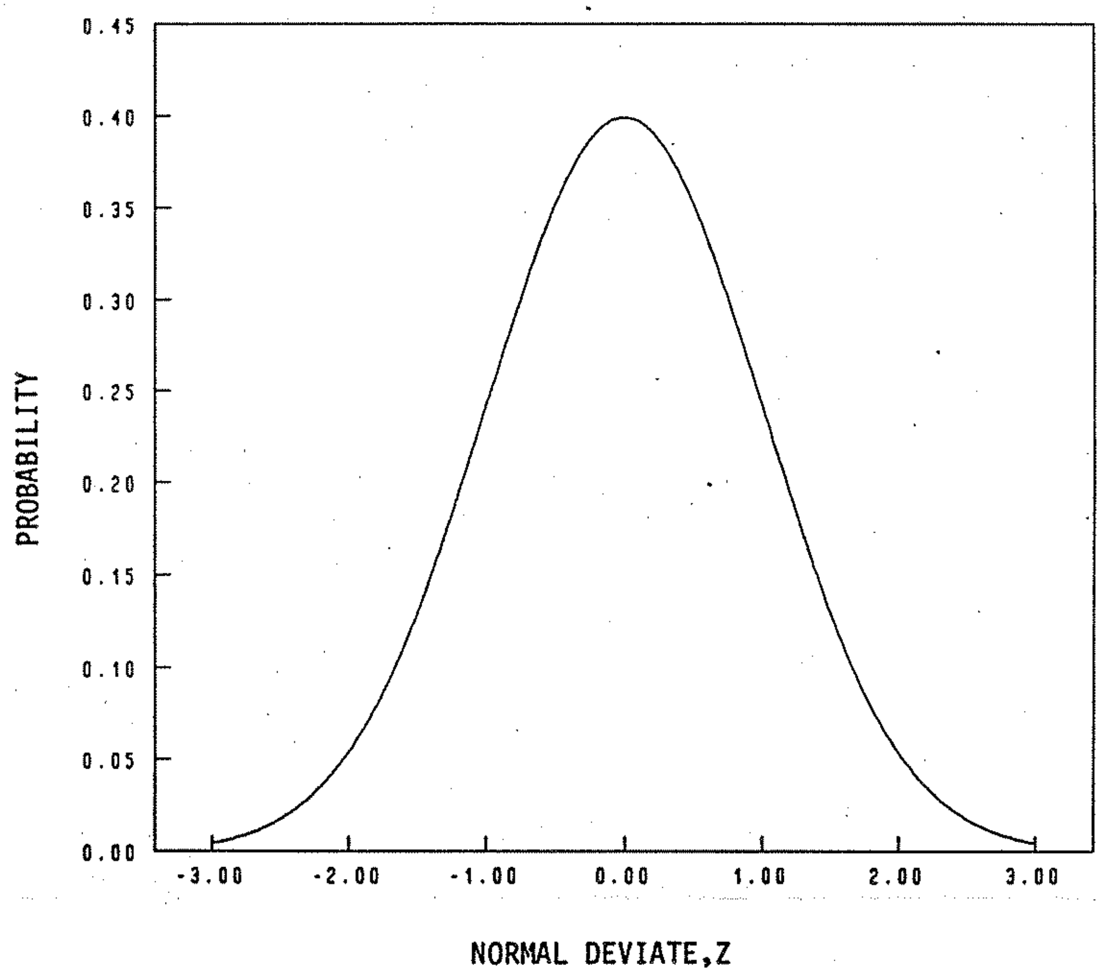


Fig. 3.10. Normal probability distribution.

distance separating that point and the mean, as expressed in units of σ. This distance from the mean is known as the normal deviate, designated as z.

$$z = \frac{x - \mu}{\sigma}$$

Since a normally distributed variable is continuous, probabilities for an event X can be found by using:

$$F(x) = Pr[X \leq x] = \int_{-\infty}^{x} \frac{1}{\sqrt{2\pi}\,\sigma} e^{-\frac{(y-\mu)^2}{2\sigma^2}}$$

This integral is rather complicated to compute. Fortunately, any statistics manual contains of table of cumulative probability values for the normal distribution (39).

For purposes of this economic model, we shall consider only those events which occur within 3 standard deviations of the mean, that is: $-3.00 \leq z \leq +3.00$. This region of the normal distribution includes 99.73 percent of all possible events.

3.4.3.2. Simulation of EUAC Results

To simulate the uncertainty in possible results of an EUAC calculation, the region between z = ±3 has been divided into 25 intervals. Each inteival represents 4 percent of the total area under the normal probability curve. These intervals are essentially the 0th, 4th, 8th, etc. percentile of possible outcomes. The z value for the endpoint of each interval is computed, and converted into a value of service life and interest rate. For example, the 0th percentile is equivalent to a z value of -3.00, the 4th percentile is equivalent to a z value of -1.76, etc. These possible values of service life and interest rate are arranged in an array of possible outcomes. There are 625 possible outcomes (25 x 25 = 625), each of which has a unique value of service life and interest rate. The EUAC for each possible outcome is computed, and various statistics (mean, standard deviation, high, and low) are computed based on these outcomes.

The user must be aware that there is <u>no one correct answer</u> to the EUAC comparison of alternatives. It is expected that the <u>actual</u> EUAC of a particular alternative will range between the high and low values as determined by the computer simulation. This range of possible outcomes must be considered in the comparison of possible alternatives.

3.4.4. Computer Spreadsheet for EUAC Comparison

A computer spreadsheet has been developed to simplify the calculations of EUAC for various bridge alternatives. In addition, this spreadsheet has been designed to allow the user to simulate the effect of various service lives and nominal interest rates on the EUAC of these alternatives. The EUAC spreadsheet (developed in Lotus 123 Release 2.3 (42)) is presented in Fig. 3.11. A brief general explanation of the use of a microcomputer spreadsheet is provided in Sec. 3.1.3.4.

If the user does not wish to utilize the simulation feature of the computer spreadsheet, simply enter a standard deviation of 0.00 for the service life (N or N') and interest rate (I or I'). In this case, the spreadsheet will only perform one iteration using the mean values of the service life and nominal interest rate. The output for the mean, standard deviation, high and low values of the EUAC will be equal to the EUAC of the particular alternative.

3.4.5. EUAC Example

To assist the reader in understanding the comparison of EUAC alternatives, a detailed example will be solved. This example will utilize hand solution techniques, but the use of the computer spreadsheet will be discussed where applicable. In the case of a computer simulation for EUAC, only one possible outcome will be analyzed by hand; the analysis procedure for other possible outcomes is similar.

The data required for the economic comparison of the alternatives can be acquired from several sources, such as: contractor estimates, historical price records, and the database of cost information presented in Sec. 3.7. It should be noted that, although this example problem is written for a rehabilitation project, the analysis procedure works equally well for bridge strengthening projects. The spreadsheet input and output for this example is shown in Fig. 3.11. For the replacement alternative, the following assumptions are made:

- Replacement structure first cost, R = \$60,000 **(Input A)**
- Net salvage value of existing structure, B = -\$5000 **(Input B)**
- Net salvage value of replacement structure, S = \$3000 **(Input C)**
- Annual maint. cost of replacement option, C_{ar} = \$4000 **(Input D)**
- Service life of replacement structure, N = 40 years **(Input E and F)**
- Single future expenditure, F_j = \$20,000 **(Input G)**
- Year of future disbursement, n_j = 20 **(Input H)**
- Nominal interest rate, I = 6.00% **(Input I and J)**

Assumptions for the rehabilitation alternative are as follows:

- Initial cost of rehabilitation, D = \$30,000 **(Input K)**
- Annual maintenance cost of rehabilitation alternative, C_{as} = \$5000 **(Input L)**
- Remaining life of existing structure, N' = 25 years **(Input M and N)**
- Single future expenditure, F_j = \$20,000 **(Input O)**
- Year of future disbursement, n_j = 15 **(Input P)**
- Nominal interest rate, I = 6.00% **(Input Q and R)**

To compute the EUAC of the replacement alternatives, Eqn. 1 should be used. For this example,

$$[\frac{A}{P}]_{40}^{6\%} = 0.0665 \qquad [\frac{A}{F}]_{40}^{6\%} = 0.00646 \qquad [\frac{P}{F}]_{20}^{6\%} = 0.3118$$

Cost-Effective Comparison of Alternatives

Press <ALT> A at any time to update the iterative module!

Replacement Alternative Input:

Replacement Structure First Cost, R =		$60,000	A
Net Salvage Value of EXISTING Structure, B =		($5,000)	B
Net Salvage Value of REPLACEMENT Structure, S =		$3,000	C
Annual Maintenance Cost of New Structure, Car =		$4,000	per year D
Service Life of Replacement Structure:			
Mean value =	40 years		E
Standard deviation =	0 years		F
Single Future Expenditure, Fj =		$20,000	G
Year of Future Expense (current year = 0), Nj =		20	H
Interest Rate, I:			
Mean value =	6.00%		I
Standard deviation =	0.00%		J

EUAC of Replacement Alternative:

Mean value of EUAC =	$8,715
Std. Dev. of EUAC =	$0
High value of EUAC =	$8,715
Low value of EUAC =	$8,715

Note: Numerical value in parenthesis for input B indicates negative number.

Fig. 3.11. Economic analysis spreadsheet, input parameters, and example problem.

Rehabilitation/Strengthening Alternative Input:

Initial Rehabilitation/Strengthening Cost, D =	$30,000	K
Annual Maintenance Cost After Rehab/Stren, Cas =	$5,000 per year	L
Remaining Service Life of Existing Structure, N':		
Mean value =	25	M
Standard deviation =	0	N
Single Future Expenditure, F'j =	$20,000	O
Year of Future Expenditure (current = 0), N'j =	15	P
Interest Rate, I:		
Mean value =	6.00%	Q
Standard deviation =	0.00%	R

EUAC of Rehab/Strengthening Alternative:

Mean value of EUAC =	$8,000
Std. Dev. of EUAC =	$0
High value of EUAC =	$8,000
Low value of EUAC =	$8,000

Fig. 3.11. Continued.

When these factors are included in Eqn. 1, the EUAC of the replacement alternative can be computed as:

$$EUAC_R = 0.0665[60{,}000-(-5000)] - 0.00646(3000) + 4000 + 0.0665[20{,}000(0.3118)] = \$8717$$

The EUAC for the rehabilitation/strengthening model can be computed from Eqn. 3. In this example,

$$[\frac{A}{P}]^{6\%}_{25} = 0.0782 \qquad [\frac{P}{F}]^{6\%}_{15} = 0.4173$$

When these factors are inserted into the Eqn. 3, the $EUAC_s$ can be computed as:

$$EUAC_s = 0.0782(30{,}000) + 5000 + 0.0782[20{,}000(0.04173)] = \$7999$$

For the above example, the EUAC of the replacement alternative was slightly more than the EUAC of the strengthening alternative. Thus, considering only EUAC's, one would select the rehabilitation/strengthening alternative.

The user must be aware that the actual difference between the two alternatives may be less than the error in estimation of one or more of the terms in the equations. Sound engineering judgement must be used in the interpretation of the results.

3.5. Strengthening Techniques for Steel Stringer Bridges (FHWA 302)

3.5.1. Replacement of Damaged Stringers

A bridge's load carrying capacity can be increased by replacing damaged or deteriorating stringers. It may not be necessary to remove the deck even if the deck and stringers are partially or fully composite. A procedure for such a replacement is described in the NCHRP 222 report (84). Traffic should be detoured to allow jacking of the bridge to provide clearance between the beam and the end supports. The web of the damaged beam should then be cut at the junction of the top flange and web (see Fig. 3.12). The exposed face of the flange, which remains in the concrete, needs to be ground flat. The width of the top flange of the replacement beam should be slightly less than that of the original beam to facilitate field welding. Using continuous fillet welds, the new steel beam is connected to the top flange of the original beam. If required, cover plates can be welded (preferably, shop welded rather than field welded) to the bottom flange of the new stringer prior to field installations to lower the neutral axis thus reducing lower flange stresses. In situations where it is necessary to replace a non-composite stringer with a composite stringer, the previously described procedure is not applicable. In these situations, a steel stringer can be added and made to act compositely with the concrete deck without removing a portion of the deck by coring through the existing deck and adding shear connectors. Pressure grouting through either the cored holes in the deck or from below can be used to fill the voids between the deck and stringers.

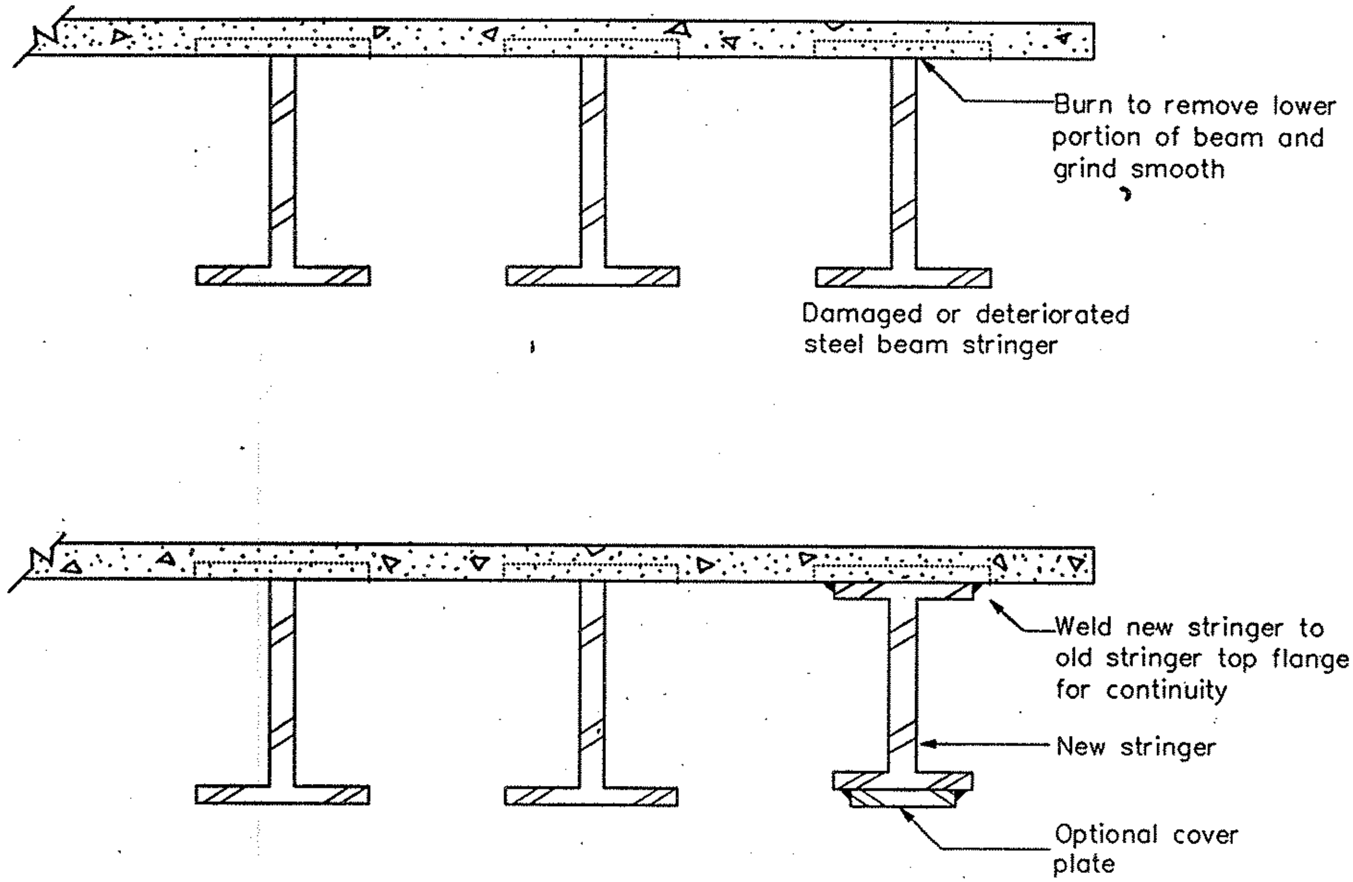


Fig. 3.12. Steel beam replacement.

The Iowa DOT V9, V11 and V13 standards for steel stringer bridges specified smaller, lower capacity exterior stringers than those used for the interior stringers. Replacement of these external stringers with stringers the same size as the internal stringers (or slightly larger) can provide additional strength for the existing bridge. However, more cost effective procedures for strengthening these bridges will be presented in the following sections.

3.5.2. Respacing Existing Stringers and Adding New Stringers

3.5.2.1. Background

Generally, the respacing of existing stringers and adding new stringers (henceforth referred to as respace and add) is implemented to increase the flexural strength of the bridge by reducing the spacing between the stringers. For most applications, removal of the existing deck is required. In these cases, the live load capacity of the bridge can be further increased by replacing the original deck with a lightweight deck. In some cases, the additional stringers can be added to allow for widening of the original bridge, assuming the original abutments/piers are of sufficient width. In the respace and add procedure, it is important to consider the two items: 1) to minimize differential deflection between the new stringers and the existing stringers, and to ensure all stringers carry similar loads, the new stringers should have stiffnesses (i.e. moment of inertias) similar to the existing stringers, 2) analysis of bridges with nonuniform stringer spacing most likely will require more involved analysis, such as finite element analysis, thus stringer spacing should be kept uniform if at all possible.

The evaluation procedure required for strengthening steel stringer bridges using the respace and add method is described in the following paragraphs. The procedure has been simplified by using a computer spreadsheet. The following example problem (Sec. 3.5.2.5) has been solved by using hand calculations and by using the computer spreadsheet developed.

3.5.2.2. Design Criteria

Iowa agencies must currently conform to the *AASHTO Standard Specifications for Highway Bridges (2).* The procedure presented in this section, as well as following sections, is based on these specifications.

3.5.2.3. Design Limitations

The bridge strengthening evaluation only investigates the bridge superstructure. Although not included in this manual, the substructure should also be investigated before any rehabilitation is undertaken. The substructure should be inspected for signs of scour, decay and damage. If the load carrying capacity of the superstructure is increased, the capacity of the substructure should be reviewed to determine if it is adequate for the increased loading; in some situations, the substructure may also require rehabilitation. As a minimum, the following items associated with the substructure should be reviewed:

- Pile axial load capacity,
- Bent/pier capacity for overturning forces, and
- Capacity of abutments, bents and piers.

The following limitations apply to steel stringer bridges evaluated by the procedure which follows:

- All stringers are assumed to have similar material and section properties.
- The superstructure has no or minimal skew.
- The deck section and material properties are homogeneous.
- Bridge stringers are assumed to be simply supported.

3.5.2.4. Design Procedure

1. **Determine the allowable and operating steel stress levels.** (G and H in the spreadsheet.) Frequently, bridge plans documenting the type of steel in the stringers can not be found. In these cases, the Iowa DOT recommends allowable and operating steel stress levels based on the year the bridge was constructed (see Table 3.3).

Table 3.3. Iowa DOT steel stresses based on the year the bridge was constructed.

Year Constructed	Allowable Stress (G) (ksi)	Operating Stress (H) (ksi)
Before 1905	14.30	19.50
1906 to 1936	16.50	22.50
1937 to 1962	18.15	24.75
1963 to 1999	20.00	27.00

2. **Determine the moment capacity of the interior stringers.** (I) This evaluation determines the moment capacity of the interior stringers based on the allowable stresses in Table 3.3 and the section modulus of the interior stringers.

Stringer Moment Capacity = Section Modulus x Allowable Stress

$$M_{capacity} = S_{modulus} \times \sigma_{all}$$

3. **Determine the dead load moment capacity of the superstructure.** (J) Dead load includes all permanent loads associated with the superstructure and roadway, including stringers, diaphragms, deck, wearing surface, fill on gravel roads, railings, sidewalks, barriers, lighting, utility lines carried by the superstructure, etc. The dead load bending moment a typical interior stringer must support based on beam theory is:

Applied Dead Load Moment = (Uniform Dead Load x Bridge Length²)/8

$$M_{DL} = (\omega_{DL} \times L^2)/8$$

4. **Determine the live load moment capacity of the superstructure.** (K) The live load the bridge must

withstand is based on Iowa legal truck loads. The live load moment capacity of the bridge is the difference between the stringer moment capacity and the applied dead load moment.

Live Load Moment Capacity =

Stringer Moment Capacity - Applied Dead Load Moment

$$M_{LL\ capacity} = M_{capacity} - M_{DL}$$

5. **Determine the AASHTO live load distribution factor. (L & M)** As previously noted, bridge loads must be appropriately distributed to a single longitudinal stringer for analysis. AASHTO wheel load distribution factors for stringers are based on the number of traffic lanes, deck material and stringer spacing. (See Table 3.23.1 in AASHTO.) Since timber plank and concrete decks are the prevalent types of decks on the deficient bridges identified in the Iowa SI&A survey, they were both considered in the evaluation spreadsheet. AASHTO wheel load distribution factors for steel stringers are given in Table 3.4.

Table 3.4. AASHTO wheel load distribution factors - steel stringers.

Deck Type	One Lane (M)	Two Lane (L)
Timber Plank	S/4.5 for 4 in. deck (1) S/5.25 for 6 in. deck	S/4.0 for 4 in. deck (2)
Concrete	S/7.0 (3)	S/5.5 (4)

S = average stringer spacing in ft.
(1) If the spacing, S, exceeds 5.5 ft, use the reaction of the wheel loads.
(2) If the spacing, S, exceeds 7.0 ft, use the reaction of the wheel loads.
(3) If the spacing, S, exceeds 10.0 ft, use the reaction of the wheel loads.
(4) If the spacing, S, exceeds 14.0 ft, use the reaction of the wheel loads.

6. **Determine the AASHTO impact factor. (N)** Impact factors account for the fact that loads are not applied statically. AASHTO impact is a fraction of the live load stress. (See Section 3.8.2 of AASHTO).

Impact = 50/(Bridge Length (ft) + 125)

$$I = 50/(L + 125)$$

This calculated value shall not exceed 30 percent.

7. **Determine the maximum truck load moment by hand calculations or from applicable tables. (O)** The truck live loads moments including impact are determined by:

$$M_{truck\ x\ 1} = M_{truck}\ x\ I\ x\ 2\ x\ Dist._{LL}$$

where:

- $M_{truck\ x\ 1}$ = live load moments with impact factor (in units of foot-kips per wheel line). See Appendix B for Iowa truck live load moments.
- 2 is for 2 wheel lines
- $Dist._{LL}$ = live load distribution factor.

8. **Determine Iowa DOT legal truck load posting values from the operating rating. (P)** The live load demand (based on the operating stress) is compared with the live load moment capacity to determine if the bridge requires posting. (See Appendix A for truck weights.)

$$Operating\ Rating = (M_{LL\ capacity}/M_{LL\ demand})\ x\ Truck\ Weight$$

The maximum operating rating loads for which a bridge can be posted are presented in Table 3.5.

Table 3.5. Maximum operating rating loads.

Truck	Maximum Load (Tons)
Type 3	25.0
Type 3S2(a)	36.5
Type 3S2(b)	40.0
Type 4	27.5
Type 3S3	40.0
Type 3-3	40.0

9. **Determine the HS inventory rating (for use in calculating the SI&A sufficiency rating). (Q)** The inventory rating used in the appraisal sheet can also be calculated to determine the effect of the proposed strengthening method on the SI&A sufficiency rating. The inventory rating differs from the operating rating only by the stress used. The NBI coding guide (25) provides the factors included in the SI&A sufficiency rating. Briefly stated, the sufficiency rating = S1 + S2 + S3 + S4. The S1 factor refers to the structural adequacy and safety; S2 to serviceability and functional obsolescence; S3 to essentially for the public use; and S4 to special reductions which include detour length, traffic safety features and structure type. Each of these four factors is a function of coded items included on the appraisal sheet.

3.5.2.5. Evaluation of Existing Stringers Example

The following steel stringer evaluation is performed for a 20 ft wide and 20 ft long non-composite steel stringer bridge. The bridge has five stringers spaced at 3.8 ft. This example follows the procedure outlined in the previous section. Input, as well as the majority of the calculated intermediate values have been identified by letters shown next to the spreadsheet cells (year built **(A)**; length **(B)**; etc.). The spreadsheet input and output (highlighted on the spreadsheet) for this example is shown in Fig. 3.13.

1. **Determine the allowable and operating steel stress levels.** The example bridge (20 ft long **(B)**) was built in 1955 **(A)**. Therefore, as presented in Table 3.3, the allowable steel inventory stress is 18,150 psi **(G)** and the operating/posting stress is 24,750 psi **(H)**.

2. **Determine the moment capacity of the interior stringers.** The stringers in this example are steel S15x50 sections with a section modulus of 64.8 in^3 **(D)**.

Stringer Inventory Moment Capacity =

$$\left(\frac{64.8\ in^3}{12\ in./ft}\right) x\ (18.15\ ksi) = 98.01\ ft\text{-}k \qquad (I)$$

Stringer Operating Moment Capacity =

$$\left(\frac{64.8\ in^3}{12\ in/ft}\right) x\ (24.75\ ksi) = 133.65\ ft\text{-}k \qquad (I)$$

3. **Determine the dead load moment capacity of the superstructure.** The weight of the stringers, deck, wearing surface and barrier is assumed to be 0.4 klf. The applied bending moment on a typical internal stringer is:

$$M_{DL} = (\omega_{DL}\ x\ L^2)/8$$

Applied Dead Load Moment =

$$= \frac{(0.4\ klf)\ \ (20\ ft)^2}{8} = 20\ ft\text{-}k \qquad (J)$$

4. **Determine the live load moment capacity of the superstructure.**

$$M_{LL\ capacity} = M_{capacity} - M_{DL}$$

Inventory Live Load Moment Capacity = 98.01 - 20.0 = 78.01 ft-k **(K)**

STEEL STRINGER EVALUATION SPREADSHEET

INPUT		
YEAR BUILT	1955	A
BRIDGE LENGTH (FEET)	20.0	B
BEAM SPACING (FEET)	3.8	C
SECTION MODULUS (INCH ^ 3)	64.8	D
DEAD LOAD (KLF)	0.4	E
FLOOR TYPE FROM BELOW	3	F
1 = 4" THICK TIMBER		
2 = 6" OR MORE THICK TIMBER		
3 = CONCRETE		

OUTPUT		
LOAD TO POST ON LOAD LIMIT SIGN		
TRUCK	2 LANES	1 LANE
HS20	15.82	NO POST
TYPE 3	22.97	NO POST
TYPE 3S2(a)	NO POST	32.07
TYPE 3S2(b)	36.76	29.24
TYPE 4	22.39	26.14
TYPE 3S3	32.87	26.14
TYPE 3-3	NO POST	35.35

INTERMEDIATE VALUES			
ALLOWABLE STRESS (KSI)	18.15	G	
OPERATING STRESS (KSI)	24.75	H	
MOMENT CAPACITY (KIP-FT)	INVENTORY	OPERATING	
TOTAL STRUCTURE	98.01	133.65	I
DEAD LOAD	20.00	20.00	J
LIVE LOAD	78.01	113.65	K
LIVE LOAD DISTRIBUTION FACTORS			
TWO LANES OR MORE	0.6909	L	
ONE LANE	0.5429	M	
IMPACT FACTOR	1.3	N	

LIVE LOAD + IMPACT MOMENT		O
TRUCK	2 LANES	1 LANE
HS20	143.71	112.91
TYPE 3	123.67	97.17
TYPE 3S2(a)	112.77	88.61
TYPE 3S2(b)	123.67	97.17
TYPE 4	138.32	108.68
TYPE 3S3	138.32	108.68
TYPE 3-3	102.28	80.36

BRIDGE LOAD IN TONS			P
TRUCK	2 LANES	1 LANE	MAX LOAD
HS20	15.82	20.13	20.00
TYPE 3	22.97	29.24	25.00
TYPE 3S2(a)	36.78	32.07	36.50
TYPE 3S2(b)	36.76	29.24	40.00
TYPE 4	22.39	26.14	27.25
TYPE 3S3	32.87	26.14	40.00
TYPE 3-3	44.45	35.35	40.00

MAX S.I. & A. RATING		Q
TRUCK	2 LANES	1 LANE
HS20	10.86	13.82
BEST S.I. & A. SUFFICENCY RATING	64.98	70.98

Fig. 3.13. Spreadsheet for steel beam replacement example.

$$Operating\ Live\ Load\ Moment\ Capacity = 133.65 - 20.0 = 113.65\ ft\text{-}k \qquad (K)$$

5. **Determine the AASHTO live load distribution factor.** The deck in this example is reinforced concrete with two lanes of traffic.

$$L.L.\ Distribution\ Factor = \frac{3.8\ ft}{5.5} = 0.6909 \qquad (L)$$

6. **Determine the AASHTO impact factor.** The bridge length is 20 ft.

$$I = 50/(L + 125)$$

$$I = 50/(20 + 125) = 34\% \ >30\% \qquad (N)$$

This calculated impact is greater than the 30 percent limit; therefore, 30 percent impact controls.

7. **Determine the maximum truck load moment by hand calculations or from applicable tables.** See Appendix B for the truck live load moments.

$$M_{truckxI} = M_{truck} \ x \ I \ x \ 2 \ x \ Dist._{LL} \qquad (O)$$

- *HS20: 104.0 ft-k x 2 x 0.6909 = 143.71 ft-k*
- *Type 3: 89.5 ft-k x 2 x 0.6909 = 123.67 ft-k*
- *Type 3S2(A): 81.6 ft-k x 2 x 0.6909 = 112.77 ft-k*
- *Type 3S2(B): 89.5 ft-k x 2 x 0.6909 = 123.67 ft-k*
- *Type 4: 100.1 ft-k x 2 x 0.6909 = 138.32 ft-k*
- *Type 3S3: 100.1 ft-k x 2 x 0.6909 = 138.32 ft-k*
- *Type 3-3: 74.0 ft-k x 2 x 0.6909 = 102.28 ft-k*

8. **Determine the Iowa DOT legal truck load posting values from the operating rating.** The live load demand (based on the operating stress) is compared with the live load capacity to determine the load values for posting. (See Table 3.5 for the maximum truck weights and Appendix A for truck configurations.)

$$Operating\ Rating = (M_{LL\ capacity}/M_{LL\ demand}) \ x \ Truck\ Weight \qquad (P)$$

- *Type 3: (113.65 ft-k/123.67 ft-k) x 25 ton = 22.97 ton < 25 ton*
- *Type 3S2(A): (113.65 ft-k/112.77 ft-k) x 36.5 ton = 36.77 ton > 36.5 ton*

- *Type 3S2(B): (113.65 ft-k/123.67 ft-k) x 40 ton = 36.76 ton < 40 ton*
- *Type 4: (113.6 ft-k/138.3 ft-k) x 27.25 ton = 22.39 ton < 27.5 ton*
- *Type 3S3: (113.6 ft-k/138.3 ft-k) x 40 ton = 32.85 ton < 40 ton*
- *Type 3-3: (113.6 ft-k/102.3 ft-k) x 40 ton = 44.43 ton > 40 ton*

Required posting for this bridge would be a combination of the following Iowa legal truck types: Type 3 at 23 tons, Type 3S2(B) at 36 tons, Type 4 at 22 tons and Type 3S3 at 32 tons, as is shown in the output section of Fig. 3.13. The Iowa DOT guidelines for posting non-interstate highways state the posted load limit for a straight truck (see Appendix A for truck configurations) may be based on the Type 4 vehicle. The posted load limit for the semi-trailer combination and the truck plus full trailer may be based upon, respectively, the Type 3S3 and Type 3-3 vehicles. Other suitable posting schemes, including those utilizing a triple axle limit sign, may be used in lieu of the described method when appropriate. The posted load limit for the triple axle, if such posting is used, shall be based upon the load rating for the Type 4 and Type 3S3 vehicles. The maximum posting weight for the triple axle is 21 tons.

9. **Determine the HS inventory rating (for use in calculating the SA&I sufficiency rating).**

HS20 Inventory Rating (tons) =

$$\left(\frac{\textit{Inventory live load moment capacity}}{\textit{Truck live load + impact moment}}\right) x\ (\textit{Weight of truck})$$

$$= \left(\frac{78.01\ ft\text{-}k}{143.71\ ft\text{-}k}\right) x\ (20\ ton) = 10.86\ ton \qquad (Q)$$

The NBI coding guide outlines in detail the factors included in the SI&A sufficiency rating. For comparison purposes, a sufficiency rating could be obtained for this bridge if all factors except the HS truck loadings are assumed to be "perfect":

- Adjusted inventory tonnage (AIT) for this HS20 truck =
  1.00 x 10.86 = 10.86
- I = $(36 - AIT)^{1.5}$ x 0.2778 = $(36 - 10.86)^{1.5}$ x .2778 = 35.01
- SI = 55 - (A + B + C + D + E + F + G + H + I)
  (Assume no reductions for A through H.)
  SI = 55 - 35.02 = 19.98
- Sufficiency rating = S1 + S2 + S3 + S4
  (Assume S2, S3, and S4 are "perfect")
  Sufficiency rating = 19.98 + 30 + 15 = 64.98.

Note that it is highly unlikely that "perfect" factors actually exist. Therefore, this is a hypothetical SI&A sufficiency rating for this bridge.

3.5.2.6. Respace and Add Stringer Example

To demonstrate how respacing and adding stringers increases a given bridge's capacity, the same bridge is re-evaluated with several different stringer spacings (and number of stringers). Shown in Fig. 3.14 is the original stringer spacing (Fig. 3.14a) and three additional cases that were investigated (Fig. 3.14b, c and d). Note in this example, two assumptions were made:

- Added stringers are the same size as the stringers in the original bridge.
- Exterior stringers are kept in their original location (i.e., the distance between exterior stringers is 15.2 ft). If the supports are of adequate width, in some instances, it may be desireable to reposition the exterior stringers also.

By adding one stringer and reducing the spacing between stringers from 3.8 ft (Case I) to 3.04 ft (Case II) the given bridge no longer requires posting. By changing the spacing and using seven stringers (Case III) or nine stringers (Case IV) the given bridge also would not require posting. One advantage to Case IV (nine stringers) is that the original five stringers would not have to be moved and the added four stringers could be placed midway between the original stringers. Also there would be a significant improvement in the SI & A sufficiency rating.

As was previously noted, the bridge with the original stringer spacing (Case I) had an SI&A sufficiency rating of 64.98. By adding stringers and thus reducing stringer spacings, the SI&A sufficiency rating improves by 8.5 percent, 16.5 percent and 30.8 percent for Cases II, III and IV respectively.

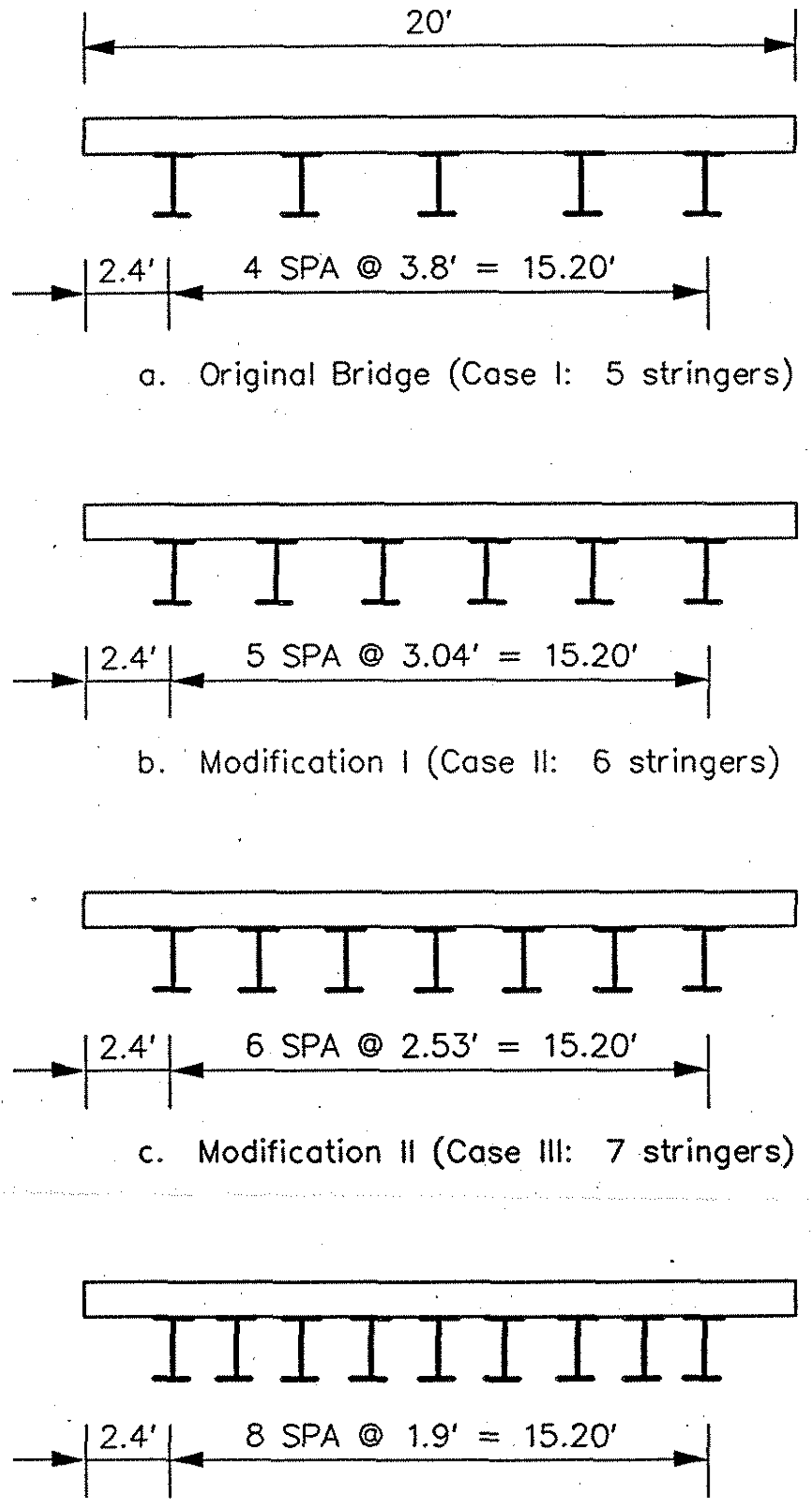
3.5.3. Increase Section Modulus

3.5.3.1. Background

Stringer section modulus may be increased by the attachment of coverplates, angles, or tee sections thus increasing the load carrying capacity of the bridge. The added material must be bolted (preferred) or welded to the existing stringers so that it acts compositely with the original stringers. Appropriate manuals should be referenced for welding criteria. The main advantage of this strengthening technique is that it is easily implemented when compared to several of the other strengthening methods. Often county maintenance crews and equipment can be used to attach the additional steel.

To optimize the benefits of this procedure, the original member should be jacked up prior to attaching the new steel; appropriate traffic control needs to be established during this procedure. The ojective of jacking the member is to relieve dead load stresses. Once the additional steel is connected and the member is released from the jacked position, the strengthened member will carry a portion of the dead load stresses as well as the live load stresses. If it is not practical to relieve dead load from the member (that is, it is not possible to jack the member) live load stresses will still be reduced by this method.

The Iowa DOT has implemented this procedure by attaching angles to the web of existing steel sections (36). One concern with this procedure is that maintenance becomes increasingly difficult as the distance



a. Original Bridge (Case I: 5 stringers)

b. Modification I (Case II: 6 stringers)

c. Modification II (Case III: 7 stringers)

d. Modification III (Case IV: 9 stringers)

Fig. 3.14. Example bridge: reduction of stringer spacing.

between the angle and the lower flange of the stringers decreases. Possible corrosion from improper maintenance of this region could cause considerable damage. If clearance under the bridge is not a limiting factor, T-sections can be attached to the bottom of the existing stringer (see Fig. 3.15). This solution has the advantage that diaphragms between stringers need not be modified as would be necessary when attaching angles directly to the web.

3.5.3.2. Increased Section Modulus Example

The inadequate steel stringer bridge used in the first example is reanalyzed considering an increased section modulus. The original bridge contained S15x50 stringers (section modulus equal to 64.8 in^3.) If 2 L2x2x0.25 are used to attach a WT5x6 to the original stringers, a section modulus of 138.83 in^3 is achieved **(D)**. The spreadsheet in Fig. 3.16 shows the evaluation of this strengthened bridge. This solution also increased the strength of the bridge so that posting is not required. Note also by adding the additional steel to each stringer, the SI&A sufficiency rating increased slightly more than 41 percent (from 64.98 to 91.78) **(Q)**.

3.5.4. Develop Composite Action

Developing composite action between the stringers and deck is another way to strengthen a noncomposite bridge. By having the deck act compositely with the stringer, an increased moment of inertia (or section modulus) is obtained. The increased section modulus, as previously discussed, increases the bridge's flexural strength and reduces live load stresses.

This procedure may be useful for those bridges which currently are considered only partially composite and are inadequate for today's increased allowable live loads. The current AASHTO manual gives ultimate strength equations for welded studs and channels. Strength of older shear connectors can be found in older AASHTO specifications and Refs. 15 and 36. Klaiber et. al. (37), Dallam (19,20) and Dedic et. al. (21) have shown that the strength and stiffness of high-strength bolts is comparable to that of welded shear studs. Therefore, the existing AASHTO ultimate strength formulas for welded stud connectors can be used conservatively to estimate the ultimate capacity of high-strength bolts.

According to the current AASHTO manual, shear connectors in new bridges should be designed for fatigue and checked for ultimate strength. In older bridges, however, the remaining fatigue life of the bridge will be considerably less than that of the new shear connectors; thus it is only necessary to design the new shear connectors for ultimate strength. If an existing composite bridge requires additional shear connectors, new shear connectors can be added even though they are not the same as the original connectors. Variation in the stiffness of the new shear connectors and original shear connectors will have essentially no effect on the bridge's elastic behavior and minimal effect on the ultimate strength.

Although concrete decks are most commonly considered for this method, composite action can be developed for various types of decks including precast concrete, cast in place concrete, laminated timber and steel grid decks filled with concrete.

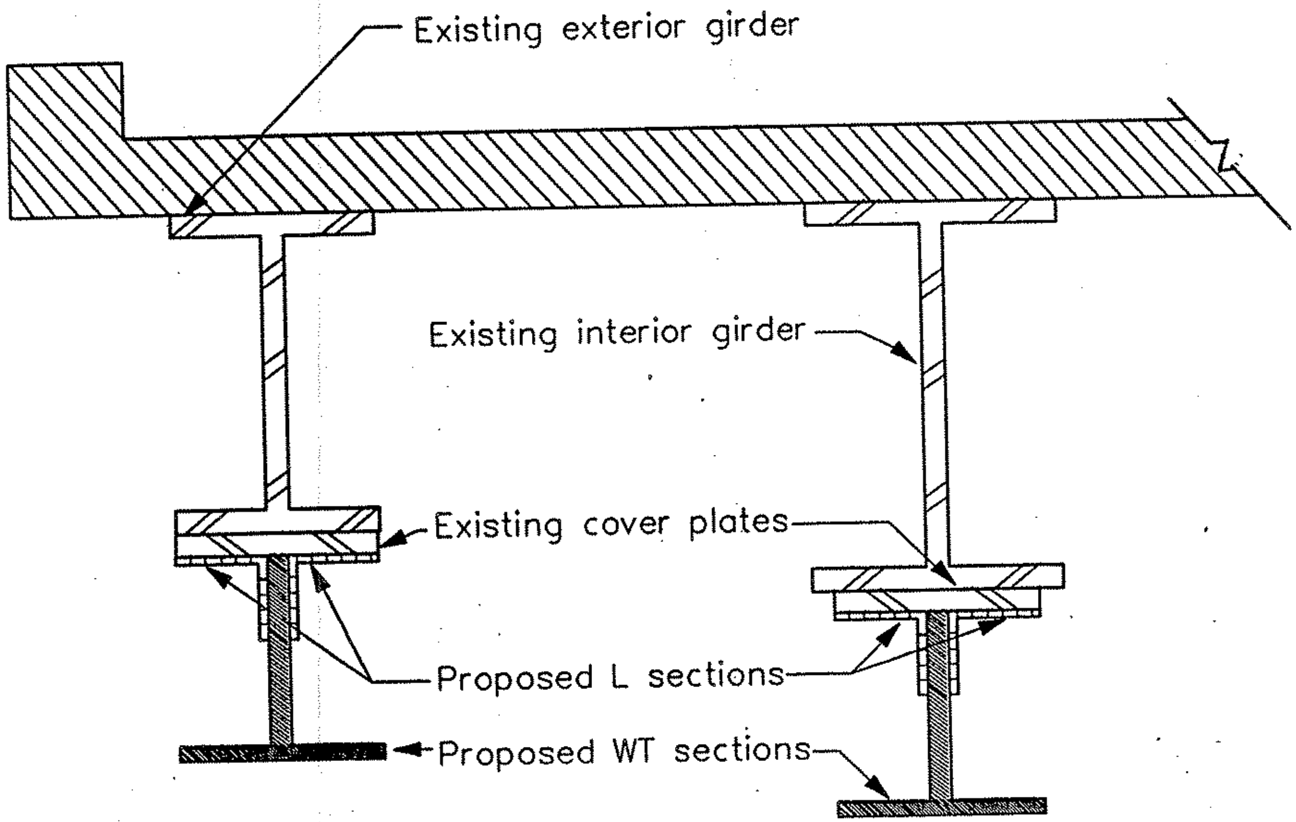


Fig. 3.15. Addition of L and WT sections.

STEEL STRINGER EVALUATION SPREADSHEET

INPUT	
YEAR BUILT	1955
BRIDGE LENGTH (FEET)	20.0
BEAM SPACING (FEET)	3.8
SECTION MODULUS (INCH ^ 3)	138.8 D
DEAD LOAD (KLF)	0.4
FLOOR TYPE FROM BELOW	3
1 = 4" THICK TIMBER	
2 = 6" OR MORE THICK TIMBER	
3 = CONCRETE	

OUTPUT		
LOAD TO POST ON LOAD LIMIT SIGN		
TRUCK	2 LANES	1 LANE
HS20	NO POST	NO POST
TYPE 3	NO POST	NO POST
TYPE 3S2(a)	NO POST	NO POST
TYPE 3S2(b)	NO POST	NO POST
TYPE 4	NO POST	NO POST
TYPE 3S3	NO POST	NO POST
TYPE 3-3	NO POST	NO POST

INTERMEDIATE VALUES		
ALLOWABLE STRESS (KSI)	18.15	
OPERATING STRESS (KSI)	24.75	
MOMENT CAPACITY (KIP-FT)	INVENTORY	OPERATING
TOTAL STRUCTURE	209.94	286.28
DEAD LOAD	20.00	20.00
LIVE LOAD	189.94	266.28
LIVE LOAD DISTRIBUTION FACTORS		
TWO LANES OR MORE	0.6909	
ONE LANE	0.5429	
IMPACT FACTOR	1.3	

LIVE LOAD + IMPACT MOMENT		
TRUCK	2 LANES	1 LANE
HS20	143.71	112.91
TYPE 3	123.67	97.17
TYPE 3S2(a)	112.77	88.61
TYPE 3S2(b)	123.67	97.17
TYPE 4	138.32	108.68
TYPE 3S3	138.32	108.68
TYPE 3-3	102.28	80.36

BRIDGE LOAD IN TONS			
TRUCK	2 LANES	1 LANE	MAX LOAD
HS20	37.06	47.16	20.00
TYPE 3	53.83	68.51	25.00
TYPE 3S2(a)	86.18	75.13	36.50
TYPE 3S2(b)	86.12	68.51	40.00
TYPE 4	52.46	61.25	27.25
TYPE 3S3	77.00	61.25	40.00
TYPE 3-3	104.13	82.83	40.00

MAX S.I. & A. RATING		Q
TRUCK	2 LANES	1 LANE
HS20	26.43	33.64
BEST S.I. & A. SUFFICENCY RATING	91.78	98.99

Fig. 3.16. Spreadsheet for increased section modulus example.

3.5.5. Deck Replacement

One simple procedure for increasing a bridge's live load capacity is to decrease its dead load. This method is especially efficient for bridges with poor decks which need replacing. Dead load can further be reduced by replacing the existing guardrail system with a lighter weight guardrail. This technique is also useful when used in combination with other strengthening methods such as increasing stringer section modulus and respacing. Lightweight deck types and weights are shown in Table 3.6.

Table 3.6. Lightweight deck types.

Lightweight Deck Type	Weight (psf)
Open Grid Steel Deck	15-25
Concrete Filled Steel Grid Deck	46-81
Exodermic Deck	40-60
Laminated Timber Deck	10-23
Aluminum Orthotropic Plate Deck	20-25
Orthotropic Steel Deck	45-130
Lightweight Concrete	100-120

Design criteria (such as distribution factors, corrosion protection, etc.) varies with the various lightweight decks and thus must be appropriately taken into account. Costs can vary from $9 per square ft to $35 per square ft. A design procedure and design aids which compare various lightweight deck alternatives, span lengths and increases in live load capacity are provided in the NCHRP 293 report (36).

3.5.6. Post-Tensioning

3.5.6.1. Background

Longitudinal post-tensioning of steel stringers is another procedure for increasing the load carrying capacity of steel stringer bridges. Post-tensioning can easily modify the elastic stresses within a given bridge and, therefore, satisfy rating criteria for service loads. A large number of Iowa steel-stringer composite concrete deck bridges designed and constructed prior to 1957 are understrength because of excessive flexural stresses in the exterior stringers. Bridge design standards used during that time period permitted exterior stringers to be designed for a wheel-load fraction considerably smaller than the fraction for interior stringers. Current design standards have increased the wheel-load-distribution fraction for exterior stringers for this bridge type by as much as 40 percent in some situations. This in addition to significantly increased in state legal loads have caused the overstress problems.

Through research sponsored by the Iowa DOT Highway Division (22,23,35,38), ISU has developed a design manual for use in designing post-tensioning systems for the bridges in question. For the design procedure developing a post-tensioning strengthening system for a given bridge, the reader is referred to p 24, Sec. 3.4 of Ref. 24. In reviewing the post-tensioning strengthening scheme one quickly realizes the most time consuming part of the process is determining the moment fractions (MF) and force fractions (FF). As the strengthening scheme presented in Ref. 24 only requires the post-tensioning of the exterior stringers, one needs to know the magnitude of the post-tensioning moment and post-tensioning force that remains on the exterior stringers and which is distributed to the interior stringers.

3.5.6.2. Post-tensioning example

Shown in Fig. 3.17 is a spreadsheet which may be used for determining the FF and MF for a four-stringer bridge. As the actual procedure has been detailed in Ref. 24, only the required input will be presented in the following paragraphs:

Input A: Length of bridge between centerlines of bearing, ft

Input B: Distance between stringers, ft

Input C: Distance from edge of bridge to centerline of exterior stringer, ft

Input D: Dead load on exterior stringer, klf

Input E: Dead load on interior stringer, klf

Input F: Long-term dead load on exterior stringer, klf

Input G: Long-term dead load on interior stringer, klf

Input H: Distance from centerline of bearing to coverplate cutoff for exterior stringer, ft

Input I: Distance from centerline of bearing to coverplate cutoff for interior stringer, ft

Input J: Distance from centerline of bearing to anchorage-Assumption 1, ft

Input K: Distance from centerline of bearing to anchorage-Assumption 2, ft

Input L: Bridge deck thickness, in.

Input M: Area of exterior stringer, $in.^2$

Input N: Distance from the bottom of the exterior stringer to the centroid of the exterior stringer, in.

Input O: Moment of inertia of exterior stringer, $in.^4$

Input P: Width of coverplate on exterior stringer, in.

Input Q: Thickness of coverplate on exterior stringer, in.

Input R: Width of Part 1 of the curb, in.

Input S: Height of Part 1 of the curb, in.

Input T: Width of Part 2 of the curb, in.

Input U: Height of Part 2 of the curb, in.

Input V: Area of interior stringer, $in.^2$

Input W: Distance from bottom of the exterior stringer to the centroid of the interior stringer, in.

	A	B	C
1	**POST-TENSIONING SPREADSHEET**		
2	INPUT		
3	LENGTH (ft.)		A
4	AVE SPACING (ft.)		B
5	1st SPACING (ft.)		C
6	DEAD LOAD - EXT BEAM (klf)		D
7	DEAD LOAD - INT BEAM (klf)		E
8	LONG TERM D.L. - EXT BEAM (klf)		F
9	LONG TERM D.L. - INT BEAM (klf)		G
10	D TO COVERPLATE - EXT (ft.)		H
11	D TO COVERPLATE - INT (ft.)		I
12	ANCHORAGE TO EXT BEAM (ft.)		J
13	ANCHORAGE TO EXT BEAM (ft.)		K
14	SLAB THICKNESS (in.)		L
15	EXTERIOR BEAM		
16	AREA (in ^ 2)		M
17	z (in.)		N
18	MOMENT OF INERTIA (in ^ 4)		O
19	EXTERIOR COVERPLATE		
20	WIDTH (in.)		P
21	HEIGHT (in.)		Q
22	CURB 1		
23	WIDTH (in.)		R
24	HEIGHT (in.)		S
25	CURB 2		
26	WIDTH (in.)		T
27	HEIGHT (in.)		U
28	INTERIOR BEAM		
29	AREA (in ^ 2)		V
30	z (in.)		W
31	MOMENT OF INERTIA (in ^ 4)		X
32	INTERIOR COVERPLATE		
33	WIDTH (in.)		Y
34	HEIGHT (in.)		Z
35	DECK		
36	WIDTH INTERIOR (in.)		AA
37	WIDTH EXTERIOR (in.)		BB
38	HEIGHT (in.)		CC
39	MODULAR RATIO OF ELASTICITY, n		DD
40	YEAR BUILT		EE
41	ECCENTRICITY		FF
42			

Fig. 3.17. Spreadsheet for determining force fractions and moment fractions.

43		
44		
45	IMPACT FACTOR	
46	COMPUTED EXT BEAM LOAD FRACTION	
47	AASHTO EXT BEAM LOAD FRACTION	
48	EXTERIOR LOAD FRACTION USED	
49	INTERIOR LOAD FRACTION	
50		
51	MIDSPAN - EXTERIOR BEAM y =	
52	DEAD LOAD MOMENT	
53	LONG TERM D.L. MOMENT	
54	HS20 TRUCK LOAD MOMENT	GG
55	LIVE + IMPACT LOAD MOMENT	
56	MIDSPAN - INTERIOR BEAM y =	
57	DEAD LOAD MOMENT	
58	LONG TERM D.L. MOMENT	
59	HS20 TRUCK LOAD MOMENT	
60	LIVE + IMPACT LOAD MOMENT	
61	COVERPLATE CUTOFF - EXT BEAM y =	
62	DEAD LOAD MOMENT	
63	LONG TERM D.L. MOMENT	
64	HS20 TRUCK LOAD MOMENT	
65	LIVE + IMPACT LOAD MOMENT	
66	COVERPLATE CUTOFF - INT BEAM y =	
67	DEAD LOAD MOMENT	
68	LONG TERM D.L. MOMENT	
69	HS20 TRUCK LOAD MOMENT	
70	LIVE + IMPACT LOAD MOMENT	
71	ANCHORAGE - EXT BEAM y =	
72	DEAD LOAD MOMENT	
73	LONG TERM D.L. MOMENT	
74	HS20 TRUCK LOAD MOMENT	
75	LIVE + IMPACT LOAD MOMENT	
76	ANCHORAGE - EXT BEAM y =	
77	DEAD LOAD MOMENT	
78	LONG TERM D.L. MOMENT	
79	HS20 TRUCK LOAD MOMENT	
80	LIVE + IMPACT LOAD MOMENT	
81	--	

Fig. 3.17. Continued.

SECTION PROPERTIES

EXTERIOR BM FLANGE WIDTH

BASIC QUANTITIES

ITEM		AREA
BEAM		
COVERPLATE		
DECK	n =	
DECK	n =	
CURB 1	n =	
CURB 1	n =	
CURB 2	n =	
CURB 2	n =	

CENTROID ELEVATIONS AND MOMENT OF INERTIA

DESCRIPTION
STEEL BEAM
STEEL BEAM WITH COVERPLATE
COMPOS BM, DECK AND CURB n=
COMPOS BM, DECK, CURB & COVRPLn=
COMPOS BM, DECK AND CURB n=
COMPOS BM, DECK, CURB & COVRPLn=

INTERIOR BM FLANGE WIDTH

BASIC QUANTITIES

ITEM		AREA
BEAM		
COVERPLATE		
DECK	n =	
DECK	n =	

CENTROID ELEVATIONS AND MOMENT OF INERTIA

DESCRIPTION	
STEEL BEAM	
STEEL BEAM WITH COVERPLATE	
COMPOS BM AND DECK	n=
COMPOS BM, DECK & COVRPL	n=
COMPOS BM AND DECK	n=
COMPOS BM, DECK & COVRPL	n=

COMPOSITE ELEVATION AND MOMENT OF INERTIA W/R/

n = z

COVERPLATES ON ALL BEAMS

COVERPL ON INT BEAM ONLY

NO COVERPL

Fig. 3.17. Continued.

137	
138	POST TENSIONING DESIGN
139	EXTERIOR BEAM, MIDSPAN, COVERPLATE TENSION STR
140	LOAD BENDING S
141	
142	DEAD
143	LONG TERM DEAD
144	LIVE PLUS IMPACT
145	
146	TOTAL
147	
148	ALLOWABLE INVENTORY STRESS, ksi
149	STRESS RELIEVED BY POST-TENSION
150	
151	
152	ANCHORAGE LOCATION @ 0.07 L, in
153	
154	DISTRIBUTION FACTORS
155	i, in^3
156	j, in^3
157	THETA
158	AR
159	DECK T/S
160	IET
161	
162	FF
163	MF
164	
165	TOTAL FORCE REQUIRED, kips
166	FORCE PER EXTERIOR BEAM, kips

Fig. 3.17. Continued.

Input X: Moment of inertia of interior stringer, in.^{4}

Input Y: Width of coverplate on interior stringer, in.

Input Z: Thickness of coverplate on interior stringer, in.

Input AA: Width of flange that may be taken as acting compositely with exterior stringer, in.

Input BB: Width of flange that may be taken as acting compositely with interior stringer, in.

Input CC: Bridge deck thickness, in. [Same as input L]

Input DD: Modular ratio of elasticity

Input EE: Year in which bridge was constructed

Input FF: Eccentricity of post-tension force measured from neutral axis of bridge, in.

As previously noted, the spreadsheet was primarily developed to assist the designer in determining the MF's and FF's for a given post-tensioning strengthening system. However, review of Fig. 3.17 shows that the total required post-tensioning force per exterior stringer is also provided. Note that this force is based on HS20-44 loading. No other Iowa legal loadings have been included in this spreadsheet. If the designer determines that another loading is more critical than HS20-44, this moment may be included as Input GG and the required post-tensioning force per exterior beam will be obtained. When loading other than HS20-44 is critical, moments at other locations noted in the output must also be appropriately modified.

The example problem, worked in Ref. 24, has been solved (see Fig. 3.18) utilizing the spreadsheet previously described. Note, the same MF and FF (except for the number of significant figures) were obtained by the two procedures (spreadsheet in Fig. 3.17 and hand calculations used in Ref. 24). The required post-tensioning forces calculated are different as the force in Ref. 24 is based on the critical Iowa loading whereas the forces in the spreadsheet are based on HS20-44 loading.

3.6. Strengthening Techniques for Timber Stringer Bridges (FHWA 702)

3.6.1. Respace Existing Stringers and Add New Timber Stringers

3.6.1.1. Background

This method is analogous to the steel stringer method where a bridge's strength can be increased by distributing load to additional stringers. Unlike the situation with steel stringers, in some timber stringer cases it may not be necessary to remove the deck for respacing.

An evaluation procedure for determining the effectiveness of the respace and add procedure in timber stringer bridges is developed in this section. In the following section, an example problem is presented to illustrate the spreadsheet developed for this method.

3.6.1.2. Design Criteria

The procedure outlined is based on the *AASHTO Standard Specifications for Highway Bridges* (2).

3.6.1.3. Design Limitations

This procedure evaluates the superstructure only. However, as is the case with all strengthening schemes, the increased strength in the superstructure should not exceed the capacity of the substructure. Not only must

POST-TENSIONING SPREADSHEET	
INPUT	
NGTH (ft.)	51.25
E SPACING (ft.)	9.6875
st SPACING (ft.)	1.15625
DEAD LOAD - EXT BEAM (klf)	0.865
DEAD LOAD - INT BEAM (klf)	1.147
LONG TERM D.L. - EXT BEAM (klf)	0.151
LONG TERM D.L. - INT BEAM (klf)	0.151
D TO COVERPLATE - EXT (ft.)	13.625
D TO COVERPLATE - INT (ft.)	9.125
ANCHORAGE TO EXT BEAM (ft.)	2
ANCHORAGE TO EXT BEAM (ft.)	6
SLAB THICKNESS (in.)	7.5
EXTERIOR BEAM	
AREA (in^2)	27.65
z (in.)	13.46
MOMENT OF INERTIA (in^4)	3266.7
EXTERIOR COVERPLATE	
WIDTH (in.)	9
HEIGHT (in.)	0.4375
CURB 1	
WIDTH (in.)	8
HEIGHT (in.)	4.5
CURB 2	
WIDTH (in.)	10.375
HEIGHT (in.)	6
INTERIOR BEAM	
AREA (in^2)	34.13
z (in.)	15.63
MOMENT OF INERTIA (in^4)	4919.1
NTERIOR COVERPLATE	
WIDTH (in.)	9
HEIGHT (in.)	1.25
DECK	
WIDTH INTERIOR (in.)	58.88
WIDTH EXTERIOR (in.)	90
HEIGHT (in.)	7.5
MODULAR RATIO OF ELASTICITY, n	9
YEAR BUILT	1937
ECCENTRICITY	20.95

g. 3.18. Spreadsheet for FF and MF example problem.

IMPACT FACTOR	0.284
COMPUTED EXT BEAM LOAD FRACTION	1.069
AASHTO EXT BEAM LOAD FRACTION	1.509
EXTERIOR LOAD FRACTION USED	1.509
INTERIOR LOAD FRACTION	1.761

MIDSPAN - EXTERIOR BEAM y =	25.625			
DEAD LOAD MOMENT	283.997			
LONG TERM D.L. MOMENT	49.576		50	313.95
HS20 TRUCK LOAD MOMENT	325.106		51.25	325.1062
LIVE + IMPACT LOAD MOMENT	629.556		52	331.8
MIDSPAN - INTERIOR BEAM y =	25.625			
DEAD LOAD MOMENT	376.583			
LONG TERM D.L. MOMENT	49.576			
HS20 TRUCK LOAD MOMENT	325.106			
LIVE + IMPACT LOAD MOMENT	735.079			
COVERPLATE CUTOFF - EXT BEAM y =	13.625		13	14
DEAD LOAD MOMENT	221.717	50	258.96	268.8
LONG TERM D.L. MOMENT	38.704	52	267	277.85
HS20 TRUCK LOAD MOMENT	270.530			
LIVE + IMPACT LOAD MOMENT	523.871			
COVERPLATE CUTOFF - INT BEAM y =	9.125		9	10
DEAD LOAD MOMENT	220.448	50	205.2	220.8
LONG TERM D.L. MOMENT	29.021	52	209.77	226.15
HS20 TRUCK LOAD MOMENT	210.067			
LIVE + IMPACT LOAD MOMENT	474.971			
ANCHORAGE - EXT BEAM y =	2		2	3
DEAD LOAD MOMENT	42.601	50	55.68	81.36
LONG TERM D.L. MOMENT	7.437	52	56.31	82.38
HS20 TRUCK LOAD MOMENT	56.074			
LIVE + IMPACT LOAD MOMENT	108.585			
ANCHORAGE - EXT BEAM y =	6		6	7
DEAD LOAD MOMENT	117.424	50	149.76	169.68
LONG TERM D.L. MOMENT	20.498	52	152.31	172.85
HS20 TRUCK LOAD MOMENT	151.354			
LIVE + IMPACT LOAD MOMENT	342.217			

Fig. 3.18. continued.

SECTION PROPERTIES

EXTERIOR BM FLANGE WIDTH		58.875		90.75	72	58.875

BASIC QUANTITIES

ITEM			AREA	Z	AREA*Z	AREA*Z^2	Io
BEAM			27.65	13.46	372.169	5009.394	3266.7
COVERPLATE			3.938	-0.21875	-0.86132	0.188415	0.062805
DECK	n =	9	49.067	29.66	1455.317	43164.71	230
DECK	n =	27	16.356	29.66	485.1057	14388.23	76.66666
CURB 1	n =	9	4	35.66	142.64	5086.542	6.75
CURB 1	n =	27	1.333	35.66	47.54666	1695.514	2.25
CURB 2	n =	9	6.917	40.91	282.9608	11575.92	20.75
CURB 2	n =	27	2.306	40.91	94.32027	3858.642	6.916666

CENTROID ELEVATIONS AND MOMENT OF INERTIA

DESCRIPTION		z	Iz
STEEL BEAM		13.46	3266.7
STEEL BEAM WITH COVERPLATE		11.75489	3911.664
COMPOS BM, DECK AND CURB n=	9	25.71038	10433.02
COMPOS BM, DECK, CURB & COVRPLn=	9	24.59544	12966.52
COMPOS BM, DECK AND CURB n=	27	20.97079	7351.529
COMPOS BM, DECK, CURB & COVRPLn=	27	19.35329	8984.562

INTERIOR BM FLANGE WIDTH	90	153.75	116.25	90

BASIC QUANTITIES

ITEM			AREA	Z	AREA*Z	AREA*Z^2	Io
BEAM			34.13	15.63	533.4519	8337.853	4919.1
COVERPLATE			11.25	0	0	0	1.464843
DECK	n =	9	75	33.38	2503.5	83566.83	351.5625
DECK	n =	27	25	33.38	834.5	27855.61	117.1875

CENTROID ELEVATIONS AND MOMENT OF INERTIA

DESCRIPTION		z	Iz
STEEL BEAM		15.63	4919.1
STEEL BEAM WITH COVERPLATE		11.75522	6987.573
COMPOS BM AND DECK n=	9	27.82875	12660.75
COMPOS BM, DECK & COVRPL n=	9	25.22804	20560.45
COMPOS BM AND DECK n=	27	23.13465	9582.661
COMPOS BM, DECK & COVRPL n=	27	19.43665	14642.80

COMPOSITE ELEVATION AND MOMENT OF INERTIA W/R/T COMPOSITE BRIDGE

n = 9	z	EXT BEAM	INT BEAM
COVERPLATES ON ALL BEAMS	24.955	12978.34	20569.44
COVERPL ON INT BEAM ONLY	25.431	10439.85	20565.42
NO COVERPL	26.885	10553.99	12757.89

g. 3.18. Continued.

POST TENSIONING DESIGN
EXTERIOR BEAM, MIDSPAN, COVERPLATE TENSION STRESSES

LOAD	BENDING STRESS, ksi
DEAD	10.622
LONG TERM DEAD	1.310
LIVE PLUS IMPACT	14.585
TOTAL	26.518

ALLOWABLE INVENTORY STRESS, ksi	18.15
STRESS RELIEVED BY POST-TENSION	8.368
ANCHORAGE LOCATION @ 0.07 L, in	43.05
DISTRIBUTION FACTORS	
i, in^3	178.209
j, in^3	6.100
THETA	0.712
AR	0.712
DECK T/S	0.065
IET	0.387
FF	0.386
MF	0.295
TOTAL FORCE REQUIRED, kips	512.753
FORCE PER EXTERIOR BEAM, kips	256.376

Fig. 3.18. Continued.

the structural elements of the substructure be evaluated, but the geotechnical aspects of the load carrying capacity must also be investigated to complete the design.

The following limitations apply to the timber stringer bridge being evaluated by this procedure:

- Stringers are assumed to have similar material and section properties.
- The superstructure is not skewed.
- The deck section and material properties are homogeneous.

The engineer should also be aware that the structural properties of timber are widely variable and that decay in existing timbers in the field may also vary greatly from stringer to stringer. The bridge should be carefully inspected to detect locations of inadequate structural strength.

3.6.1.4. Design Procedure

1. **Determine the section modulus of the stringers. (D)** This analysis approach is based on the load distributed to individual stringers. Initially, the section modulus of the stringer must be calculated. Note, nominal dimensions of the stringers may be based on surfaced green or dry lumber. However, minimum dry dressed dimensions should be used in design calculations. Dry dressed dimensions for beams and stringers are usually assumed to be 1/2 in. less than the nominal dimensions. The section modulus for a rectangular stringer is:

   $$\text{Section Modulus } (S_{modulus}) = [\text{Width x (Beam Height)}^3]/12$$

2. **Determine the allowable timber stress. (A)** See AASHTO (2) Table 13.2.1(A) for the appropriate allowable unit stresses. In the questionnaires, county engineers indicated that the majority of their existing bridges are Douglas Fir. The extreme bending fiber stress, F_b, for Douglas Fir stringers varies from 1200 psi to 1900 psi, depending on the grade of timber.

3. **Determine the moment capacity of the interior stringers. (I)** In this step, the moment capacity of a typical internal stringer is determined.

   Stringer Moment Capacity = Section Modulus x Allowable Stress

   $$M_{capacity} = S_{modulus} \times \sigma_{all}$$

4. **Determine the dead load moment on the superstructure. (J)** Dead load includes all permanent load associated with the superstructure and roadway, including stringers, deck, wearing surface, railings, lighting, etc.

   Applied Dead Load Moment = [Uniform Dead Load x (Bridge Length)²]/8

   $$M_{DL} = (\omega_{DL} \times L^2)/8$$

5. **Determine the live load moment capacity of the superstructure. (K)** The stringer live load moment capacity is the moment capacity of the internal stringer minus the applied dead load moment capacity.

Live Load Moment Capacity = Total Moment Capacity - Applied Dead Load Moment

$$M_{LL\ capacity} = M_{capacity} - M_{DL}$$

6. **Determine the AASHTO live load distribution factor. (L & M)** See AASHTO Table 3.23.1 for the appropriate distribution factors. Timber bridges with timber plank decks and concrete decks represent the greatest percentage of deficient bridges in the SI&A survey for Iowa secondary bridges. AASHTO distribution factors for timber stringers supporting timber or concrete decks are shown in Table 3.7.

Table 3.7. AASHTO wheel load distribution factors - timber stringers.

Kind of Floor	One Lane Bridge (M)	Two Lane Bridge (L)
Timber Plank	S/4.0 (1)	S/3.75 (2)
Concrete	S/6.0 (3)	S/5.0 (4)

S = average stringer spacing
(1) If the spacing, S, exceeds 5.0 ft use the reaction of the wheel loads.
(2) If the spacing, S, exceeds 6.5 ft use the reaction of the wheel loads.
(3) If the spacing, S, exceeds 6.0 ft use the reaction of the wheel loads.
(4) If the spacing, S, exceeds 10.0 ft use the reaction of the wheel loads.

7. **Determine the maximum truck load moment demand by hand calculations or from applicable tables. (N)** As with the steel stringer calculations, the truck live load moments are determined:

$$M_{LL\ demand} = M_{truck} \times 2 \times Dist_{LL}$$

M_{truck} = live load moments (foot-kips per wheel line). See Appendix B for Iowa live load truck moments.

$Dist_{LL}$ = distribution factor.

2 represents 2 wheel lines

As noted in the AASHTO specifications, impact allowances need not be applied to timber structures.

8. **Determine Iowa DOT legal truck load posting values from operating rating. (O)** The live load demand (based on the operating stress) is compared with the live load capacity to determine the loadings which require posting. (See Appendix A for truck weights.)

   Operating Rating = $(M_{LLcapacity}/M_{LLdemand})$ x *Truck Weight*

9. **Determine the HS inventory rating (for use in calculating the SA&I sufficiency rating). (P)** The HS20 inventory rating is determined by:

   Inventory Rating = $(M_{LLcapacity}/M_{LLdemand})$ x *Truck Weight*

3.6.1.5. Respace and add timber stringers example: Spacing = 2 ft - Case I

The following example evaluation is for a 20 ft wide and 20 ft long bridge with 2 ft spacing between the stringers. In Case II, the stringer spacing has been reduced to 1 ft spacing between the stringers. As will be seen, reducing the stringer spacing increases the maximum SI&A sufficiency rating. The given calculations follow the spreadsheet developed for timber evaluations shown in Fig. 3.19.

1. **Determine the section modulus of the stringers. (D)** The nominal dimension of the timber stringers in this example are 4 in. x 14 in. Therefore, the minimum dry dressed dimensions are 3 1/2 in. x 13 1/2 in.

   Section Modulus = [*Beam Width* x (*Beam Height*)^3]/12

   $$S_{modulus} = (3\ 1/2\ in. \times 13\ 1/2\ in.^3)/12 = 717.6\ in.^4$$

2. **Determine the allowable timber stress. (A)** See AASHTO (2) Table 13.2.1A for the appropriate allowable unit stresses. The timber used in this example is Douglas Fir. Conservatively, in this example, assume the extreme bending fiber stress, F_b, for the stringers to be 1200 psi.

   $$\sigma_{all} = 1200\ psi$$

3. **Determine the moment capacity of the interior stringers. (I)**

   Stringer Moment Capacity = Section Modulus x Allowable Stress

   $$M_{capacity} = S_{modulus} \times \sigma_{all}$$

   $$M_{capacity} = (717.6\ in^4 \times 1200\ psi)/12{,}000 = 71.76\ ft\text{-}k$$

4. **Determine the dead load moment on the superstructure. (J)** The uniform dead load in this example includes the stringers, deck and compacted gravel load and is assumed to be *200* lb/ft.

TIMBER STRINGER EVALUATION SPREADSHEET

INPUT		
ALLOWABLE STRESS, (PSI)	1200	A
BRIDGE LENGTH (FEET)	20	B
BEAM SPACING (FEET)	2	C
SECTION MODULUS (INCH^3)	717.6	D
DEAD LOAD (KLF)	0.2	E
FLOOR TYPE FROM BELOW 1 = TIMBER 2 = CONCRETE	1	F

OUTPUT LOAD TO POSTED ON LOAD LIMIT SIGN			
TRUCK	2 LANES	1 LANE	
HS20	14.48	15.44	O
TYPE 3	21.03	22.43	
TYPE 3S2(a)	33.66	24.60	
TYPE 3S2(b)	33.64	22.43	
TYPE 4	20.49	20.05	
TYPE 3S3	30.08	20.05	
TYPE 3-3	NO POST	27.12	

INTERMEDIATE VALUES			
ALLOWABLE STRESS (KSI)	1.2		
OPERATING STRESS (KSI)	1.2		
MOMENT CAPACITY (KIP-FT)	INVENTORY	OPERATING	
TOTAL STRUCTURE	71.76	71.76	I
DEAD LOAD	10.00	10.00	J
LIVE LOAD	61.76	61.76	K
LIVE LOAD DISTRIBUTION FACTORS			
TWO LANES OR MORE	0.5333	L	
ONE LANE	0.5000	M	

LIVE LOAD + IMPACT MOMENT			
TRUCK	2 LANES	1 LANE	
HS20	85.33	80.00	N
TYPE 3	73.44	68.85	
TYPE 3S2(a)	66.96	62.78	
TYPE 3S2(b)	73.44	68.85	
TYPE 4	82.13	77.00	
TYPE 3S3	82.13	77.00	
TYPE 3-3	60.73	56.94	

BRIDGE LOAD IN TONS			
TRUCK	2 LANES	1 LANE	MAX LOAD
HS20	14.48	15.44	20.00
TYPE 3	21.03	22.43	25.00
TYPE 3S2(a)	33.66	24.60	36.50
TYPE 3S2(b)	33.64	22.43	40.00
TYPE 4	20.49	20.05	27.25
TYPE 3S3	30.08	20.05	40.00
TYPE 3-3	40.68	27.12	40.00

MAX S.I. & A. RATING			
TRUCK	2 LANES	1 LANE	
HS20	14.48	15.44	P
BEST S.I. & A. SUFFICENCY RATING	72.26	74.10	

Fig. 3.19. Spreadsheet for timber stringer example.

The length of the bridge is 20 ft (B).

Applied Dead Load Moment = [Uniform Dead Load x (Bridge Length)²]/8

$$M_{DL} = (\omega_{DL} \times L^2)/8$$

$$M_{DL} = [0.200\ k/ft \times (20\ ft.)^2]/8 = 10.0\ ft\text{-}k$$

5. Determine the live load moment capacity of the superstructure. (K)

The moment capacity of the interior stringer is:

Live Load Moment Capacity = Total Moment Capacity - Applied Dead Load Moment

$$M_{LL\ capacity} = M_{capacity} - M_{DL}$$

$$M_{LL\ capacity} = 71.76\ ft\text{-}k - 10.0\ ft\text{-}k = 61.76\ ft\text{-}k$$

6. Determine the AASHTO live load distribution factor. (L & M) See Table 3.7 for the appropriate AASHTO distribution factors. A timber plank deck with two lanes of traffic and stringer spacing of 2 ft is used in this example.

$$LL\ Distribution\ Factor = \frac{2.0\ ft}{3.75} = 0.533$$

7. Determine the maximum truck load moment demand by hand calculations or from applicable tables. (N)

$$M_{LL\ demand} = M_{truck} \times 2 \times Dist._{LL}$$

- *HS20: 80.0 ft-k x 2 x 0.533 = 85.28 ft-k*
- *Type 3: 68.9 ft-k x 2 x 0.533 = 73.45 ft-k*
- *Type 3S2(A): 62.8 ft-k x 2 x 0.533 = 66.95 ft-k*
- *Type 3S2(B): 68.9 ft-k x 2 x 0.533 = 73.45 ft-k*
- *Type 4: 77.0 ft-k x 2 x 0.533 = 82.08 ft-k*
- *Type 3S3: 77.0 ft-k x 2 x 0.533 = 82.08 ft-k*
- *Type 3-3: 56.94 ft-k x 2 x 0.533 = 60.73 ft-k*

Note, these values are slightly different than those shown in Fig. 3.19 due to the significant figures used in the (.533) term in the spreadsheet.

8. **Determine Iowa DOT legal truck load posting values from the operating rating. (O)** The live load demand (based on the operating stress) is compared with the live load capacity to determine the loadings which require posting. (See Appendix A for truck weights.)

$$Operating\ Rating = (M_{LL\ capacity}/M_{LL\ demand}) \times Truck\ Weight$$

- *Type 3: (61.76 ft-k/73.45 ft-k) x 25 ton = 21.02 ton < 25 ton*
- *Type 3S2(A): (61.76 ft-k/66.95 ft-k) x 36.5 ton = 33.67 ton < 36.5 ton*
- *Type 3S2(B): (61.76 ft-k/73.45 ft-k) x 40 ton = 33.64 ton < 40 ton*
- *Type 4: (61.76 ft-k/82.08 ft-k) x 27.25 ton = 20.50 ton < 27.5 ton*
- *Type 3S3: (61.76 ft-k/82.08 ft-k) x 40 ton = 30.10 ton < 40 ton*
- *Type 3-3: (61.76 ft-k/82.08 ft-k) x 40 ton = 40.68 ton > 40 ton*

9. **Determine the HS inventory rating (for use in calculating the SA&I sufficiency rating). (P)** The maximum *SI&A sufficiency rating for this configuration* is 72.26.

3.6.1.6. Respace and Add Timber Stringer Example: Spacing = 1 ft - Case II

Decrease the stringer spacing to *1* ft by adding stringers. Conservatively, the dead load has been assumed not to change from the previous example (Case I with 2 ft stringer spacing). The maximum SI&A sufficiency rating for this configuration is determined to be 94.80 (increased by 31 percent), and the bridge no longer requires posting. See spreadsheet calculations for this configuration in Fig. 3.20.

3.6.2. Replace Limited Number of Timber Stringers with Steel Stringers

Frequently counties have access to surplus steel beams. Rather than adding and respacing timber stringers as was presented in Sec. 3.6.1, another strengthening alternative is to replace a limited number of the timber stringers with surplus steel beams. As the resulting bridge is one with stringers of different strengths and stiffnesses, it is necessary to analyze the bridge utilizing the finite-element method (FEM). Timber bridges of various lengths (12 ft through 30 ft), widths (16 ft-one lane and 24 ft-two lanes), stringer sizes (4 in. x 12 in. and 6 in. x 12 in.), and stringer spacings (8 in., 12 in., and 16 in.) were analyzed using the FEM to determine flexural stresses resulting from Iowa legal loads (see Appendix A). Based on a thorough preliminary analysis -- the required number, position, and size of steel stringers were determined.

Depths of steel stringers used in the analysis was limited to either 12 in. or 16 in. since timber stringers used in the field are usually one of these depths. Obviously, other sizes of steel stringers could be used, however, using greater depths would be more involved in that the greater depths would require modification of the support so that the elevations of the top surfaces of the steel stringers and timber stringers were essentially the same to facilitate replacement of the timber deck. Shown in Fig. 3.21 are the positions of the steel stringers which have replaced existing timber stringers. As illustrated, four steel stringers are required in a two lane bridge (Fig. 3.21a) and three steel stringers are required in a one lane bridge (Fig. 3.21b) to increase the capacity of the bridge for Iowa legal loads. Figure 3.22 illustrates the effect of the steel stringers on the stress in the timber stringers.

TIMBER STRINGER EVALUATION SPREADSHEET

INPUT	
ALLOWABLE STRESS, (PSI)	1200
BRIDGE LENGTH (FEET)	20
BEAM SPACING (FEET)	1
SECTION MODULUS (INCH^3)	717.6
DEAD LOAD (KLF)	0.2
FLOOR TYPE FROM BELOW 1 = TIMBER 2 = CONCRETE	1

OUTPUT LOAD TO POSTED ON LOAD LIMIT SIGN		
TRUCK	2 LANES	1 LANE
HS20	NO POST	NO POST
TYPE 3	NO POST	NO POST
TYPE 3S2(a)	NO POST	NO POST
TYPE 3S2(b)	NO POST	NO POST
TYPE 4	NO POST	NO POST
TYPE 3S3	NO POST	NO POST
TYPE 3-3	NO POST	NO POST

INTERMEDIATE VALUES

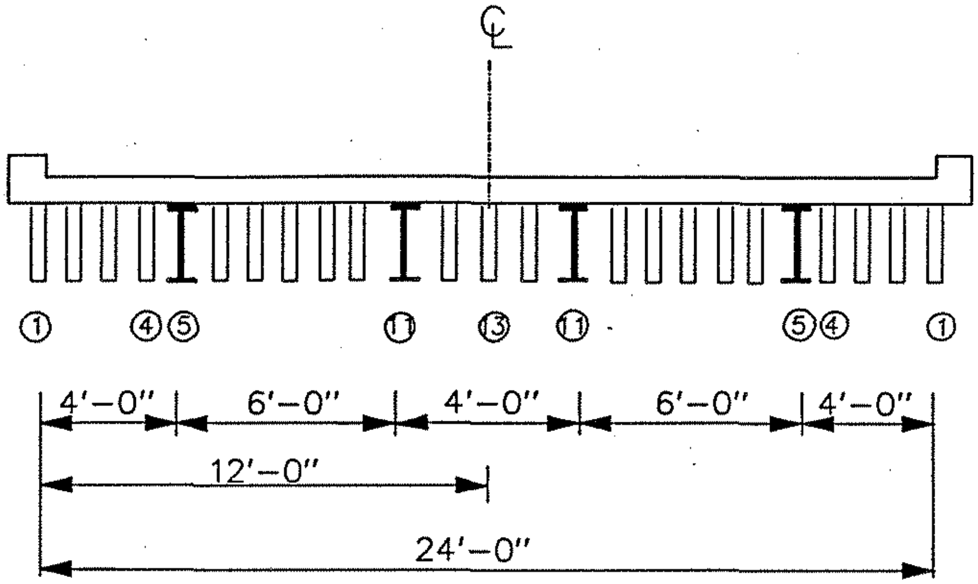
ALLOWABLE STRESS (KSI)	1.2	
OPERATING STRESS (KSI)	1.2	
MOMENT CAPACITY (KIP-FT)	INVENTORY	OPERATING
TOTAL STRUCTURE	71.76	71.76
DEAD LOAD	10.00	10.00
LIVE LOAD	61.76	61.76
LIVE LOAD DISTRIBUTION FACTORS		
TWO LANES OR MORE	0.2667	
ONE LANE	0.2500	

LIVE LOAD + IMPACT MOMENT		
TRUCK	2 LANES	1 LANE
HS20	42.67	40.00
TYPE 3	36.72	34.42
TYPE 3S2(a)	33.48	31.39
TYPE 3S2(b)	36.72	34.42
TYPE 4	41.07	38.50
TYPE 3S3	41.07	38.50
TYPE 3-3	30.37	28.47

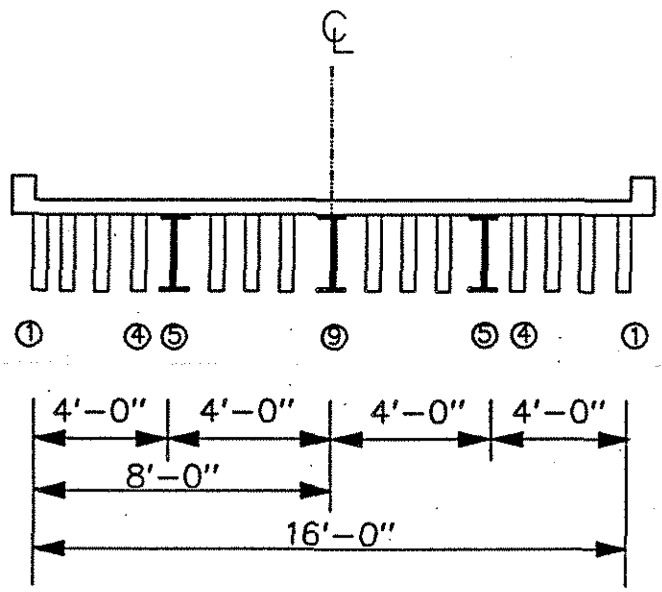
BRIDGE LOAD IN TONS			
TRUCK	2 LANES	1 LANE	MAX LOAD
HS20	28.95	30.88	20.00
TYPE 3	42.05	44.85	25.00
TYPE 3S2(a)	67.33	49.19	36.50
TYPE 3S2(b)	67.28	44.85	40.00
TYPE 4	40.98	40.10	27.25
TYPE 3S3	60.16	40.10	40.00
TYPE 3-3	81.35	54.23	40.00

MAX S.I. & A. RATING		
TRUCK	2 LANES	1 LANE
HS20	28.95	30.88
BEST S.I. & A. SUFFICENCY RATING	94.80	96.78

Fig. 3.20. Spreadsheet for timber stringer respace and add example.



a. Two Lane Bridge



b. One Lane Bridge

Fig. 3.21. Position of steel stringers in two lane and single lane bridge.

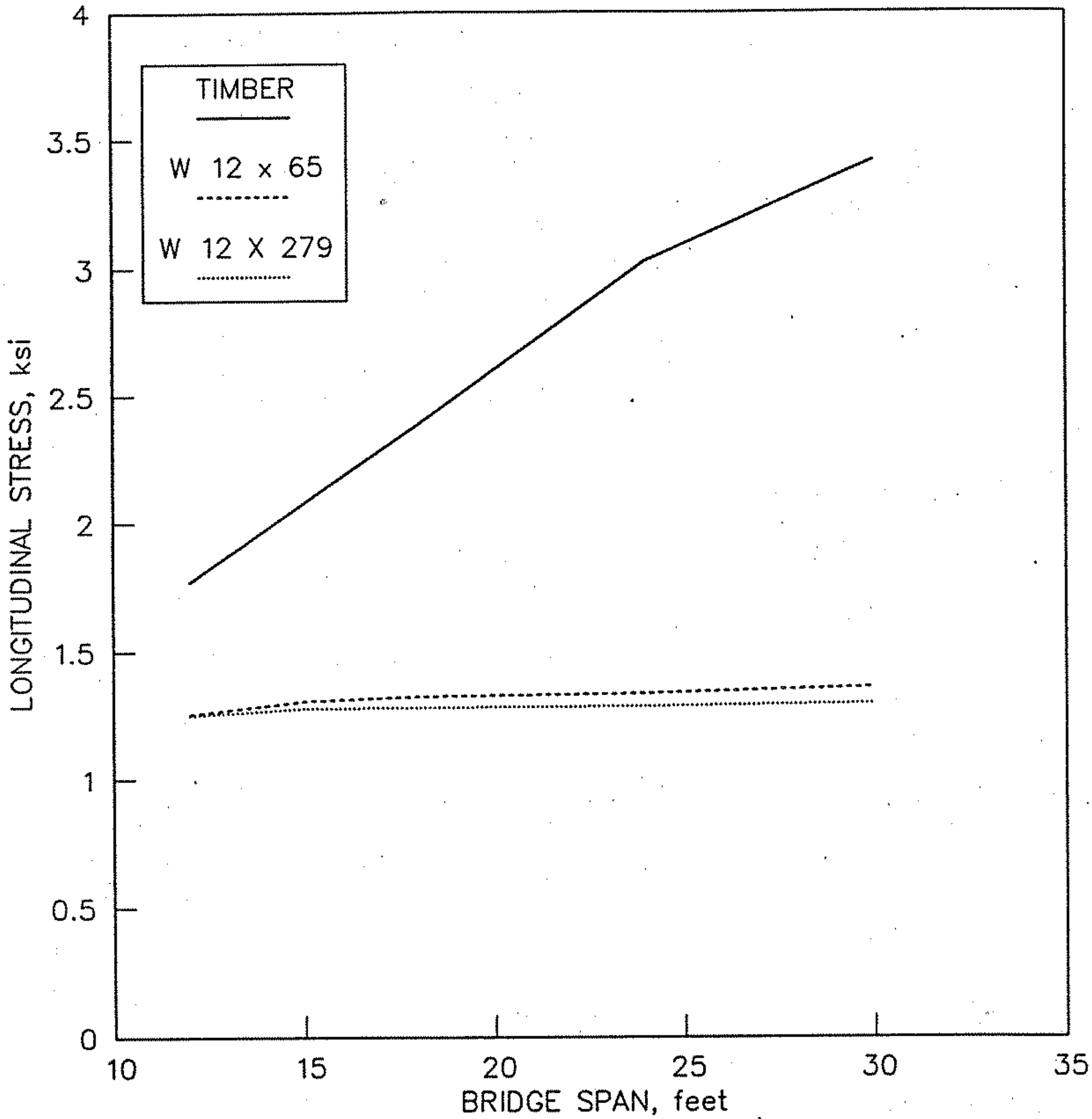


Fig. 3.22. Maximum longitudinal stress in timber stringers: Bridge width = 24 ft; Stringer size = 4 in. x 12 in.; Stringer spacing = 12 in.

Stringer stresses (timber and steel) are calculated assuming that adequate support to the compression portion of the stringer has been provided. As the span length increases, different legal loads govern--thus, the change in slope in the curves shown in Fig. 3.22. Several steel stringers in addition to those illustrated were investigated, however curves for the other stringers analyzed lie between the two steel curves shown. Thus, one can conclude within limits the reduction in timber stresses are essentially independent of steel stringer size. Stresses in the added steel stringers are obviously a function of steel stringer size. This is illustrated in Fig. 3.23 where the reduction in steel stresses with increased, steel stringer size is shown. This same effect is illustrated in Fig. 3.24, where steel stringer stresses vs. steel stringer moment of inertias are presented. Additional computer evaluations verified that stresses in the steel stringers and timber stringers were essentially independent of stringer spacings.

The effect of the addition of steel stringers on midspan bridge displacements for one particular situation is shown in Fig. 3.25. As may be seen, the addition of steel stringers significantly reduces the displacements; the larger the steel stringers the greater the reduction in displacements.

Figures 3.22 and 3.23 may be used to determine if a given bridge can be strengthened the desired amount by adding steel stringers. Although these two figures are for one timber stringer size (4 in. x 12 in.) and one stringer spacing (12 in.), it has been shown that this strengthening procedure is essentially independent of these two variables, and thus these two figures may be used for essentially any practical stringer spacing or size. Using Fig. 3.22 for a given length of span, one may determine the stress reduction in the timber stringers resulting from the addition of steel stringers. Entering Fig. 3.23 with the given length of span and limiting stress, one can determine the size of stringer required. Obviously, stringer sizes other than those shown in Fig. 3.23 may be used if the moment of inertias are essentially the same. Figures 3.22 and 3.23 were developed for two lane bridges - however, conservatively they may be used for one lane bridges where three steel stringers are required rather than four.

With the additional steel stringers, load carried by the remaining timber stringers is reduced, since the added steel stringer carries a larger percentage of the loading. The load carried by the steel stringer is directly proportional to the stiffness of the stringer (i.e. the stiffer the stringer the more load it carries). Abutments which were originally designed for essentially uniform loading now need to be reanalyzed and possibly strengthened to support the reactions from the added steel stringers.

3.7. Replacement Bridges

This section provides a range of replacement bridges with short span application. While some of the alternatives are technically not bridge structures (e.g. low water stream crossing and corrugated metal pipe culverts), they serve as means for vehicles to traverse roadway obstacles. Some of the briges presented are proprietary and information is provided so preliminary decisions can be made regarding potential use of a particular bridge type. Detailed design procedures and microcomputer design spreadsheets are provided for three different timber bridge types. The following sections describe each bridge replacement type.

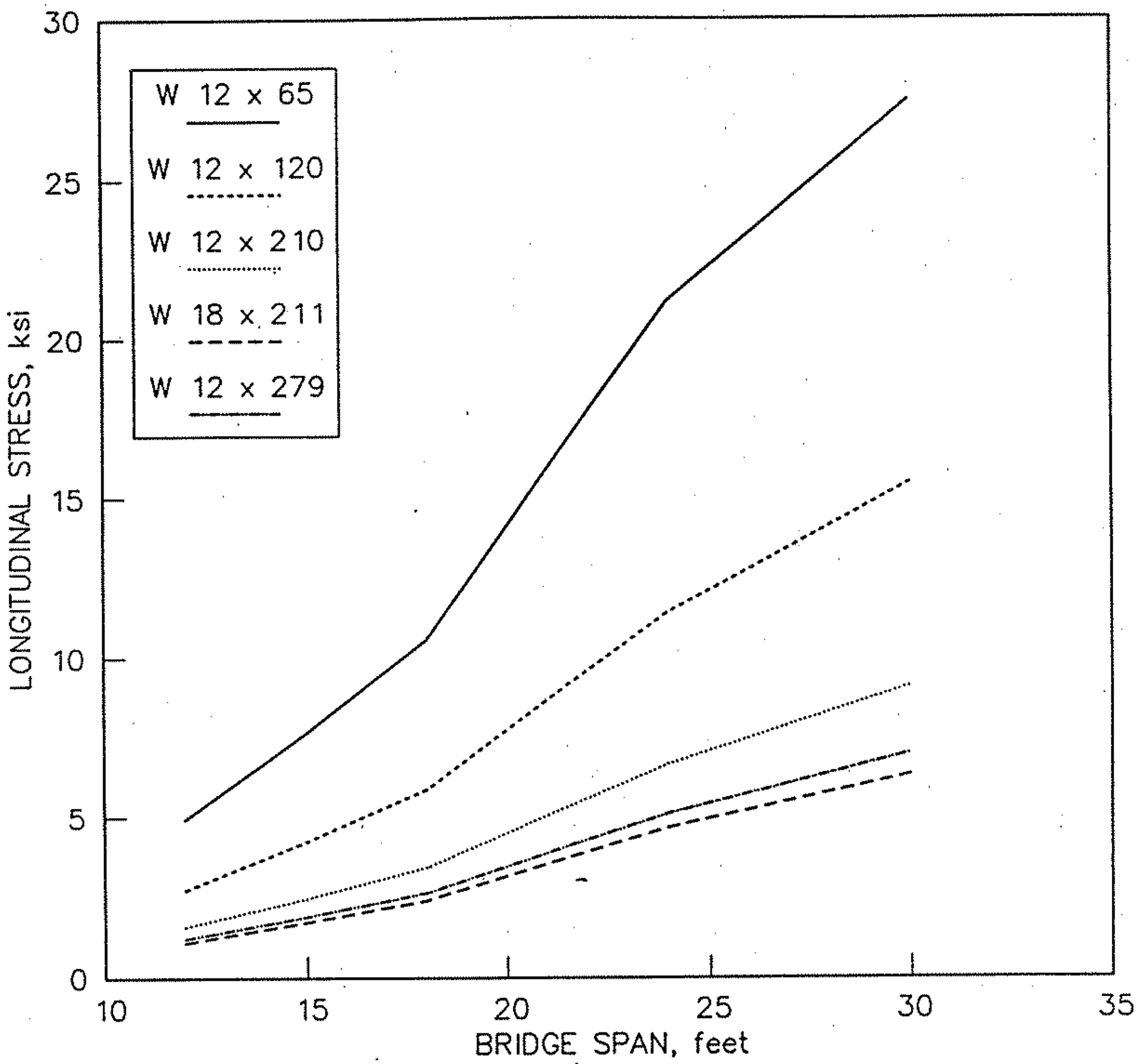


Fig. 3.23. Maximum longitudinal stress in steel stringers: Bridge width = 24 ft; Stringer size = 4 in. x 12 in.; Stringer spacing = 12 in.

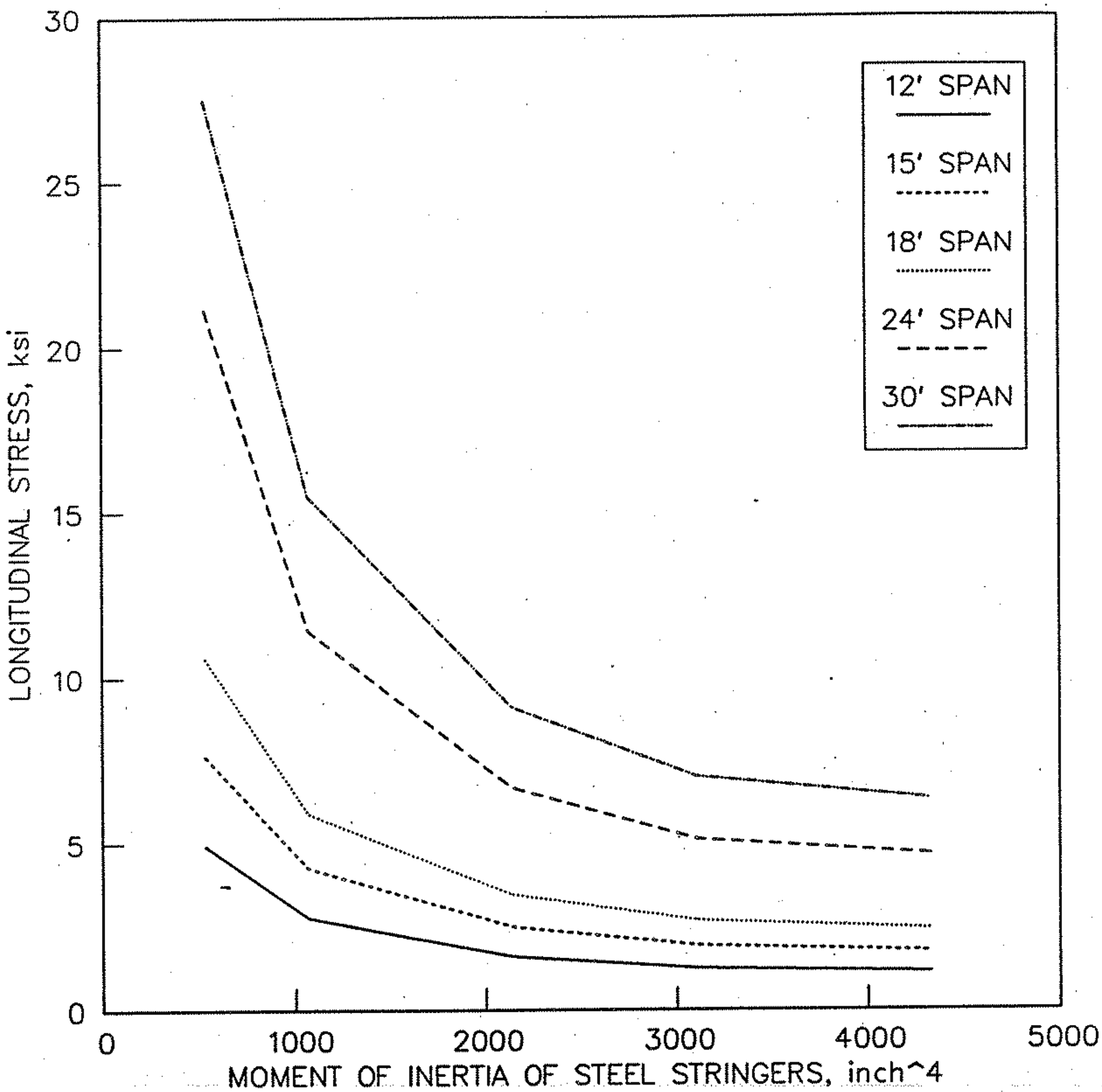


Fig. 3.24. Maximum longitudinal stress in steel stringers: Bridge width = 24 ft; Stringer size = 4 in. x 12 in.; Stringer spacing = 12 in.

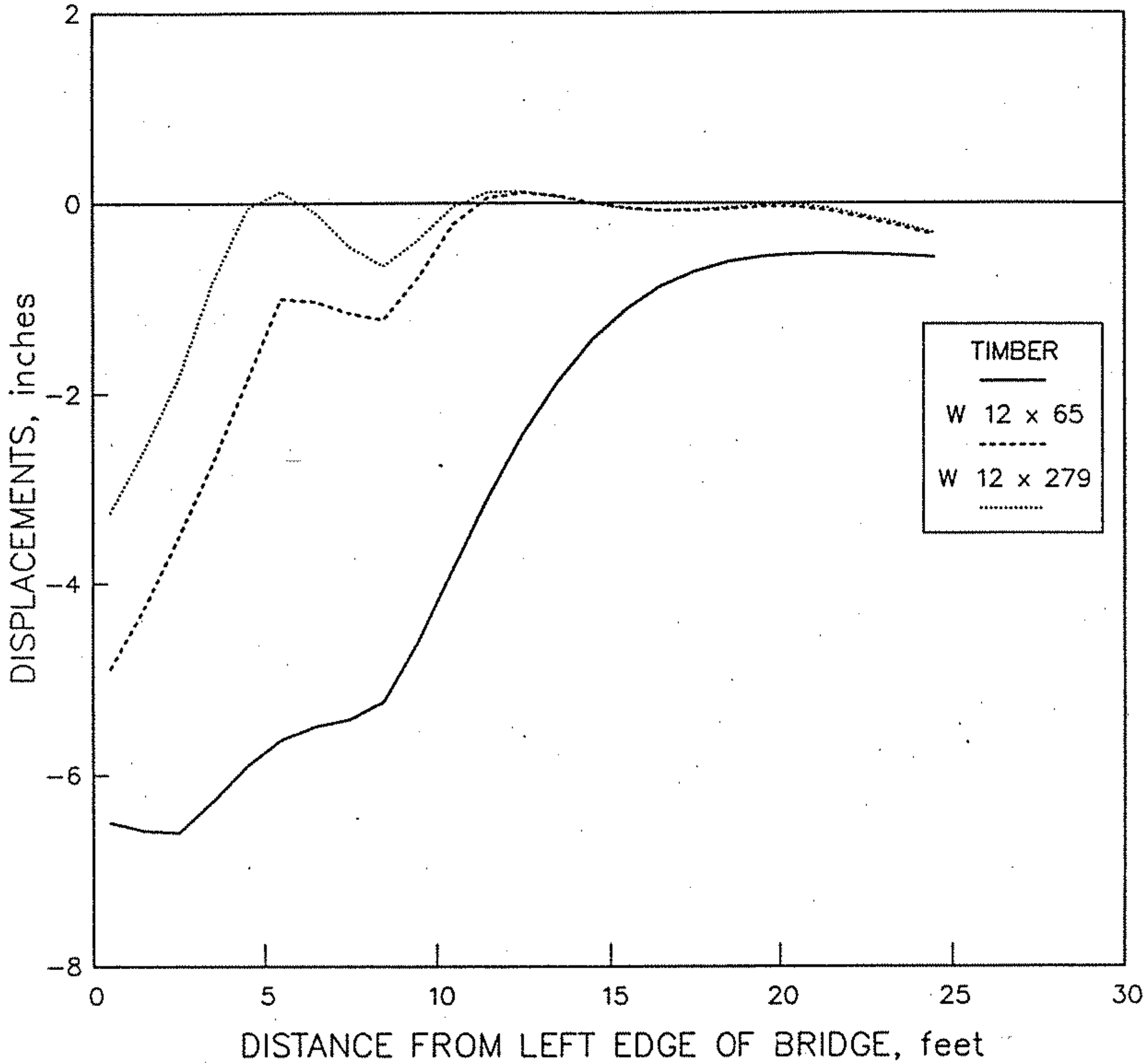


Fig. 3.25. Displacements in timber and steel stringers:
Bridge width = 24 ft; Stringer spacing = 12 in.;
Bridge span = 18 ft; Stringer size = 4 in. x 12 in.

3.7.1. Precast Culvert/Bridge

The Con-Span precast culvert system is a proprietary system licensed by the Con-Span culvert company of Dayton, OH.

3.7.1.1. Background

The Con-Span precast culvert system was specifically developed to provide a large hydraulic cross-section with a limited vertical clearance. These precast culvert sections are available in 4 span lengths: 16 ft, 20 ft, 24 ft and 36 ft and in rises from 5 ft to 10 ft. If these span lengths are insufficient to meet the needs of a particular location, multiple opening arrangements may be used.

Figure 3.26 illustrates the Con-Span culvert system. The arch-box shape allows the culvert to carry more load than an ordinary reinforced concrete box culvert. When the culvert deflects, thrust is developed by the passive earth pressure of the backfill, thus resisting deflection of the arch top. The culvert cannot collapse without displacing the block of soil behind the sidewalls a sufficient amount to allow the arch to collapse. In load tests, the culvert supported a load nearly twice the specified design load of 35 kips (HS-20 loading). The results of the load test demonstrated the amount of reserve load-carrying capacity present in the Con-Span system.

Although each installation has site specific details, the procedure for installing a Con-Span culvert is:

- Pour strip footings to support precast units.
- Set precast Con-Span units in place on footings in a bed of cement grout.
- Install engineering fabric over joints to prevent the intrusion of any backfill.
- Bolt units together with simple joint connectors on vertical sides.
- Install precast wingwall (if desired).

One advantage of the Con-Span system is the availability of precast headwalls and wingwalls. These eliminate the use of time-consuming cast-in-place operations. The headwalls are monolithic with the archbox end units, while the wingwalls are bolted on after the units are set in place.

3.7.1.2. Design criteria

All Con-Span culvert units are designed to meet AASHTO HS-20 loading criteria. It is possible, however, to design the culverts to carry essentially any loading. The actual structural design work is performed by Con-Span engineers in Dayton, Ohio. Engineers desiring to use the Con-Span system simply supply the desired span length, height of rise, and depth of cover at the particular site location, along with the desired design load. Con-Span has established a telephone facsimile station to provide rapid preliminary designs. Con-Span engineers provide a preliminary design within 1 hr. after receiving the necessary information.

3.7.1.3. Design requirements

To compute the required hydraulic capacity for a particular culvert location, Con-Span, Inc. has developed a series of graphs which assist the engineer in determining the required culvert size. Plots of headwater depth (ft) vs. discharge (ft^3/sec) along with tables of waterway areas for the various span/rise combinations are available for preliminary calculations.

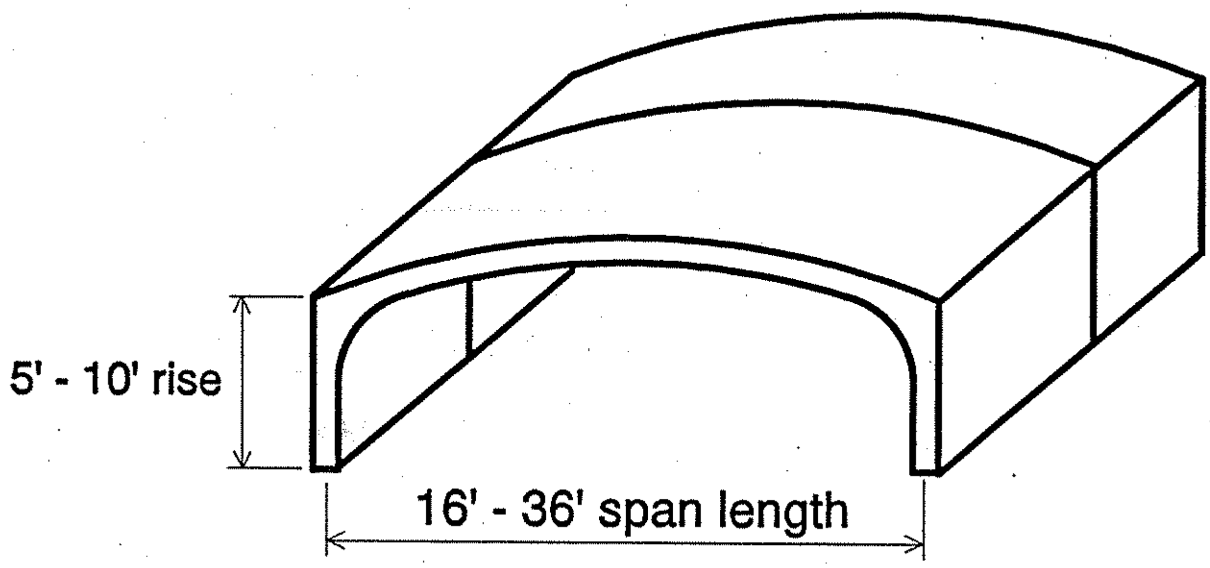


Fig. 3.26. Con–Span precast bridge segment.

For those engineers who have a microcomputer available, a series of input files for the FHWA HY-8 culvert analysis microcomputer program have been developed. This program, available from the University of Florida, automates the FHWA culvert design procedure. The Con-Span input files, along with the HY-8 program, assist the engineer in determining the optimum culvert section for a particular site. To use the program, basic hydraulic information needs to be provided, such as: culvert inlet conditions, slope of culvert invert, desired culvert capacity, etc. The HY-8 program also provides a limited amount of hydrograph generation and other hydrologic computations if no other methods are available.

3.7.1.4. General cost data

Specific cost information is available for the Con-Span system. In Iowa, the Con-Span system is available from Iowa Concrete Products (West Des Moines, Iowa).

The cost of the Con-Span system can be divided into several parts: substructure costs, cost of precast units, transportation charges, backfilling, paving, etc.

The cost of substructure work is the most difficult to quantify accurately. As with any other bridge replacement option, the required substructure is extremely site-specific. The detailed case studies described later provides a general estimate of the substructure cost.

The cost of the precast units alone (F.O.B. plant) provided by Iowa Concrete Products is presented in Table 3.8. In addition to the unit prices shown, a royalty fee of $500 per structure must be paid to Con-Span, Inc.

Transportation charges from the Hampton plant can be computed from the following:

Loads $\geq$ 45,000 lb	$2.60/mi (loaded)
< 45,000 lb	$2.50/mi (loaded)

In addition, trucks detained at a project site will be billed at a rate of $40.00/hr. after the first hour.

Table 3.8. Prices for Con-Span culvert units.

Span (ft)	HEIGHT		OF		RISE		
	5 ft	6 ft	7 ft	8 ft	9 ft	10 ft	add per wingwall
16	$1727	$1818	$1914	$2010	$2106	$2202	$632
20	$1980	$2087	$2193	$2300	$2402	$2508	$643
24	$2252	$2345	$2474	$2584	$2695	$2802	$684

Joint connectors: $40.00/joint

3.7.1.5. Cost Information: Case Studies

Two detailed case studies from Bremer County and Winnebago County, Iowa are presented as examples of Con-Span projects.

The Bremer County project, constructed during the winter of 1988-89, consisted of replacing a quad 4 ft x 6.5 ft x 44 ft laminated wood box culvert with a 20 ft x 9 ft x 64 ft Con-Span culvert. County costs for the project are presented in Table 3.9.

The road was closed for less than 2 weeks for the entire installation project. In addition, by extending the culvert to a length of 64 ft, the need for guardrails was eliminated. This resulted in a savings of an additional $10,273 on the project.

The Winnebago County project involved the replacement of an existing timber bridge with a 16 ft x 5 ft x 136 ft Con-Span culvert. This particular site was situated on a 45 degree skew, which necessitated a considerably longer culvert. The labor on this project, with the exception of substructure work, which was completed by a private contractor, was performed by county forces. Costs for the project are presented in Table 3.10.

As with the Bremer County example, this installation eliminated the guardrails at the site, further reducing project costs.

3.7.2. Air Formed Arch Culvert

3.7.2.1. Background

A new method of culvert construction has recently been developed. The Air-O-Form process, designed by Concepts in Concrete, of Norman, OK, uses an inflatable rubber membrane as the inside form for the construction of a reinforced concrete arch culvert. The inflatable form can be used to construct numerous cross-section shapes and can be inflated quickly with a minimum of labor.

The following steps are involved in the installation of an Air-O-Form culvert:

- Place a reinforced concrete slab or footing.
- Place flexible metal straps in the desired shape of the culvert. Inflate the balloon form inside the straps.
- Place longitudinal and vertical reinforcing steel.
- Adjust air pressure inside the "balloon" to the required pressure.
- Apply 6 in. of shotcrete (in one lift).
- Deflate and remove the membrane after the shotcrete has attained the required strength.

A demonstration project has recently been completed by the Iowa DOT to demonstrate the construction of an Air-O-Form culvert. The conclusions of the Iowa DOT suggest that the Air-O-Form system is better suited for longer and larger diameter culvert applications, where the economics are more favorable (92).

Table 3.9. Bremer County Con-Span culvert installation costs.

Item and Description	Cost
• Labor (all county employees)	$ 5,581
Remove existing pavement	
Excavation	
Remove existing wood box culvert	
Set and connect Con-span units	
Backfill with flowable mortar	
• Materials	$ 22,352
20 ft x 9 ft x 64 ft Con-span units	
• Equipment	$ 3,282
TOTAL COST WITHOUT WINGWALLS	$ 31,215
• Labor to place optional wingwalls	$ 2,666
Form and pour wingwalls	
• Materials	$ 1,670
Reinforcing steel, ties and concrete	
• Equipment	$ 410
Air compressor for drilling holes	
Generator	
TOTAL COST FOR CONCRETE WINGWALLS	$ 4,746
TOTAL COST OF PROJECT	$ 35,961

3.7.2.2. Cost Information - Case Study

The Crawford County installation of an Air-O-Form culvert was completed in the summer of 1991. This arch culvert was designed for a 950 acre drainage area; preliminary calculations indicated a required drainage area of 110 ft². The culvert was designed for a 52 ft length, with a 9 ft semicircular arch section. The costs on this project are presented in Table 3.11.

3.7.3. Welded Steel Truss Bridge

3.7.3.1. Background

A low volume bridge replacement option which has been used extensively in the eastern U.S. is the welded steel Warren truss bridge designed by the U.S./Ohio Bridge Corporation of Cambridge, Ohio (32,54) shown in Fig. 3.27.

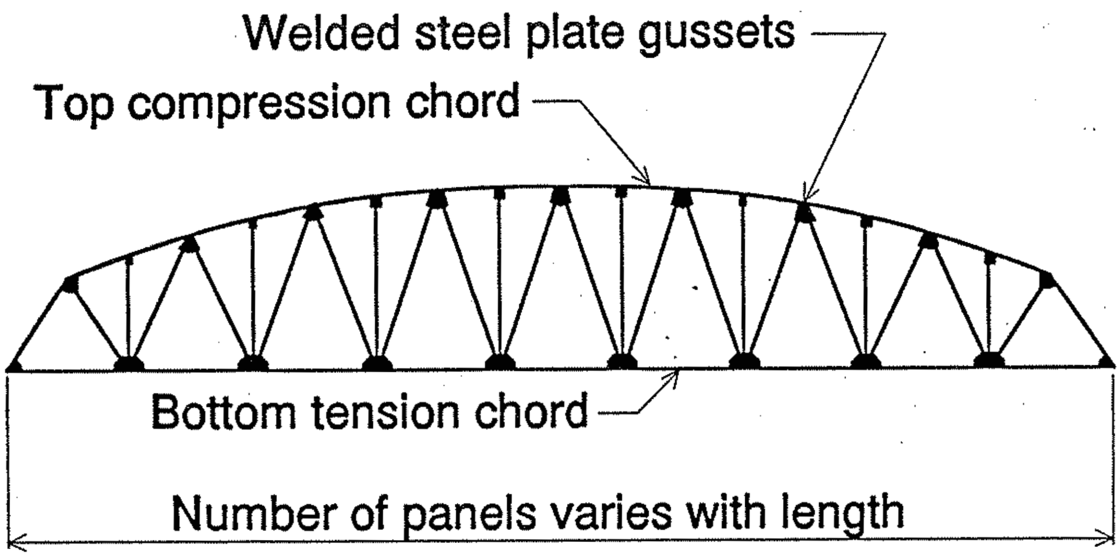


Fig. 3.27. U.S. bridge welded steel truss bridge.

Table 3.10. Winnebago County Con-Span culvert installation costs.

Item and Description	Item Cost	Total Cost
• Labor (county employees)		$ 6,217
• Substructure labor - contractor		$ 4,419
• Materials		$ 64,866
Con-span precast units	$ 33,764	
3/4 in. aggregate (4576 c.y.)	$ 19,449	
Concrete (101 c.y.)	$ 4,924	
Backfill aggregate (814 c.y.)	$ 3,461	
2½ in. aggregate (263 c.y.)	$ 1,117	
Plastic pipe	$ 970	
Eyebolts to connect units	$ 640	
Lumber for forms	$ 261	
Roofing tar (to seal joints)	$ 111	
Engineering fabric	$ 99	
Flowable mortar (2 c.y.)	$ 70	
• Equipment		$ 5,765
Rent CAT D4 bulldozer	$ 2,900	
Rent crane to set units	$ 2,865	
• Miscellaneous Expenses		$ 4,790
A.C.C. patching	$ 4,565	
Dust control for detour	$ 225	
TOTAL COST FOR PROJECT		$ 86,057

The U.S. Bridge is available in span lengths of 40 to 145 ft and is generally designed to carry an HS-20 loading, although it can be fabricated to carry heavier design loadings. The bridge can be shop painted any color, or is available in A588 weathering steel. The U.S. Bridge can be designed to accomodate cast-in-place concrete flooring or a wood plank deck supported by steel floor beams. The company, however, recommends the use of a corrugated metal deck with an asphalt riding surface.

The wood plank flooring is usually supplied in either 3 in. x 4 in. or 3 in. x 6 in. nominal sizes. These planks are placed transversely, with the smaller dimension horizontal and parallel with the roadway centerline. If the deck in a particular bridge is too wide for a plank to span the entire width, splices are permitted. A special expansion angle has been developed to allow for the thermal expansion of the wood plank flooring.

Table 3.11. Air-O-Form culvert installation costs.

Item	Quantity	Cost
Concrete, Footing & Headwall	65.8 c.y.	$11,844
Concrete, Arch	36.2 c.y.	$24,254
Excavation, Class 10 Channel	300 c.y.	$900
Excavation, Class 20	480 c.y.	$3,360
Granular Material, place only	71 tons	$213
Mobilization	Lump Sum	$2,000
Piling, Steel Sheet	435 s.f.	$4,350
Steel, Reinforcing, Footing & Headwall	5,870 lb.	$2,348
Steel, Reinforcing, Arch	6,235 lb.	$2,494
TOTAL COST FOR PROJECT		$51,763

3.7.3.2. Cost Information - Case Study

Albany County, NY currently has three U.S. Bridge welded steel truss bridges in service. The rapid installation of the bridge system was a primary reason for using the system. A small crew (5-8 people) were able to install two welded steel truss bridges in approximately three weeks. Recently, a 47 ft long, 24 ft wide welded steel truss bridge was installed to replace an existing structure. This installation was designed for HS-20 loading and utilized a wood plank floor system. A summary of the cost of this particular installation is provided in Table 3.12.

3.7.4. Prestressed Concrete Beam Bridge

3.7.4.1. Background

There are numerous examples of precast, prestressed concrete sections which are suitable for low-volume bridge replacements. The standard bridge sections which have been developed by the Prestressed Concrete Institute (PCI) (60) and AASHTO are well-known and have been used extensively throughout the United States (see Fig. 3.28). The majority of these sections were designed for long-span bridges and so are usually over-designed for a low volume bridge application. The AASHTO shapes are particularly inefficient, since they were developed several years ago when full prestressing (no tensile stresses at service load) was considered essential (55).

In addition, many prestress plants have designed their own non-standard sections. The majority of these non-standard sections are not patented, so that in most instances another prestress plant can obtain the necessary dimensions and enter bids on a proposed project. One significant benefit of many of these non-

Of all the shapes being used, the bulb tee is the most efficient. On the other hand, the double and multi-stemmed tees and the channel sections have the advantage of being more stable during handling and placing, and therefore are generally preferred by contractors (82). Sections which have nearly vertical flush sides, such as the box beam, can be connected transversly by bolting through the legs, which eliminates the need for intermediate diaphragms.

One section which takes advantage of this configuration is the "OK" bridge system, which was developed at Oklahoma State University. This system is discussed in greater detail later in this report (see Sec. 3.7.6).

3.7.4.2. Design Criteria

The actual design of a prestressed concrete girder bridge is relatively straight-forward. Several references are available to assist the practicing engineer with the design procedure (62,63). Brief guidelines for the design of a prestressed concrete double-tee follow. All dimensions and section properties are general values. Because each prestressed concrete manufacturer provides slightly different products, the exact values may differ slightly.

A design aid has been developed by the PCI to simplify the selection of a prestressed double-tee section. Table 3.14 provides the section properties for a number of standard size prestressed double-tee girders (see Figure 3.29).

Table 3.14. Section properties - prestressed double-tee girders.

	Dimensions (in.)*						Wt.	Area	I_x	S_b
	W	D	T	A	C	E	(lb/ft)	(in²)	(in⁴)	(in³)
LIGHT	60	27	5	4.50	8.00	36	599	575	33,740	4020
	72	23	5	4.50	6.50	36	582	558	21,366	3345
	72	27	5	4.50	8.00	36	662	635	35,758	4560
	96	27	5	3.75	5.75	48	718	689	32,888	5171
	96	35	5	3.75	6.50	48	820	787	72,421	8230
HEAVY	60	36	6	6.00	8.00	30	812	780	90,286	7334
	72	35	5	6.00	9.75	48	876	840	90,164	7706
	84	35	5	6.00	9.75	48	938	900	95,028	8569
	96	35	5	6.00	9.75	48	998	960	99,299	9412
	72	27	5	7.00	9.75	48	761	731	45,084	5060
	84	27	5	7.00	9.75	48	824	791	47,486	5640
	96	27	5	7.00	9.75	48	886	851	49,566	6196
	72	21	5	7.75	9.75	48	671	644	22,720	3298
	84	21	5	7.75	9.75	48	733	704	23,903	3666
	96	21	5	7.75	9.75	48	796	764	24,920	4019

*See Fig. 3.29.

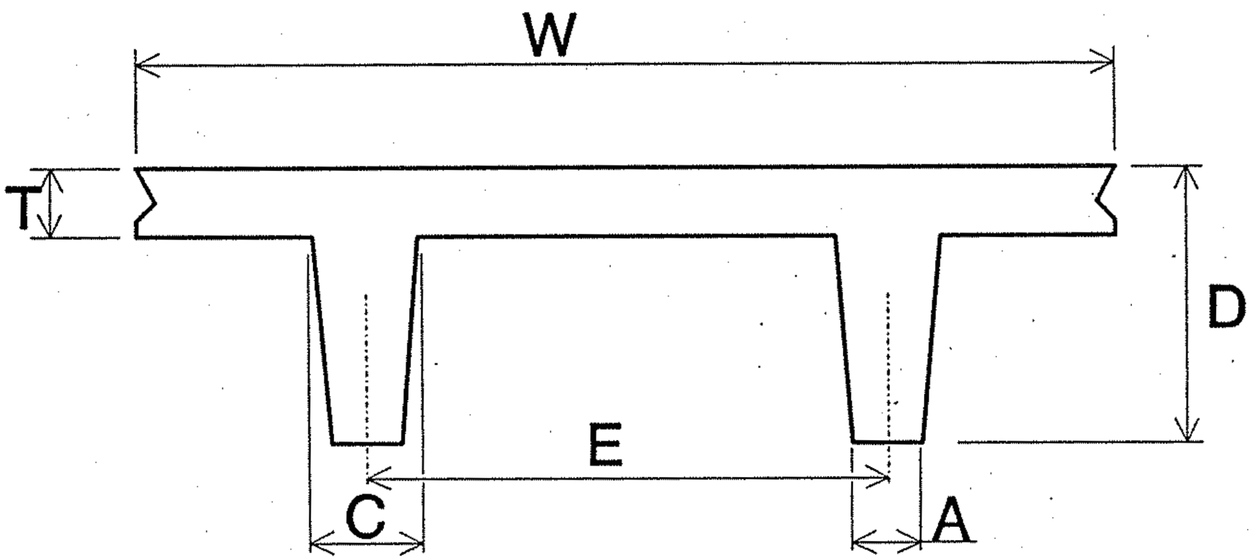


Fig. 3.29. Standard prestressed concrete double–tee bridge girder.

3.7.4.3. Cost Information: Case Study

A 124 ft long by 24 ft wide two-span continuous prestressed concrete double-tee bridge was installed in Washington County, Iowa in 1988. Costs incurred by the county are presented in Table 3.15.

3.7.5. Inverset Bridge System

3.7.5.1. Background

A unique method of utilizing the best features of both steel and concrete has been developed by Grossman and Keith Engineering of Norman, OK. The Inverset bridge (see Fig. 3.30) is a proprietary system which is cast upside down to utilize the compressive strength of concrete and the tensile strength of structural steel.

Table 3.15. Prestressed double-tee beam installation costs.

Item Description	Amount	Cost
Concrete, Structural	143.2 c.y.	$14,833
Reinforcing steel	27,133 lbs.	$6,783
Prestressed Concrete Double Tee 56 ft - 4½ in.	6 only	$27,000
Prestressed Concrete Double Tee 67 ft - 4½ in.	6 only	$31,500
Steel piling HP10x42 - furnish	1305 l.f.	$15,660
Steel piling HP10x42 - drive	1305 l.f.	$3,263
Steel piling HP10x42 - encase	159.4 l.f.	$4,782
Excavation, Class 20	85 c.y.	$680
Excavation, Class 21	16 c.y.	$320
Excavation, Class 10, channel	951 c.y.	$1,902
Rail, concrete barrier	284 l.f.	$4,260
Removal of existing structures	lump sum	$5,000
Mobilization	lump sum	$3,000
TOTAL COST FOR PROJECT		$118,983
COST PER SQUARE FOOT		$39.98/ft²

In precasting, the forms are suspended from steel beams. In this configuration, the weight of the forms, steel W sections, and wet concrete places compressive stress in what would be the bottom flange when the unit is inverted. When the concrete cures, the units are turned "rightside-up". The casting procedure results in the

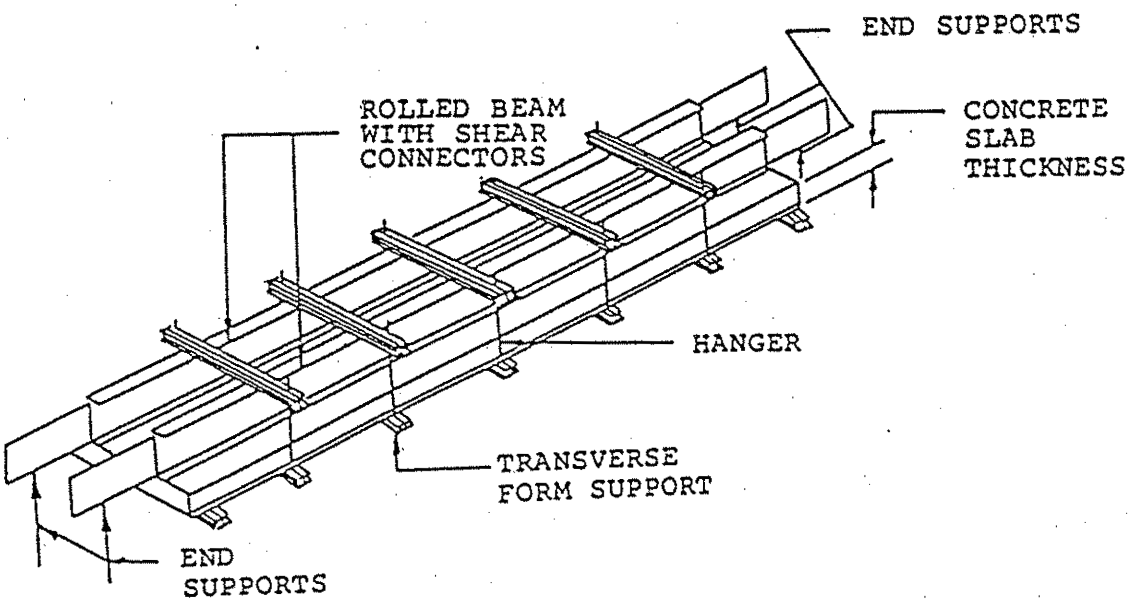


Fig. 3.30. Inverset bridge section during fabrication.

bottom flange having essentially zero stress. This stress condition, combined with the increased moment of inertia of a composite section, allows the Inverset system to carry additional load without overstress.

When the unit in turned over, longitudinal compressive stress is applied to the slab, which makes the deck extremely crack resistant and impervious to water intrusion (because air bubbles formed on what will eventually be the bottom of the slab).

3.7.5.2. Example

A sample Inverset design and fabrication is described in the following paragraphs. Particular attention should be paid to the stresses in the steel and concrete during the various steps of fabrication. In the example, stresses will be computed at the following locations in the cross-section:

- Concrete - extreme compression fiber of the deck,
- Steel - top flange of steel girder (which contains shear connectors) when the unit is placed into service, and
- Steel - bottom flange of steel girder once unit is erected. This flange is in compression during fabrication and is in tension when in service.

A summary of the stresses at various stages of the life of an Inverset bridge section are presented in Table 3.17.

1. **Design criteria.**

Simply supported span = 34 ft

Width of one unit, W = 11.833 ft

Design live load: AASHTO HS-25

Steel girder: W24x55 ASTM A572 grade 50

Yield strength = 50 ksi

Section modulus = 114.5 in.^{3}

Form weight = 100 lbs/ft

Concrete strength, $f_c' = 5$ ksi

The allowable stresses in steel and concrete are:

Temporary stresses (See art. 9.15.1, AASHTO (2)):

Steel:

tension: $0.80\ F_y = (0.8)(50) = 40$ ksi

compression: $0.70\ F_y = (0.7)(50) = 35$ ksi

Final stresses:

Steel (See 10.32.1A, AASHTO):

tension: $0.55\ F_y = (0.55)(50) = 27$ ksi

compression: $0.55\ F_y = (0.55)(50) = 27$ ksi

Concrete (See art. 8.15.2.1, AASHTO):

$f_c = 0.40\ f_c' = (0.40)(5) = 2$ ksi

2. **Stresses due to beam weight.**

$$M = \frac{wl^2}{8} = \frac{(2)(55\ lb/ft)(34ft)^2}{(8)(1000lb/k)} = 15.90\ ft\text{-}k$$

$$f_b = \frac{M}{S} = \frac{(15.90ftk)(12in/ft)}{(2\ beams)(114.5in^3)} = 0.83\ ksi \quad \begin{matrix} \text{T} & \text{top} \\ \text{C} & \text{bott.} \end{matrix}$$

3. **Stresses due to form weight.**

$$M = \frac{wl^2}{8} = \frac{(100\ lb/ft)(34ft)^2}{(8)(1000lb/k)} = 14.45\ ft\text{-}k$$

$$f_b = \frac{M}{S} = \frac{(14.45\ ftk)(12in/ft)}{(2\ beams)(114.5in^3)} = 0.76\ ksi \quad \begin{matrix} \text{T} & \text{top} \\ \text{C} & \text{bott.} \end{matrix}$$

4. **Stresses due to weight of concrete.**

$$w = \frac{(7.50\ in.)(11.833\ ft)(150\ lb/ft^3)}{12\ in./ft} = 1109\ lb/ft$$

$$M = \frac{wl^2}{8} = \frac{(1109\ lb/ft)(34ft)^2}{(8)(1000lb/k)} = 160.26\ ft\text{-}k$$

$$f_b = \frac{M}{S} = \frac{(160.26\ ftk)(12in/ft)}{(2\ beams)(114.5in^3)} = 8.40\ ksi \quad \begin{matrix} \text{T} & \text{top} \\ \text{C} & \text{bott.} \end{matrix}$$

At this stage, the stress in the top and bottom flange can be expressed by:

$$f_{top} = 0.83 + 0.76 + 8.40 = 9.99 \text{ ksi T}$$

$$f_{bot} = 0.83 + 0.76 + 8.40 = 9.99 \text{ ksi C}$$

5. **Computation of composite section properties.** When the concrete has attained a strength of 2.00 ksi, the forms can be stripped. At this point, a composite member has been developed. The section modulus of the composite member is calculated by the transformed area method. The modular ratio, n, is taken as 7 for strength calculations and as (3 x 7) = 21 for creep effects.

(See Section 10.38.1, AASHTO (2)).

For n = 7:

I = 9,508.48 in^4, N.A. located @ 24.23 in.

(Note: N.A. is located 0.66 in. above top flange)

For n = 21: I = 7,516.02 in^4, N.A. located @ 21.06 in.

Note: N.A. is located within the steel girder.

A summary of section moduli for composite Inverset sections is presented in Table 3.16.

Table 3.16. Section moduli for composite Inverset sections.

Section Moduli for Composite Members (in.^{3})		
Location	n = 7	n = 21
Concrete (top of deck)	1,389.84	750.83
Steel girder (top flange)	14,137	2994.1
Steel girder (bottom flange)	392.45	356.89

6. **Removal of forms.** When the forms are removed, the load acting on the composite section is reduced. This reduced load reduces the stresses in the steel girders, and the concrete deck becomes prestressed. The bending moment caused by the weight of the forms is the same as computed earlier, M = 14.45 ftk.

The stress in the concrete, f_{conc}:

$$f_{conc} = \frac{M}{S} = \frac{(14.45\,ftk)(12in/ft)}{(21)(750.83in^3)} = 0.011\ ksi \qquad C$$

The stress in the top flange, f_{top}:

$$f_{top} = \frac{M}{S} = \frac{(14.45\,ftk)(12in/ft)}{2994.1\ in^3} = 0.06\ ksi \qquad C$$

The stress in the bottom flange, f_{bot}:

$$f_{bot} = \frac{M}{S} = \frac{(14.45\,ftk)(12\ in/ft)}{356.89\ in^3} = 0.49\ ksi \qquad T$$

7. **Units turned to upright position.** When the units are turned over to their upright position, the forces acting on the units are essentially reversed. The process of inverting the unit causes a change in stresses equal to twice the weight of the structure. The moment due to overturning of units can be computed as:

M = 2(15.89 ft-k + 160.25 ft-k) = 352.29 ft-k.

The stress in the concrete, f_{conc}:

$$f_{conc} = \frac{M}{S} = \frac{(352.29ftk)(12in/ft)}{(21)(750.83in^3)} = 0.268\ ksi \qquad C$$

The stress in the top flange, f_{top}:

$$f_{top} = \frac{M}{S} = \frac{(352.29\ ftk)(12in/ft)}{2994.1\ in^3} = 1.41\ ksi \qquad C$$

The stress in the bottom flange, f_{bot}:

$$f_{bot} = \frac{M}{S} = \frac{(352.29\ ftk)(12\ in/ft)}{356.89\ in^3} = 11.85\ ksi \qquad T$$

8. **Application of superimposed dead loads.** Superimposed dead loads are assumed to act uniformly over the bridge deck surface. Superimposed dead loads include such things as: curbs, utility lines, guardrails, and parapets or other additional weights. For purposes of design, the dead load of any future wearing course should be included in the superimposed dead load. For this example, assume a DL_{super} = 65 lb/ft².
   So, w_{super} = (65 lb/ft²)(11.83 ft wide) = 769.0 lb/ft
   The moment due to this DL_{super} is:

$$M = \frac{wl^2}{8} = \frac{(769.0\ lb/ft)(34ft)^2}{(8)(1000lb/k)} = 111.1\ ft\text{-}k$$

The stress in the concrete, f_{conc}:

$$f_{conc} = \frac{M}{S} = \frac{(111.1ftk)(12in/ft)}{(21)(750.83in^3)} = 0.085\ ksi \qquad C$$

The stress in the top flange, f_{top}:

$$f_{top} = \frac{M}{S} = \frac{(111.1\ ftk)(12in/ft)}{2994.1\ in^3} = 0.45\ ksi \qquad C$$

The stress in the bottom flange, f_{bot}:

$$f_{bot} = \frac{M}{S} = \frac{(111.1\ ftk)(12\ in/ft)}{356.89\ in^3} = 3.74\ ksi \qquad T$$

9. **Application of AASHTO design live load.** For this example, an HS-25 loading will be used. The maximum live load moment for a 34 ft span is 429.4 ft-k (see Appendix A, AASHTO Standard Specifications (2)). This design live loading must be adjusted for the wheel load distribution width and impact to determine the actual design moment.

The wheel load fraction can be calculated from 3.23.1, AASHTO, with the beam spacing taken as one-half of the width of the Inverset unit (See design criteria).

$$WLF = \frac{S}{5.5} = \frac{Width\ of\ unit}{(2)(5.5)} = \frac{11.83ft}{(2)(5.5)} = 1.076$$

The impact factor (See Article 3.8, AASHTO) can be computed as:

$$I = 1 + \frac{50}{L + 125} = 1 + \frac{50}{34 + 125} = 1.315$$

$$I = 1.315 \geq 1.300(\text{max.}) \therefore I = 1.300$$

The design live load moment, M_{LL}, is computed by:

$$M_{LL} = M\ x\ WLF x\ I = (429.4\ ft\text{-}k)(1.076)(1.300) = 600.51\ ft\text{-}k$$

The stresses due to the live load moment must be computed with the modular ratio = 7. This increases the effective section modulus of each element of the composite member.
The stress in the concrete, f_{conc}:

$$f_{conc} = \frac{M}{S} = \frac{(600.5ft\text{-}k)(12in/ft)}{(7)(1389.84in^3)} = 0.741\ ksi \qquad C$$

The stress in the top flange, f_{top}:

$$f_{top} = \frac{M}{S} = \frac{(600.5\ ft\text{-}k)(12in/ft)}{14{,}137\ in^3} = 0.51\ ksi \qquad T$$

The stress in the bottom flange, f_{bot}:

$$f_{bot} = \frac{M}{S} = \frac{(600.5\ ft\text{-}k)(12\ in/ft)}{392.45\ in^3} = 18.36\ ksi \qquad T$$

This example problem illustrates that at no time during fabrication, installation, or service load conditions does the Inverset cross section exceed the allowable stress and that at final service load conditions, a compressive stress of 1.105 ksi exists in the concrete. This final compressive stress serves to reduce deck cracking and the intrusion of water.

Load transfer between adjacent units is provided by steel diaphragms which are bolted in place after the units are set in place. The longitudinal joints are sealed with a non-shrink grout in conjunction with an elastomeric concrete sealer (see Fig. 3.31).

The technique of casting Inverset bridge decks upside down allows the manufacturer to incorporate a number of deck finishes at minimal additional cost. If the desired finish is smooth (as when a waterproof membrane and asphalt overlay are used), the concrete bed is constructed with smooth finished plywood. If

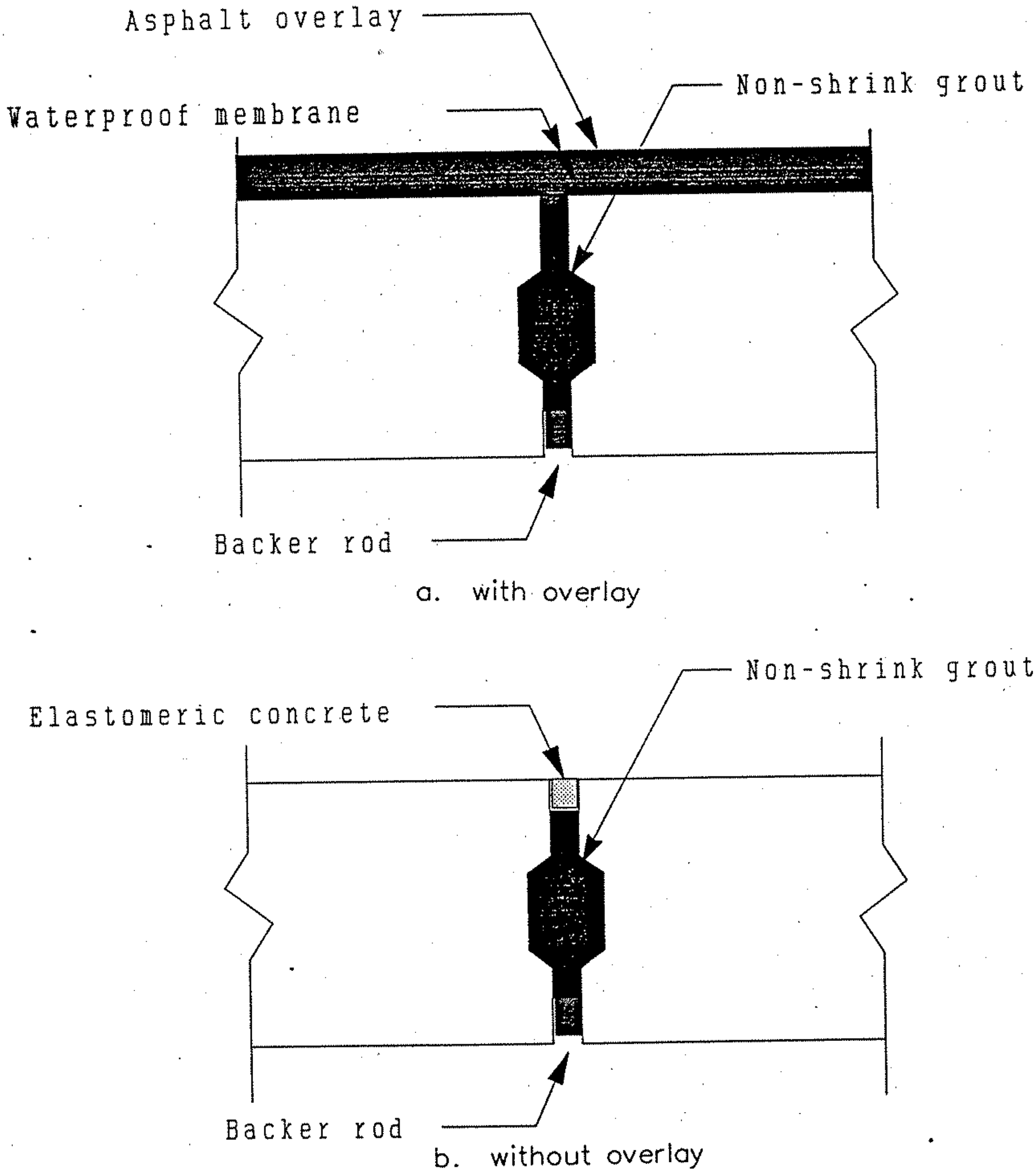


Fig. 3.31. Longitudinal joint details for Inverset bridge.

a textured wearing surface is desired, a urethane form liner is included before the concrete is placed. A natural finish may be achieved by using the standard coarse sandblast form liner and sawing transverse grooves after deck erection at an estimated cost of $0.50/ft^2. For additional information on the Inverset system, the reader is directed to Ref. 31.

Table 3.17. Inverset bridge section stresses over life of bridge.

Stresses in various elements of composite section due to fabrication and service loads						
Load	Bottom Flange		Top Flange		Concrete	
	Stress (ksi)	Total (ksi)	Stress (ksi)	Total (ksi)	Stress (ksi)	Total (ksi)
Beam Weight		0.83 C		0.83 T		
Form Weight		1.59 C		1.59 T		
Concrete		9.99 C		9.99 T		
Remove forms		9.50 C		9.93 T		0.011 C
Invert		2.35 T		8.52 T		0.279 C
Super. DL		6.09 T		8.07 T		0.364 C
Live Load	18.36 T	24.45 T	0.51 T	8.58 T	0.741 C	1.105 C

3.7.5.3. Cost Information: Case Study

A series of case studies are provided for the Inverset bridge system. Several of the Inverset bridges have been installed in the state of Texas as complete design-build, or turnkey, projects. Cost information for these projects have been provided by Steele Construction Company. The costs for these projects includes the following items:

- Royalty fee paid to the designer
- Engineering with stamped plans
- Demolition of existing bridge
- Pile driving (using 16 in. square precast concrete piles)
- Pile caps
- Deck
- Guardrail

- Embankment at ends of bridge (no deck overlay)
- Bonding

In addition, similar components have been utilized for all bridges:

- 3 pile bents
- Pile caps: 2 ft-3 in. x 2 ft x 28 ft
- Abutments: 2 ft-3 in. x 2 ft x 28 ft (with backwall)
- 7 in. concrete deck
- Type T-6 guardrail with safety end treatments

A summary of the project costs is presented in Table 3.18.

Table 3.18. Inverset bridge installation costs.

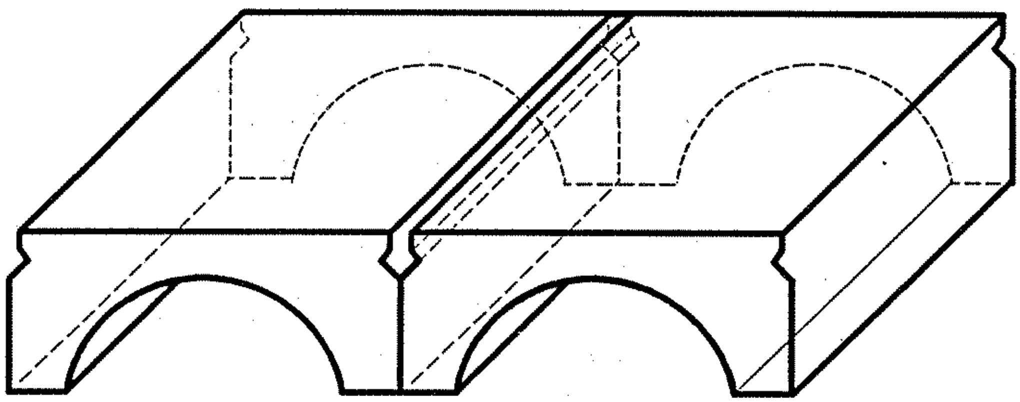
Project Name	Span Lengths	Bridge Width	Pile Length (ft)	Project Cost ($)	Cost ($/ft²)
Whipporwill Road	3 x 30′-0"	26′-1"	40	79,730	33.96
Stidham Road	30′-40′-30′	26′-1"	40	88,589	33.96
Nichol Road	4 x 42′-10"	26′-1"	45	150,961	33.78
Walnut Creek Road	4 x 37′-10"	26′-1"	45	148,775	37.69
Uncle Glen Road	4 x 45′-3"	18′-1"	50	147,000	44.91
Humble Pie Rd.	2 x 45′-8"	28′-1"	35	85,900	31.43
Brazos River	10 x 40′-0"	16′-0"	25	201,253	31.44

3.7.6. Precast Multiple Tee Beam Bridge

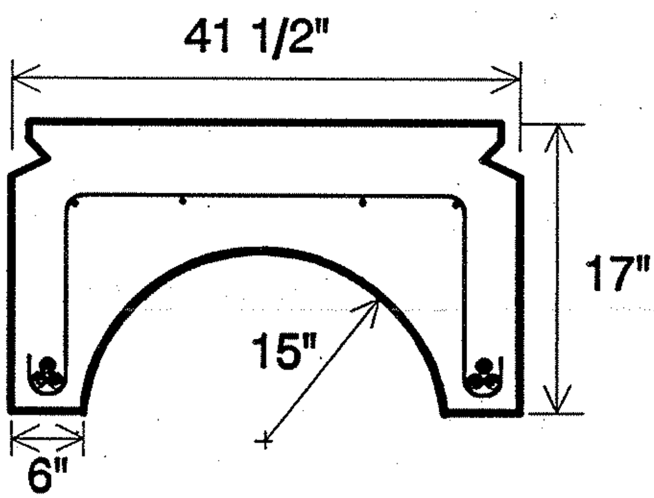
3.7.6.1. Background

A non-prestressed double tee beam girders has been developed by the Oklahoma State University Center for Local Technology which can be fabricated by local crews during the slack time of the year. The "OK" girders can be fabricated with reuseable steel forms that county crews can construct from standard structural shapes. This system offers a significant reduction in material costs and construction time over comparable alternatives. Oklahoma State University believes that the design can provide savings of "at least 15 percent" over conventional designs.

The "OK" bridge girder is a modified double tee reinforced concrete beam 17 in. deep by 41½ in. wide (see Fig. 3.32) which can carry an HS-20 loading over spans of 20-25 ft. The beams are bolted together at third points with 1 in. diameter threaded rods to form various widths; seven of the units bolted together will provide a width of 24 ft - 2½ in. The shear key at the top corners of each girder provides for



Adjacent precast sections



Typical section view

Fig. 3.32. "OK" precast multiple tee beam section.

shear transfer, and when filled with a non-shrink grout, prevents independent movement of adjacent girders. The two exterior girders are designed with connections for guardrail.

Each "OK" bridge girder weighs 5.67 tons and contains 2.6 cu. yd. of 3500 psi concrete. The top face of the girder is rough-finished providing ready-made wearing surface, although the design provides for a 20 psf wearing surface.

Several counties in Indiana have been using the "OK" bridge system for the past two years and report great success. The Daviess County, Indiana Highway Supervisor estimates the design will save the county a half-million dollars in the next 10 years (17). For additional information on the "OK" bridge girder system, see Refs. (46,65).

3.7.6.2. Cost Information: Case Study

The cost information for the "OK" girder has been well documented by Daviess County officials. The cost of the project, including one-time expenses such as the concrete pads, rebar jigs and forming are shown in Table 3.19.

3.7.7. Low Water Stream Crossing

3.7.7.1. Background

One relatively low-cost bridge replacement option available to the county engineer is the Low Water Stream Crossing (LWSC). A LWSC is defined as a stream crossing structure that is designed and constructed so that it is overtopped by floods or high water several times a year (49).

Since a LWSC is a low structure with a simple substructure, it is relatively inexpensive to construct. Unfortunately, since it is overtopped by high water several times per year, a LWSC creates regular traffic detours and inconveniences. In addition, the damage caused by high water erosion necessitates frequent inspection and inexpensive repair. These features may make a LWSC an economical replacement structure for low volume rural roads especially in areas with broad floodplains or where the normal streamflow is quite shallow.

Listed below in decreasing order of complexity and expense are three types of low water stream crossings:

- Low water bridge,
- Vented ford (a dip with vent or drain pipes),
- Simple ford or dip.

A low water bridge can be constructed by lowering the vertical alignment of the approaching roadway and constructing the bridge deck so normal stream flow can pass beneath. A low water bridge is especially suitable in areas where the potential excessive debris exists or where environmental conditions do not allow alterations of the existing streambed.

A vented ford is a dip in the existing roadway grade with pipes installed under the roadway to allow for day-to-day streamflow. A vented ford is the generally preferred alternative when the normal depth of the stream exceeds 4 to 6 in.

Table 3.19. "OK" multiple-tee beam bridge girder project costs.

Item or Description	Cost
Pour 40' x 14' x 6" concrete pad:	
Concrete (13.25 c.y., 3500 psi)	$ 563.13
Trowel machine rental	$ 30.00
Anchor bolts, assorted hardware	$ 38.58
Reinforcing steel	$ 146.00
Labor (4 men, 1 day)	$ 300.00
TOTAL COST FOR PAD	$ 1,071.71
Build steel form and rebar :	
Materials	$ 1,795.67
Machine shop (cut, drill, weld steel)	$ 99.00
Labor (80 hours - form, 40 hours - place steel)	$ 1,125.00
TOTAL COST FOR FORM AND REBAR	$ 3,019.67
TOTAL ONE-TIME COSTS FOR "OK" SYSTEM	$ 4,091.38
Pour interior bridge beam (5 per bridge):	
Reinforcing steel	$ 243.36
Concrete (2.75 c.y., 4000 psi)	$ 129.25
Miscellaneous materials	$ 6.79
Contract labor to form rebar	$ 49.00
Labor to build rebar cage (2 men, 2 hours)	$ 40.00
Labor to pour (4 men, 1 hour)	$ 40.00
TOTAL COST FOR INTERIOR BEAM	$ 508.40
Pour exterior girder (2 per bridge):	
Same materials/labor as for exterior beam	$ 508.40
Exterior bearing plates (4 per beam)	$ 16.56
Reinforcing steel	$ 2.00
TOTAL COST FOR EXTERIOR BEAM	$ 526.96
Materials to bolt beams together	$ 185.00
TOTAL COST FOR ONE BRIDGE DECK	$ 3,780.92

A simple ford is constucted by lowering the approach grades to the steambed level and providing some sort of unsurfaced crossing. Numerous improvements can be made to this arrangement by providing an ACC or PCC paved crossing or building some type of end walls. It is recommended that the user place reflective markers to delineate the edges of the improved crossing.

3.7.7.2. Design Criteria

A risk-based design approach has been suggested for the selection of LWSC locations. Unfortunately, a detailed risk analysis would require a significant amount of case study data which has not yet been compiled. A simplified selection criteria was developed in Ref. 49 and is shown in Table 3.20. The possibility of loss of human life criteria noted in Table 3.20 is the most difficult to quantify and is also the most important criteria for a public works project.

Table 3.20. Low water stream crossing selection criteria.

Criteria	Most Favorable for LWSC	Least Favorable for LWSC
Average Daily Traffic	< 5 v.p.d.	> 200 v.p.d.
Avg. Annual Flooding	< twice/year	>10/year
Average duration of traffic interruption during high water	< 24 hours	> 3 days
Extra travel time for detour	< 1 hour	> 2 hours
Possibility of danger to human life	< 1 : 1 billion	> 1 : 100,000
Possible amount of property damage	none	$1 million

3.7.7.3. Cost Information: Cast Study

Three low water stream crossings have been installed in Lucas County, Iowa in the past five years. A case study is presented for each of these structures. Labor on all three of the projects includes the removal of the existing thru-truss bridge at the site and the installation of the low water stream crossing.
The first installation was over the Chariton River in 1987. The structure is a simple vented ford, with four CMP pipes placed under the crossing. A summary of the costs on this project are shown in Table 3.21.

The second installation was placed over the South Otter Creek in 1987. This structure was another vented ford, with nine CMP culverts to allow stream flow beneath the structure. Costs on the project are shown in Table 3.22.

Table 3.21. Low water stream crossing costs - Example 1.

Item and Description	Item Cost	Cost
• Labor (county employees)		$ 3,062.41
• Equipment		$ 4,485.96
• Materials		$ 8,150.92
Rip rap	$ 2,690.65	
28 - 20 ft sheet pile	$ 2,618.00	
Concrete (29.5 c.y.)	$ 1,519.25	
4 - 18 in. x 28 ft CMP	$ 944.48	
Sand (90.13 tons)	$ 216.31	
Reinforcing steel	$ 107.92	
Welding supplies	$ 31.68	
Engineering fabric	$ 22.63	
TOTAL COST FOR PROJECT		$ 15,699.29

Table 3.22. Low water stream crossing costs - Example 2.

Item and Description	Item Cost	Cost
• Labor (county employees)		$ 4,231.51
• Equipment		$ 3,328.50
• Materials		$ 14,540.95
Rip rap & road stone (+ haul)	$ 5,648.70	
64 - 18 in. x 10 ft sheet piling	$ 2,992.00	
Concrete (46 c.y.)	$ 2,369.00	
9 - 18 in. x 30 ft CMP	$ 2,250.00	
Sand (125.16 T + hauling)	$ 738.45	
Lumber for formwork	$ 205.13	
Reinforcing steel (1,030 lb.)	$ 185.40	
Engineering fabric (128 c.y.)	$ 122.64	
Welding supplies	$ 29.63	
TOTAL COST FOR PROJECT		$ 22,100.96

The third installation was over the Wolf Creek in 1989. This installation included a skewed arrangement of seven CMP under the roadway surface. Costs on this installation are presented in Table 3.23.

Table 3.23. Low water stream crossing costs - Example 3.

Item and Description	Item Cost	Cost
• Labor (county employees)		$ 4,131.67
• Equipment		$ 2,729.00
• Materials		$ 14,652.10
Rip rap & hauling	$ 3,862.21	
80 - 18 in. x 10 ft sheet piling	$ 3,700.00	
Sand (incl. hauling)	$ 2,815.79	
Concrete (29.8 c.y.)	$ 2,079.00	
7 - 18 in. x 32 ft CMP	$ 1,847.16	
Reinforcing steel (812 lb.)	$ 273.78	
Engineering fabric (128 c.y.)	$ 122.64	
Lumber for formwork	$ 74.16	
TOTAL COST FOR PROJECT		$ 21,572.77

3.7.8. Corrugated Metal Pipe Culvert

3.7.8.1. Background

A corrugated metal pipe culvert (CMP) offers many advantages over other bridge replacement alternatives. A CMP is faster and easier to install than a cast-in-place concrete structure. In addition, there are no forms to set and remove, and no curing time is required. In most cases, county forces can install a CMP using ordinary county-owned equipment, which eliminates the need to hire an expensive outside crew or rent equipment. To simplify design, numerous standard CMP sections have been developed. These standard sections are mass produced which lowers the material costs.

There are actually several types of CMP culverts available. Many of these have been available for years, and have proven to be a cost-effective bridge replacement alternative. This report will concentrate on one particular type of CMP, the corrugated aluminum box culvert. Although only one type will be discussed in detail, many of the same considerations can be applied to corrugated steel pipes, and other CMP culverts. Much of the information printed in this report was developed with the assistance of Contech Construction Products, Inc. Although the culvert dimensions and available accessories discussed are specific to Contech there are other CMP manufactures which produce similar CMP. The design and installation procedure is similar for each of the different brands of CMP culverts, thus generalized instructions are presented.

The CMP culvert which offers the best potential for low volume bridge replacement is the aluminum structural plate box culvert. Corrugated aluminum box culverts (CABC) combine the low profile shape of rigid box culverts with the strength and flexibility of flexible structures. Contech Aluminum Box Culverts are available in standard sizes ranging from 8 ft-9 in. x 2 ft-6 in. to 25 ft-5 in. x 10 ft-2 in. The box culvert consists of aluminum structural plates and reinforcing ribs which are curved and bolt-hole punched at the plant.

One advantage the CABC has over other culvert types is the corrosion resistance. The aluminum alloys in the structural plates have an excellent resistance to corrosion, due to a thin oxide layer which forms on the metal surface when exposed to air. Many agencies are predicting a service life of more than 50 years for 16 gauge aluminum culverts when subjected to a normal environment. To minimize corrosion in the system, no dissimilar metals should be allowed to come in contact with the culvert. Although galvanized fasteners are acceptable, other metals must be insulated with non-conductive coatings.

One of the main advantages of an aluminum culvert system is its lightweight. Aluminum structural plate weights approximately 2 percent of a similar size reinforced concrete pipe. This lower weight reduces assembly and equipment costs and facilitates easier handling of the larger sections. The aluminum structural plates are usually light enough to be handled by a single worker, thus reducing labor costs. In addition, it is possible to assemble the culvert offsite and place it with smaller lifting equipment. This saves the cost of a heavy duty crane, and reduces the time the site needs to be closed for construction. All of these advantages make it possible to re-open the road more quickly, which reduces the inconvenience to the public.

3.7.8.2. Design Criteria

All Contech Aluminum Box Culverts are designed to meet or exceed an AASHTO HS-20 live loading. The actual structural design is performed at the Contech headquarters in Middletown, OH, and utilizes a finite element procedure to calculate the culvert load-carrying capacity.

The practicing engineer is required to perform the hydraulic design for the proposed culvert location. In following section, information on the hydraulic design of a CABC is presented. The footings used by a CABC require that the foundation soil be able to support a bearing stress of at least 4000 lb/sq ft. Existing foundation materials which are unable to support a load of this magnitude should be replaced before installing a CABC.

To retain the culvert's design load carrying capacity, the proper amount of earth cover must be maintained above the culvert. Contech recommends that the roadway above the structure be designed with either a flexible or rigid pavement. The minimum amount of cover *must* be maintained to prevent high-impact loads from being applied to the culvert. Particular attention should be paid to the shoulder of the proposed roadway, where a combination of substandard cover and an applied wheel loading can damage the culvert.

3.7.8.3. Design information

The actual hydraulic design of a CMP culvert is beyond the scope of this project. There are however, a number of sources available which discuss the hydraulic design procedure in detail. The reader is particularly encouraged to review the AISI Steel Drainage Handbook. The engineer should be aware of several optional design features of CABC.

Several types of footings are available for corrugated aluminum box culverts. A corrugated aluminum invert can be supplied for those installations which do not merit a full paved invert. The engineers at Contech *strongly recommend* that steps be taken to avoid water intrusion under the invert. Intrusion may be prevented by installing a toewall on the upstream end of the culvert. A concrete toewall may be cast on-site, or an aluminum flat sheet toewall is available. Note that most short-span culverts are goverened by inlet control. In such cases, the roughness coefficient of the invert does not affect the hydraulic capacity of the culvert, and the corrugated invert is often the most economical footing available.

In those locations where a full corrugated invert is used, it is essential that no backfill material intrude through the corrugations at the sidewall-invert interface. A scalloped closure plate is available to minimize the amount of this backfill infiltration. If the backfill material contains a significant amount of fine silts or sands, a layer of geotextile should be installed along the joint as well.

A pad footing is available for sites where the stream bed consists of non-erodible material; these are generally more economical than a full invert. A pad footing should be buried to a minimum depth of 12 in. to allow the inside soil to balance the pressure due to backfilling. It should be noted that the flow area of a box culvert includes the area from the crown to the invert or footing pads. If the pads are buried, the reduction in hydraulic capacity must be considered.

CABC's arrive at the jobsite ready for assembling. The parts are numbered and lettered for ease of erection. No unusual tools are required for assembly; drift pins and an impact wrench with a capacity of 150 ft-lbs. are considered mandatory. The keys to efficient assembly of the culvert are the use of a pair of drift pins to align the holes and proper bolt sequencing.

Site preparation, excavation, bedding, and backfill operations are essential to develop the maximum strength of any flexible culvert. The soil around the culvert must be sound granular material, placed and compacted following accepted procedures. The following remarks are specifically directed toward a CABC installation, but can be generalized to other types of culverts as well.

If a full aluminum invert is used, the trench bottom must be equal to the span of the culvert plus sufficient room to allow proper compaction. The bedding directly beneath the culvert sidewalls is particularly important. This region must receive proper compaction to develop the maximum load carrying capacity of the culvert. When toewalls are added, whether concrete or aluminum, a crosstrench must be included across the full width of the invert.

Installations utilizing aluminum foot pads require the excavation of two trenches. These must be of sufficient width to install the pads, and must be deep enough to avoid possible scour and frost heave of the pads.

The preparation of the pipe bedding is *critical* to both culvert performance and service life. Avoid any distortions that may create stress concentrations in the culvert. The bedding material must be free of rocks, frozen material, and organic material that might cause unequal settlement. Contech recommends that the bedding material be a well graded granular material.

The proper placement and compaction of backfill material is essential to developing the maximum strength of the culvert. The same basic restrictions apply to the backfill as to the bedding material. One should avoid anything which might create uneven compaction. The backfill material must be placed symmetrically on each side of the structure in 6-8 in. lifts. Each lift should be compacted to a minimum of 90 percent maximum density before applying more backfill material. During the backfilling operation, only small tracked vehicles should be near the culvert. If larger vehicles must be used, it may be necessary to increase the minimum cover depth to carry the temporary loading.

3.7.8.4. General cost information

A double 21 ft-2 in. by 8 ft-10 in. x 64 ft Con-Tech Corrugated Aluminum Box Culvert was installed by the city of Galesburg, IL in the summer of 1991. The existing timber bridge had to be removed and replaced after fire caused major damage to the deck and timber pile abutments.

The labor on the project was performed by a city maintenance crew, with the exception of the the removal and disposal of the existing creosote-treated bridge deck. Costs for the project are presented in Table 3.24.

3.7.9. Stress-Laminated Timber Bridge

3.7.9.1. Background

The stress-laminated timber (stresslam) system is a relatively new concept for timber bridge construction. In this system, sawn lumber laminations are placed vertically and squeezed or clamped together on their wide faces by the use of high strength steel post-tensioning rods. A stresslam bridge offers several significant advantages over conventional nail-laminated timber bridge systems. The deck superstructure can be prefabricated into panels, which can then be transported to the site and lifted into place. As long as the post-tensioning force is properly maintained, the stresslam timber deck will not delaminate over time. In the stresslam system, it is not necessary to have individual laminates span the entire length of the bridge. Since the load transfer between laminates is entirely by friction at the interface, all laminations do not need to be continuous. Butt joints of individual laminates are permitted within certain limitations (usually no more than 1 butt joint in 4 laminations within any 4 ft segment of deck width). The forces in a discontinuous lamina at

Table 3.24. Corrugated aluminum box culvert installation costs.

Item and Description	Item Cost	Total Cost
• Labor (city employees)		$ 8,410
• Removal of existing structure (contractor)		$ 8,000
• Materials		$ 42,294
Aluminum box sections	$ 24,450	
3/4" Aggregate (2875 c.y.)	$ 12,219	
Concrete (62 c.y.)	$ 4,650	
Miscellaneous materials	$ 975	
• Heavy Equipment Rental		$ 4,500
• A.C.C. wearing surface		$ 1,688
TOTAL COST FOR PROJECT		$ 64,892

a butt joint are transferred to the adjacent lamina through friction, which carry the forces past the butt joint (57). This behavior allows the use of shorter lumber and also allows longer spans to be cambered to offset dead load deflections.

The stress-laminated timber bridge system was developed by the Ontario (Canada) Mininstry of Transportation and Communication (Ontario M.O.T.C.) in the mid-1970s as a method of rehabilitating existing nail-laminted bridges (80). Traffic loading had caused timber deck members to separate, reducing the load distribution ability of the bridge deck and causing severe deterioration of the asphalt wearing surface. As shown in Fig. 3.33, the original technique used steel prestressing rods placed above and below the existing deck, which were then tensioned to compress the deck. The effects of the stress-laminating were dramatic and substantially increased the load-carrying capacity of the bridge (81).

Based on the successful application of this procedure to existing bridges, a method for constructing new bridges with a stresslam deck was developed. The system for new construction is similar to the original system, except that the prestressing rods are placed in transverse holes in the laminates (see Figure 3.34).

A series of studies have been undertaken by Ontario M.O.T.C., the U.S. Forest Products Laboratory and several universities to determine potential problems with the system and to develop a design procedure for the stresslam deck system. Only the most significant results of these investigations are briefly presented in this manual.

3.7.9.2. Design criteria

It was determined from load testing that stresslam bridge decks behave essentially like orthotropic plates, with different stiffnesses perpendicular and parallel to the laminations (9). This orthotropic plate behavior

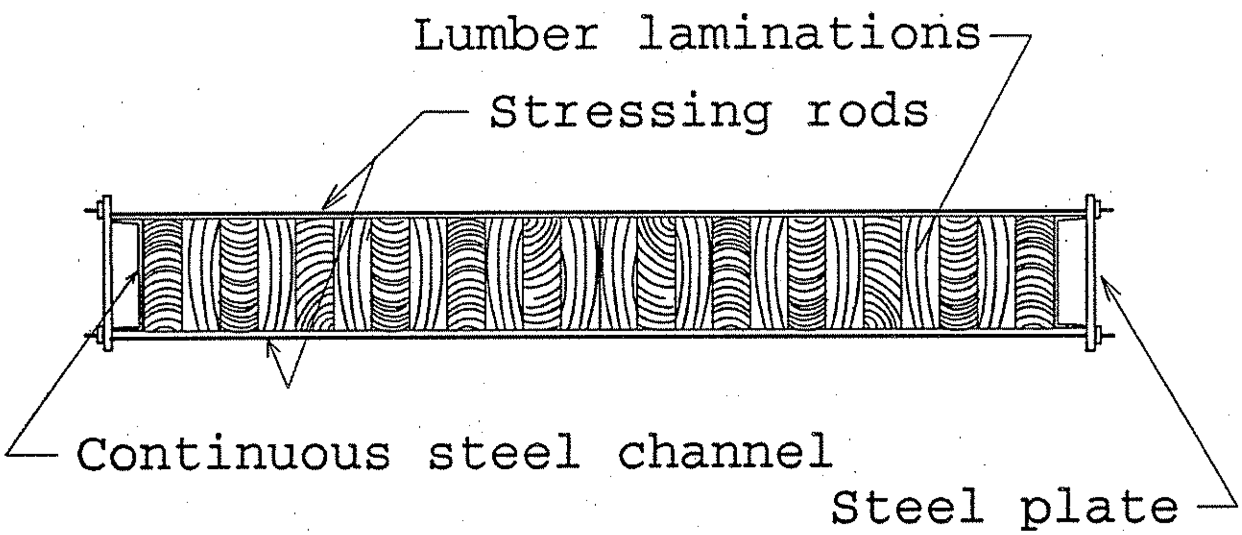


Fig. 3.33. Orginal stresslam bridge deck configuration.

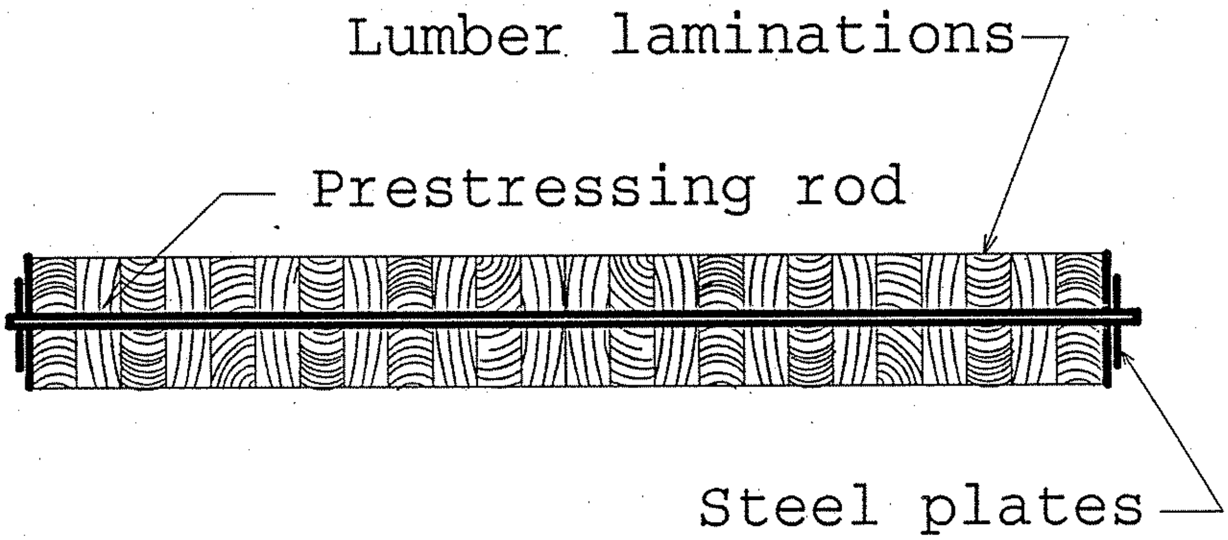


Fig. 3.34. Stresslam configuration for new construction.

allows the stresslam deck to distribute wheel loads laterally across some finite width of the deck and longitudinally to the supports.

Loss of prestress in the post-tensioning rods is the most significant problem with the stresslam system. Loss of the prestress force is a function of the ratio of the stiffness of the prestressing rod to the compressive strength of the timber; it is also affected by the sequence of tensioning the rods, the moisture content of the wood (which causes shrinking and swelling in the wood), the ambient temperature and also by the relative humidity (8). Loss of prestress in the deck is primarily the result of creep in the wood, as long as the moisture content of the wood is essentially constant (57). The 1983 Ontario Highway Bridge Design Code (OHBDC) reccomends that decks be restressed three times within the first five to eight weeks after assembly. A total loss of 50 percent of the original prestress force can still be expected over the life of the deck even after following the prestressing schedule given above.

The OHBDC requires that a steel channel bulkhead be placed along the longitudinal edge of the deck to distribute the stressing force uniformly along the length of the bridge and to provide a bearing surface for the post-tensioning rods. Oliva and Dimakis (57) determined that a bridge which uses steel bearing plates (16 in. x 16 in. by 1 in.) for each post-tensioning rod is more economical than the one that uses channels for bearing, is easier to construct, and maintains the desired prestress distribution.

Several studies have been made to determine possible ways to maintain the post-tensioning force in the rods. A system developed at the University of Connecticut (70) replaced several of the anchor plates with disc springs to minimize the loss of prestress force in the rods due to wood creep. In addition, this system allows the engineer to conveniently monitor the remaining force in the rods by measuring the deflection of the calibrated springs.

Numerous variations of the basic stresslam bridge have been recently developed. West Virginia University has constructed and is monitoring a "Stressed T" timber bridge (30), one in which total composite action is developed between the deck and stringers. The USDA Forest Service is investigating the use of a parallel chord timber truss to increase the longitudinal stiffness of the bridge.

The design procedure described herein is based on the AASHTO *Guide Specifications for the Design of Stress-Laminated Wood Decks* (1). The design of a stresslam bridge deck is governed by four design constraints:

- limit material stresses to allowable values.
- provide sufficient longitudinal stiffness to limit live load deflections.
- maintain a minimum uniform level of compressive prestress to prevent delamination.
- limit the compressive stress due to the post-tensioning force to acceptable limits.

3.7.9.3. General cost data

Very little cost data are available on the stress-laminated timber bridge. Many of the bridges have been constructed as part of a national bridge initiation, thus unit prices remain both relatively unknown and rather expensive. This is primarily due to the experimental nature of the construction and the lack of a competitive bid process.

A 34 ft x 24 ft stresslam bridge was constructed in Shelby County, Iowa through the U.S. Forest Service Timber Bridge Initiative in 1990. Because of the rather unique design and the lack of AASHTO guidelines, a consulting firm was hired to provide engineering for the project.

Wheeler Consolidated contracted with Shelby County to provide the design, fabrication and materials for the bridge. For ease of handling and construction, this project was designed with prefabricated timber deck panels. A lump sum fee of $39,400 was charged which included the engineering design,plans, specifications, lumber, fabrication, treatment, hardware and shipping.

Capital Construction provided all construction services on the project. This included the construction of 12 ft high timber abutments, the assembly of the stressed timber deck panels and two applications of the post-tensioning force. A breakdown of the construction costs is presented in Table 3.25.

The placement of a 2½ in. asphalt wearing surface and the third (and final) application of post-tensioning force was performed by Shelby County forces.

3.7.9.4. Design limitations

Several limitations apply to stress-laminated timber decks designed by this procedure:

- The deck is constructed of sawn lumber laminations that are placed edgewise between supports and stressed transversely with high strength steel rods.
- Deck width is assumed constant.
- Deck thickness is assumed constant and is not less than 8 in. nominal thickness.
- The deck is a rectangle in plan, or is skewed less than 20 degrees.
- End or intermediate supports are continuous across the entire width of the deck.
- Butt joints are permitted in the laminations provided no more than one butt joint occurs in any four adjacent laminations within a span of 4 ft.

Design loads for this procedure are based on AASHTO loading requirements and are limited to AASHTO Load Group I and IB, where design is essentially controlled by a combination of structure dead load and vehicle live load.

This design procedure is valid for sawn lumber laminations of the following species: Douglas Fir-Larch, Hem-Fir (North), Red Pine, Eastern White Pine, and Southern Pine. Design values for other species are currently being developed. All wood components are assumed to be pressure treated with an oil-type preservative prior to fabrication. To account for the load-sharing characteristics of the stresslam

Table 3.25. Construction costs for stresslam timber bridge.

Item	Quantity	Unit Price	Amount
TOTAL DESIGN COST	Lump sum	$39,400	$39,400
Stressed Timber Bridge (construct only)	Lump sum	$13,500	$13,500
Excavation - class 20	375 c.y.	$ 7.00	$ 2,625
Excavation - class 10	155 c.y.	$ 3.00	$465.00
Removal of exist. structure	Lump sum	$ 5,000	$ 5,000
Mobilization	Lump sum	$ 2,000	$ 2,000
Guardrail, thrie beam	37.5 l.f.	$ 13.00	$487.00
Guardrail, W beam	75 l.f.	$ 6.00	$450.00
Guardrail anchorages, RE-52	2 only	$500.00	$ 1,000
Guardrail anchorages, RE-27B	2 only	$165.00	$330.00
Guardrail posts, beam	24 only	$ 50.00	$ 1,200
Object marker, type 3	4 only	$ 60.00	$240.00
Delineator, single white	6 only	$ 20.00	$120.00
Object marker, triple yellow	8 only	$ 60.00	$480.00
Markers, guardrail	2 only	$ 10.00	$ 20.00
Drive creosoted piling	1200 l.f.	$ 3.00	$ 3,600
Creosoted test piling	Lump sum	$440.00	$440.00
E.W.O. - Guardrail #1	Lump sum	$325.00	$325.00
TOTAL CONSTRUCTION COST			$32,283
Install 2½" Wearing Surface			
Materials	Lump sum	$384.00	$384.00
Labor	Lump sum	$924.00	$924.00
Equipment	Lump sum	$366.00	$366.00
Third stressing of rods			
Labor	Lump sum	$292.00	$292.00
Equipment	Lump sum	$ 28.00	$ 28.00
TOTAL COUNTY FORCE ACCT.			$1,994
TOTAL COST OF PROJECT			$73,677
COST PER SQUARE FOOT			$90.29

system, allowable bending stresses have been increased by 30 percent for lumber graded Select Structural, and by 50 percent for lumber graded No. 1 or No. 2.

Prestressing elements are high strength steel rods which meet ASTM A722. The rods are placed through the laminations and are attached to anchorages with high strength nuts.

3.7.9.5. Design procedure

The design procedure discussed in this section has been automated by the use a spreadsheet. In the procedure which follows, the appropriate input values are indicated by a capital letter enclosed in parentheses. The letters shown correspond to the spreadsheet shown in Figure 3.35.

1. **Define the geometric requirements and the desired design loads.** Determine the required bridge span, L (**Input A**), and bridge width, W (**Input B**) which is the required roadway width plus curbs and railing, and the applicable design live loading to be applied (**Input C**). In many cases, a design live loading will be equivalent to an AASHTO HS-20 loading, depending on any local loading conditions which may exist. An asphalt wearing surface can be applied to the proposed bridge. The decision to use a wearing surface (**Input D**), and the thickness of this wearing surface (**Input E**), must be determined by the user.

2. **Select the species and grade of material to be used for the laminae and compute the allowable design material properties.** As noted earlier, the AASHTO guide specifications are applicable for Douglas Fir-Larch, Hem-Fir (North), Red Pine, Eastern White Pine, and Southern Pine. Although properties are available for other grades, primarily No. 1 grade is used. Material properties for the desired species can be found in Table 13.2.1A of the AASHTO Standards (2), and should be modified by the appropriate moisture content factors. Because of the load sharing capability of the stresslam system, the allowable flexural fiber stress can be increased by a factor of 1.30 for select structural grade and by a factor of 1.50 for No. 1 and No. 2 grade lumber (see Art. 13.2.7 of the Guide Specifications). The user should determine the species, grade, moisture conditions, and surface conditions of the proposed laminae (**Inputs F, G, H, and I respectively**). Based upon the species and grade selected, the spreadsheet computes the allowable stresses.

3. **Estimate a deck thickness and determine the wheel load distribution width.** For design purposes, a preliminary estimate of the deck thickness, t_d, can be made from the following:

span less than 10 ft	10 in.
span of 10 to 20 ft	12 in.
span of 20 to 30 ft	14 in.
span more than 30 ft	16 in.

Design of Stress-Laminated Timber Deck Bridges

Input deck geometric requirements:

Input bridge length, L =	25.00	ft	A
Input bridge width, W =	26.00	ft	B

Select design live loading:

1) HS 20-44
2) HS 15-44
3) H 20-44
4) H 15-44

Please enter number of your choice:	1	C

Select type and thickness of wearing surface:

1) No wearing surface
2) Ashpalt wearing surface

Please enter number of your choice:	2		D
Thickness of a/c wearing surface (if any):	3.00	in.	E

Select species and grade of material to be used for laminae:

1) Douglas fir-larch
2) Hem-fir (north)
3) Red pine
4) Eastern white pine
5) Southern pine

Please enter number of your choice:	1	F

Fig. 3.35. Stresslam timber deck spreadsheet, input parameters, and example problem.

Select species and grade of material for laminae (cont.):

6) Select structural
7) Grade #1
8) Grade #2

Please enter number of your choice: [7] G

9) All thicknesses surfaced dry or green and used at 19% max. M.C.
10) Nominal 4" or less in thickness, used at greater than 19% max. M.C.
11) Nominal 4" or less thickness, used at 15% or less max. M.C.
12) Nominal 5" or thicker, used where M.C. exceeds 19%

Please enter number of your choice: [9] H

13) Surfaced wood laminates
14) Rough sawn wood laminates

Please enter number of your choice: [14] I

Moisture content factor for Fb =	1.00
Moisture content factor for Fcp =	1.00
Moisture content factor for E =	1.00
Load sharing factor =	1.5

Allowable Bending stress, Fb' =	2.625 k.s.i.
Modulus of Elasticity =	1,800 k.s.i.
Perp. compression stress, Fcp'=	0.385 k.s.i.

Estimated deck thickness and computed wheel load distribution width:

Initial est. of deck thickness, td = *[16] in. (based on span length)

Number of continuous adjacent laminae in 4' length = [4] J
Butt joint adjustment factor, cbj = 0.8

Wheel load distribution width = 41.60 in.

Fig. 3.35. Continued.

Computed design live load and dead load moments:

Dead load of timber deck =	43,333	lbs
Dead load of curb, railing, etc.=	4,625	lbs
Dead load of wearing surface =	22,500	lbs
Total design dead load =	70,458	lbs
Design dead load moment =	220.18	ft-kips
Maximum design live load moment =	103.68	ft-kips
Total design moment (DL + LL) =	323.86	ft-kips

Computed *required deck thickness* based on allowable flexural fiber stress:

Effective section modulus, S =	1774.93	in.^3
Flexural bending stress, Fb =	2.19	k.s.i.
Allowable Bending stress, Fb' =	2.625	k.s.i.

Check: Fb < Fb' ? ***Deck is sufficient in flexure.***

Computed check of live load deflection:

Effective deck moment of inertia, I =	16329.39	in.^4
Live load deflection, DELTAL =	0.38	in.
Allowable live load deflection = L/500 =	0.6	in.

Check DELTAL < allowable deflection ? **Deck is sufficient for deflection.**

Computed dead load deflection and camber:

Dead load applied to deflection:	65,833	lbs
Long term DL deflection, DELTAD =	0.91	in.
Proposed design camber =	2.72	in.

Fig. 3.35. Continued.

Computed required level of prestress force:

Case A - Transverse bending:

Transverse bending moment, Mt =	708.77	in.-lbs/in.
Minimum prestress force, p =	16.61	p.s.i.

Case B - Transverse shear:

Transverse shear force, Vt =	158.08	lbs/in.
Coefficient of friction, mu =	0.45	
Minimum prestress force, p =	32.93	p.s.i.

Case B controls - minimum interlaminar prestress force = 32.93 p.s.i.

Check: Minimum prestress force must be >= to: 40.00 p.s.i.

Intial prestress force applied at construction = *100.00* p.s.i.

Select size and spacing for prestressing elements:

Input trial value for tendon spacing: 50.00 in. K

Area of steel must be >=	0.76	in^2
Area of steel must be <=	1.28	in^2

1) 5/8", 150 ksi tendon
2) 1", 150 ksi tendon
3) 1 1/4", 150 ksi tendon

Please enter number of your choice: 2 L

Area of steel rod, As =	0.85	in.^2	**Rod size is OK**
Force in prestressing tendon, Fps =	80,000	lb/in.^2	

Fig. 3.35. Continued.

Computed size of bearing plates

Input yield strenth of steel plates: 36,000 lb/in.^2 M

Required area of plate, Aplr = 207.8 in.^2

Input trial dimensions of bearing plate:

Longitudinal length, Lp = 16.00 in. N

Transverse width, Wp = 14.00 in. O

Check: 1.0 < Lp/Wp < 2.0 ? Ratio of Lp/Wp OK

Area of bearing plate, Apl = 224.00 in.^2

Check: Apl > Aplr ? Bearing plate has sufficient area

Bearing stress in timber due to plate = 357.14 lb/in.^2

Typical dimensions for anchorage plate (varies with manufacturer):

Longitudinal length of anchor plate, La = 6.50 in.

Transverse width of anchor plate, Wa = 4.00 in.

Thickness of anchor plate, ta = 1.25 in.

k based on relative plate widths, k1 = 5.00

k based on relative plate lengths, k2 = 4.75

Value of k for use in plate bending equation = 5

Minimum thickness of bearing plate, tp = 1.16 in.

Check bearing stress at abutments:

Input width of abutments, wabut = 12.00 in. P

Reaction due to dead load, RDL = 4,514 lbs

Reaction due to live load, RLL = 23,040 lbs

Bearing stress at abutments, fcabut = 55.20 lb/in^2 Bearing stress is OK

Fig. 3.35. Continued.

Summary of Design Values

Length, L =	25.00	ft			
Width, W =	26.00	ft			
Design live loading:	AASHTO	HS 20			
Lumber species:	Douglas fir-larch				
Lumber grade:	Grade #1				
Maximum moisture content:	19% maximum				
Lumber condition:	Rough sawn laminates				
Thickness of deck, td =	16.00	in.			
Stressing system:	1 in. dia., 150 ksi				
	Spacing =	50.00	in.		
Rod anchorage system:					
Yield strength of steel =	36.00	k.s.i.			
Bearing plate (inches):	16.00	x	14.00	x	1.16
Anchorage plate (inches):	6.50	x	4.00	x	1.25
Stresses and deflections:					
Bending stress, fb =	2.19	k.s.i.			
Allowable bending stress, Fb' =	2.63	k.s.i.			
Live load deflection, deltaL =	0.38	in.			
Allowable LL deflection, L/500 =	0.60	in.			
Dead load deflection, deltaD =	0.91	in.			
Design camber =	2.72	in.			

Fig. 3.35. Continued.

Summary of Design Values, cont.

Tensioning system:

Minimum prestress force, p =	40.00	p.s.i.
Force @ construction, pi =	100.00	p.s.i.
Force in stressing tendon, Fps =	80,000	lbs
Bearing stress @ anchorage =	357	p.s.i.
Bearing stress at abutment =	55	p.s.i.
Allowable bearing stress, Fcp' =	385	p.s.i.

Fig. 3.35. Continued.

The spreadsheet automatically computes this initial deck thickness based on these values. This initial thickness estimate may be revised if the ensuing calculations show a deck which is significantly over-designed. The spreadsheet deck thickness can be changed by over-writing the cell which contains the automatically computed value (**Input ***).

The wheel load distribution width, D_w, is taken as:

$$D_w = c_{bj}[tire\ contact\ width + 2t_d]$$

where the tire contact width is determined from Art. 3.30 of the AASHTO Specifications (2). The butt joint adjustment factor, c_{bj}, is determined from Art. 3.25.5.4 of the Guide Specifications (1) as:

$$c_{bj} = \frac{j}{(j+1)}$$

where j is the minimum number of continuous laminae in any four foot longitudinal length (**Input J**).

4. **Compute the design live and dead load moments.** The dead load of the deck is based on the assumed thickness, along with any additional dead load from a wearing surface (if employed), guardrail, curbs, and other fixtures. The total dead load moment should be calculated for a width of the deck equal to the wheel width plus twice the deck thickness.

   The maximum live load moment due to a wheel line should be calculated. A table has been developed which gives the maximum live load moment for various span lengths and design loadings. For AASHTO live load moments, see Appendix A of the AASHTO Standard Specifications (2); for Iowa legal truck live load moments, see Appendix B of this manual.

5. **Determine the required deck thickness based on allowable flexural stress under combined dead and live loads.** The dead load and live load moment diagrams should be combined to determine the maximum total bending moment, M_{tot}. In the case of a simple span bridge, the maximum moment is assumed to be the sum of the maximum dead load moment, M_{DL}, and maximum live load moment, M_{LL} (since M_{DL} and M_{LL} occur at different locations).
   In the case of a simple span bridge, the dead load moment at any position along the span is given by:

$$M_{DL} = w_{DL}\, x \left(\frac{L}{2} - \frac{x}{2}\right)$$

where w_{DL} is the uniform dead load over the wheel load distribution width, x is the position along the span, and L is the total span length. The maximum live load moment, M_{LL}, occurs when the center of gravity of the design wheel loads and the nearest heavy wheel load are positioned equidistant from the centerline of the span. An expression for the maximum live load moment and shear for an HS-20 loading has been developed and is presented in Table 3.26. The spreadsheet automatically calculates the design dead and live load moments based upon the span length, design vehicle and any wearing surface, etc. which has been included.

An idealized portion of the deck, with width equal to D_w, and thickness t_d, shall be assumed to resist the total maximum moment. The flexural stress, f_b, is given by:

$$f_b = \frac{M_{tot}}{S}$$

where the effective section modulus, S, is given by:

$$S = \frac{D_w c_{bf} t_d^2}{6}$$

If the calculated flexural stress exceeds the allowable value computed in design Step 2), either the deck thickness, t_d, must be increased (see design Step 7)), or a higher grade of lumber (one which has better material properties) must be used. If f_b is significantly less than the allowable stress value, a thinner deck or a lower-grade material may be more economical. In any case, changes should not be made until the live load deflection is checked.

6. **Check the live load deflection.** Live load deflection, Δ_{LL}, is computed by standard elastic analysis methods for one wheel line of the design vehicle. The deflection is due to this wheel line applied over a width equal to D_w and modified by a factor of 1.15 (See Appendix A, Ref. 1. The live load deflection is given by:

$$\Delta_{LL} = \frac{\textit{Deflection coeff.}}{1.15\, E' I}$$

where E′ is the modulus of elasticity of the laminae, corrected for moisture content. The deflection coefficient can be found in Table 16-8, Ref. 66, and the effective deck moment of inertia, I, is given by:

$$I = \frac{D_w c_{bf} t_d^2}{12}$$

Table 3.26. Maximum moments and shears for HS-20 loading.

Span Length, (ft)	Moment, (ft-kips)
0 - 23.9	8L
23.9 - 33.8	16L + 784/L - 224
33.8 - 145.6	18L + 392/L - 280
>145.6	$0.08L^2 + 4.5L$
Span Length, (ft)	**Shear, (kips)**
0 - 14	32
14 - 28	64 - 448/L
28-127.5	72- 672/L
>127.5	0.32L + 26

An abbreviated version of the deflection coefficient table is given in Table 3.27. To obtain the live load deflection for one wheel line in inches, divide the deflection coefficient by EI (lb-in^3). The live load deflection should be compared to the AASHTO allowable value of *L/500*. The spreadsheet automatically computes this deflection and compares it to the allowable value.

7. **Revise thickness if necessary.** If the flexural stress and/or live load deflection computed in steps 5) and 6) are significantly different from the allowable values, a new thickness should be assumed and the calculations for dead load, distribution width, flexural stress, and live load deflection repeated until acceptable values are attained. Note, any assumed thickness values should be taken as multiples of the common lumber dimension (7¼ in., 8 in., 9¼ in., 10 in., etc.).

   Spreadsheet users need to enter a larger deck thickness value (**Input ***) and the remaining calculations will be performed automatically.

8. **Calculate dead load deflection and camber.** The dead load deflection of the deck is calculated assuming the dead load weight from a width of the deck equal to the wheel width plus twice the deck thickness is resisted by a width of the deck equal to the distribution width, D_w. The dead load deflection, Δ_{DL}, can be computed as:

$$\Delta_{DL} = \frac{5\omega_{DL}L^4}{384\ E'I}$$

Table 3.27. Deflection coefficients for HS-20 live loading.

Span Length, (ft)	Deflection Coefficient
10	5.76×10^8
15	1.94×10^9
20	4.61×10^9
22	6.40×10^9
24	9.38×10^9
26	1.30×10^{10}
28	1.74×10^{10}
30	2.25×10^{10}
32	2.85×10^{10}
34	3.53×10^{10}
36	4.31×10^{10}
38	5.19×10^{10}
40	6.18×10^{10}
42	7.34×10^{10}
44	8.65×10^{10}
46	1.01×10^{11}
48	1.17×10^{11}
50	1.34×10^{11}
55	1.84×10^{11}
60	2.45×10^{11}
65	3.18×10^{11}
70	4.03×10^{11}
75	5.02×10^{11}

As mentioned earlier, one of the benefits of including butt joints in the design of the stresslam bridge is that camber can be built-in to offset the effect of dead load deflection. If camber is to be used, Ritter (66) has recommended the design camber should be $3\Delta_{DL}$. This calculation also is performed automatically by the spreadsheet.

9. **Determine the required level of prestress to be used in laminating.** In the Guide Specifications (1), the required prestress force to be used in laminating is given in Article 13.11.1. This force is the uniform force between the laminates, not the force in the individual post-tensioning tendons.

Two conditions must be satisfied by the post-tensioning system. First, sufficient prestress force must be applied to offset the effect of transverse bending stresses. The amount of force, p (in psi), required to satisfy this first condition is:

$$p = \frac{6\ M_T}{t_d^2}$$

where the transverse bending moment in in.-lbs/in., M_T, is given by:
for one lane bridges:

$$M_T = \frac{1.54\ M_x}{1000\ (c_{bj})^{1/4}}\left(\frac{b}{L}\right)$$

for two lane bridges with L < 50 ft:

$$M_T = \frac{0.79\ M_x}{1000}\sqrt{\frac{b}{L}}$$

and where M_x = the longitudinal moment caused by a single wheel line in in.-lbs. and b = half of the bridge deck width in inches.

The second condition which must be satisfied is that sufficient prestress force must be applied to resist any interlaminar slippage due to transverse shear. The required prestress force, p, in psi, shall be computed as:

$$p = \frac{1.5\ V_T}{\mu\ t_d}$$

where μ is the coefficient of interlaminar friction and is equal to 0.35 for surfaced wood and 0.45 for rough sawn wood. The transverse shear, V_T, in lbs/in., shall be taken as:

$$V_T = \frac{P}{1000}\left(10.4 - \frac{b}{L}\right)$$

where P is the maximum single wheel load in lbs.

The initial prestress applied to the deck must compensate for prestress losses due to creep and relaxation. The initial minimum compressive force, p_i, shall be equal to 2.5p. The deck shall be prestressed to the same level during the second week and again between the fifth and eighth week after the initial stressing. The spreadsheet automatically performs these calculations.

10. **Select a spacing for the prestressing elements.** The spacing of the prestressing elements should be based on the span length and the maximum allowable spacing of 60 in. (see Article 13.11.2.2, AASHTO (2)). Table 3.28 can be used to determine an approximate post-tensioning tendon spacing **(Input K)**.

Table 3.28. Approximate spacing for prestressing rods.

t, (in.)	Rod spacing, (in.)					
	⅝" in. ϕ rods		1" in. ϕ rods		1 ¼" in. ϕ rods	
	Max.	Min.	Max.	Min.	Max.	Min.
7¼	41	24	-	-	-	-
8	37	22	-	-	-	-
9¼	32	19	-	-	-	-
10	29	18	89	53	-	-
11¼	26	16	79	47	-	-
12	25	15	74	44	-	-
13¼	-	-	67	40	99	59
14	-	-	64	38	94	56
15¼	-	-	59	35	86	51
16	-	-	56	33	82	49

11. **Size the prestressing elements.** The type of prestressing system to be used must be selected by the engineer. The most common means of prestressing utilizes high strength threaded steel rods.

Each element, spaced as determined in Step 10, must be able to provide the initial prestressing force for an area determined by the deck thickness times the element spacing. The compressive force in the rod is then calculated as the area (thickness x spacing) multiplied by the initial prestressing force. The minimum area of the prestressing element must satisfy the following equation (Eqn. 13-26 and 13-27, AASHTO (1)):

$$A_s = \frac{p_i\, s\, t_d}{f_s} \leq 0.0016\, s\, t_d$$

The limitation on total steel area is to control the loss of prestress due to creep in the timber. The engineer should select a prestressing element with a cross-sectional area that meets the above requirements (**Input L**).

12. **Size the bearing plates.** The compressive force carried by each prestressing element must be resisted by the timber immediately under the bearing plate. The required area for the bearing plate is determined by the following equation (Eqn. 13-28, AASHTO (1)):

$$A_{pl} = \frac{p_i\, s\, t_d}{F_{c_\perp}}$$

The engineer must provide the yield strength of the bearing plates (**Input M**) and should select a bearing plate with the required area and the proper ratio of length to width as shown below (**Input N, O**). Once a plate has been selected based on area, calculate the actual bearing stress. The bearing stress is calculated as:

$$f_{bp} = \frac{p_i}{A_{plate}}$$

The minimum bearing plate thickness should be computed from the following (Eqn. 13-29, AASHTO (1)):

$$t_p = \sqrt{\frac{3 f_{bp} k^2}{F_s}}$$

The factor k depends on the shape of the bearing plate and anchorage plate (if used) and is the greater of:

$$k = \frac{(W_P - W_A)}{2} \ or \ \frac{(L_P - L_A)}{2}$$

and where W_p, L_p are bearing plate dimensions in inches, W_A, L_A are anchorage plate dimensions in inches (if used). Figure 3.36 shows the dimensions required to determine the k factor. Once the area of the plates has been entered, the spreadsheet performs the remaining calculations.

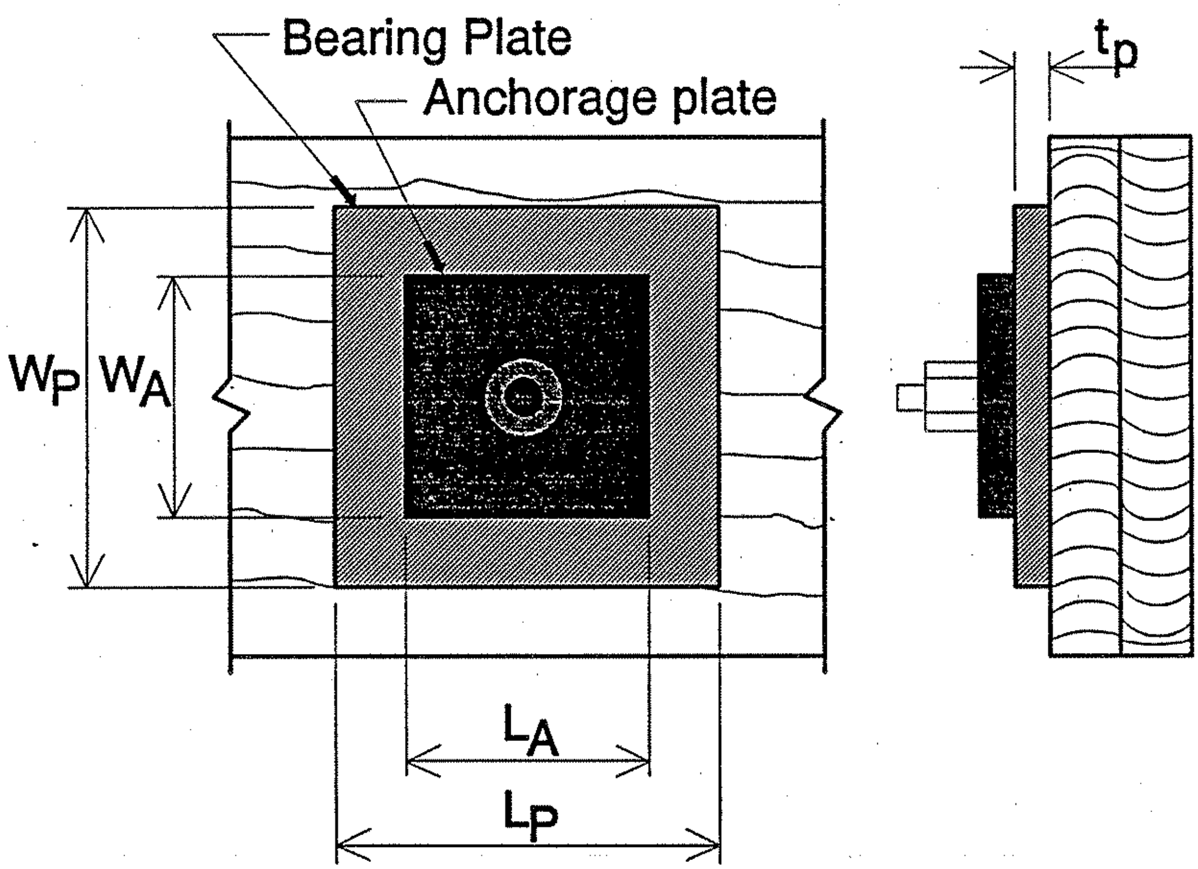


Fig. 3.36. Dimensions required to determine k factor.

13. **Check bearing stress at abutment.** The compressive stress on the abutment should be checked against the allowable compressive stress perpendicular to the grain, $F_{c\perp}$. For this type of bridge, a timber abutment cap beam over piles (either steel or timber) is normally used. The design of the piles is left to the designer. Assume a 12 in. wide capbeam unless a more exact value is known.

    The dead load reaction at the abutment, R_{DL}, is computed as one half of the total dead load on the structure from the deck, the wearing surface (if any), and any curbs, rails or hardware which is used.

    The live load reaction to the abutment, R_{LL}, can be found in Appendix A, AASHTO (2) for the appropriate span length.

3.7.9.6. Example

For illustration, an example stresslam bridge will be designed. The spreadsheet input and output for this example is shown in Fig. 3.35. The following criteria will apply to the example bridge:

Length = 25 ft c-c of bearings

Roadway width = 24 ft

Bridge width = 24 ft roadway + 2 ft allowance for curb and/or rail = 26 ft

Existing abutments - bearing length = 24 in.

1. **Define geometric requirements and design loads.** Set L = 25 ft **(A)**, W = 26 ft **(B)**

   Use HS-20 live loading **(C)**:

   From Table 16-8, Ref. 47:

   Maximum LL moment = 103.68 ft-k

   Maximum LL reaction = 23.04 k

   LL deflection coefficient = 1.11×10^{10}

   Use 3 in. asphalt overlay for wearing surface **(D and E)**:

   Calculate DL of wearing surface:

   $DL_{wearing} = (37 \text{ ft})(26 \text{ ft})(3/12)(150 \text{ lb/ft}^3) = 34{,}425$ lbs.

2. **Select species and grade of timber to use.** For this example, choose Grade #1 Douglas fir-larch **(F, G, H and I)**:

   From the 1989 AASHTO specifications (2), Table 13.2.1A:

   $F_b = 1750$ psi

   $F_{c\perp} = 385$ psi

   $E = 1{,}800{,}000$ psi

Apply modification factors to material properties:

$F_b' = F_b\, C_{MFb}\, C_{LS}$

$F_{c\perp}' = F_{c\perp}\, C_{MFc\perp}$

$E' = E\, C_{ME}$

The moisture content factor, C_M can be found in Table 5.7, Ref. (66). Each of the three design properties can have a different value for C_M, so caution is advised.

The load sharing factor, C_{LS}, is determined by the grade of lumber used and can be found in Sec. 13.2.7, AASHTO (1).

For this example:

$C_{LS} = 1.50$ $\quad$ $C_{MFb} = 1.00$

$C_{MFc\perp} = 1.00$ $\quad$ $C_{ME} = 1.00$

The revised design properties for this example:

$F_b' = (1750)(1.00)(1.50) = 2625$ psi

$F_{c\perp}' = (385)(1.00) = 385$ psi

$E' = (1{,}800{,}000)(1.00) = 1{,}800{,}000$ psi

3. Estimate deck thickness and determine wheel load distribution width.

Span = 25 ft thus estimated $t_d = 14.00$ in.

Wheel load distribution width:

$D = c_{bj}(\text{wheel width} + 2\ t_d)$

Article 3.30, AASHTO specifications:

Tire contact area = 0.01 P

For HS-20 design load, P = 16,000 lbs.

So tire contact area = (0.01)(16,000) = 160 in.^{2}

$$\frac{\textit{length in direction of traffic}}{\textit{width of tire}} = \frac{1}{2.5}$$

Length = 0.4 width, so 0.40 w^2 = 160 in.^{2}

For this example, w = 20.00 in.

If butt joints are positioned every 4th laminae (J),

$$c_{bj} = \frac{j}{j+1} = \frac{4}{4+1} = 0.80$$

The wheel load distribution width, D_w, is:

D_w = (0.80)(20.00 in. + 2(14.00 in.)) = 38.40 in.

4. **Compute design dead and live load moments.**

Dead load of timber deck = (25 ft)(26 ft)(14/12)(50 lb/ft^3)
= 37,917 lbs.

Dead load of curb, railing, etc. = (25 ft)(185 lb/ft)
= 4625 lbs.

Dead load of wearing surface = 22,500 lbs.

Total dead load = 65,042 lbs

Distributed dead load, $$w_{DL} = \frac{65{,}042\ lbs}{25\ ft} = 2602\ lbs/ft$$

Design DL moment, $$M_{DL} = \frac{(2.602\ k/ft)(25\ ft)^2}{8} = 203.3\ ft\text{-}k$$

Design live load moment, M_{LL} = 103.68 ft-k

Total design moment = 203.3 + 103.68 = 306.98 ft-k

5. **Compute flexural stress and compare to allowable.**

Effective section modulus, S:

$$S = \frac{D\,t^2}{6} = \frac{(38.40\ in.)(14\ in.)^2}{6} = 1254\ in.^3$$

Actual flexural stress in deck, f_b:

$$f_b = \frac{M_{tot}}{S} = \frac{(306.98 ft\text{-}k)(12 in/ft)}{1254\ in.^3} = 2.94\ k/in.^2$$

Check: 2.94 ksi > 2.625 ksi ∴ NO GOOD —> recycle.

Note: don't recycle until after checking LL deflection

6. **Check live load deflection.**

I = 1.15 x I used for stress calculation

(see Article 3.25.5.3, AASHTO (2))

$$I = \frac{1.15\,D\,t^3}{12} = \frac{(1.15)(38.40\ in.)(14)^3\ in.}{12} = 10{,}098\ in.^4$$

$$\Delta_{LL} = \frac{\textit{Deflection coeff.}}{E'I} = \frac{1.11 \times 10^{10}}{(1{,}800{,}000\ lb/in.^2)(10{,}098\ in.^4)} = 0.61\ in.$$

Compare to allowable deflection = L/500 = 0.60 in.

Check: 0.61 in. > 0.60 in. ∴ NO GOOD —> recycle.

7. **Revise thickness if necessary.** Try a deck thickness of 16 in. (**Input** *). For brevity, the calculations will be omitted for the 2nd cycle of calculations.

   For a deck thickness of 16 in.:

   D_w = 41.60 in.

   Dead load of timber deck = 43,333 lbs.

   Total design dead load = 70,458 lbs.

   Total design moment (DL + LL) = 323.86 ft-k

   Effective section modulus, S = 1774 in^3

   Flexural bending stress, F_b = 2.19 ksi

   Check: 2.19 ksi < 2.625 ksi ∴ OK for flexure

   Check live load deflection:

   I = 16,329 in^4

   Live load deflection = 0.38 in.

   Check: 0.38 in. < 0.50 in. ∴ OK for live load deflection.

8. **Calculate dead load deflection and camber.**

$$I = \frac{(41.60\ in.)(16\ in.)^3}{12} = 14{,}199\ in.^4$$

$$w_{DL} = \frac{DL_{deck} + DL_{wearing}}{Length,\ in.} = \frac{43{,}333 + 22{,}500}{(25)(12)} = 219.4\ lb/in.$$

$$\Delta_{DL} = \frac{5 w_{DL} L^4}{384\ E'\ I} = \frac{(5)(219.4([(25)(12)]^4}{(384)(1{,}800{,}000)(14{,}199)} = 0.19\ in.$$

   Camber should be set to $3\Delta_{DL}$ = 3(0.91) = 2.72 in.

9. **Determine required level of prestress force.**

   Case A - transverse bending:

$$M_T = \frac{0.79 M_x}{1000}\left(\frac{b}{L}\right)^{0.5} = (0.79)(103.68)(12)\left(\frac{13}{12}\right)^{0.5} = 708.8\ in\text{-}lb/in.$$

$$p = \frac{6\,M_T}{t_d^2} = \frac{(6)(708.8)}{(16)^2} = 16.61\ lb/in.^2$$

Case B - transverse shear:

For an HS-20 load, P = 16,000 lbs., so:

$$V_T = \frac{P}{1000}(10.4-\frac{b}{L}) = \frac{16000}{1000}(10.4-\frac{(13)(12)}{(25)(12)}) = 158.10\ lb/in.$$

$$p = \frac{1.5\,V_T}{\mu\,t_d} = \frac{(1.5)(158.1)}{(0.45)(16)} = 32.94\ lb/in.^2$$

Note: AASHTO requires that p must be ≥ 40 lb/in.^{2}, so set p = 40 lb/in.^{2}

The initial prestressing force, p_i = 2.5 p = 100.0 lb/in.^{2}

10. **Select size and spacing of prestressing elements.**

Two conditions must be satisfied:

$$A_s \geq \frac{p_i\,s\,t_d}{0.70 f_{pu}}$$

$$A_s \leq 0.0016\,s\,t_d$$

Try a spacing of 50 in. (K):

$$A_s \geq \frac{p_i\,s\,t_d}{0.70 f_{pu}} = \frac{(100)(50)(16)}{(0.70)(150{,}000\ psi)} = 0.76\ in^2$$

$$A_s \leq 0.0016\,s\,t_d = (0.0016)(50)(16) = 1.28\ in.^2$$

Check: s ≤ 60 in. ∴ OK. (see Art. 13.11.2.2, AASHTO (1))

11. **Size the prestressing elements.** For this example, use 1 in. diameter, 150 ksi rods (L).

The force in the prestressing rods is computed as:

$$F_{ps} = p_i\,s\,t_d = (100 lb/in.^2)(50\ in.)(16\ in.) = 80{,}000\ lbs$$

12. **Size bearing and anchorage plate.** The required area for the bearing plate is calculated as:

$$A_{PL_{reqd}} = \frac{p_i\, s\, t_d}{F'_{c\perp}} = \frac{(100)(50)(16)}{385\ lb/in.^2} = 207.8\ in.^2$$

Assume A36 steel (M)

Try L_p = 16 in., W_p = 14 in. (N and 0) A_{PL} = (16)(14) = 224 in.^{2}

Check (advisory): $1.0 \leq \frac{L_p}{W_p} = \frac{16}{14} = 1.14 \leq 2.0$ ∴ OK

Actual bearing stress is calculated as:

$$f_{bp} = \frac{p_i\, s\, t_d}{A_{PL}} = \frac{(100)(50)(16)}{224} = 357.1\ psi$$

Anchorage plate design:

Exact size depends on manufacturer.

Typical values (from Table 9-6, Ref. 66):

For 1 in. dia. rod: W_A = 4 in., L_A = 6.5 in., t_A = 1.25 in.

Calculation of bending stress in bearing plate:

$$k = \frac{W_p - W_A}{2} = \frac{14-4}{2} = 5$$

or

$$= \frac{L_p - L_A}{2} = \frac{16-6.5}{2} = 4.75$$

Use k = 5.

The thickness of the bearing plate must be great enough to prevent bending in the plate.

$F_s = 0.55\ F_y$

For A36 steel, F_s = 19,800 psi

$$t_p = \sqrt{\frac{3 f_{bk} k^2}{F_s}} = \sqrt{\frac{(3)(357.1)(5)^2}{19{,}800}} = 1.16\ in.^2$$

Use $t_p = 1.25$ in.

13. **Check bearing stress at abutments.** Assume a timber cap beam width of 12 in. for this example.
Reaction due to dead load, R_{DL}:

$$R_{DL} = \frac{1}{2}\left(\frac{41.60\ in}{12}\right)(25\ ft)\left[\frac{(16\ in.)}{12}(50\ lb\text{-}ft^3) + \left(\frac{3\ in.}{12}\right)(150\ lb/ft^3)\right] = 4{,}513\ lbs. = 451\ k$$

$R_{LL} = 23.04$ k. (from Table 16.8, Ref. 66).

$$f_{c\perp} = \frac{R_{DL}+R_{LL}}{D_w\, l_b} = \frac{4.51 + 23.04}{(41.60)(12)} = 55.2\ psi$$

Check: 55.2 lb/in.^{2} < $F'_{c\perp}$ = 385 lb/in.^{2} ∴ OK.

Summary of Design Values:

Length = 25 ft, Width = 26 ft

Design loading: AASHTO HS-20, 2 lanes wide.

Lumber: Grade No. 1, Douglas fir-larch

Deck thickness = 16 in.

Stressing system: 1 in. dia. 150 grade rods at 50 in. centers

End rods 25 in. from end of bridge

Rod Anchorage system: Use 16 in. x 14 in. x 1.25 in. bearing plate

Use 4 in. x 6.5 in. x 1.25 in. anchor plate

Both plates of A36 steel

Stresses and deflections:

f_b = 2.19 ksi

F_b' = 2.625 ksi

Δ_{LL} = 0.38 in.

Δ_{DL} = 0.91 in.

Camber = 2.72 in.

P = 40 psi

$P_i = 100$ psi

$F_{ps} = 80{,}000$ lbs

$f_{c\perp}$ at anchorage = 357 psi

$f_{c\perp}$ at bearing = 55 psi

$F_{c\perp}' = 385$ psi

3.7.10. Glue-Laminated Timber Beam Bridge

3.7.10.1. Background

In the past few years, a number of new wood products, such as structural composite lumber, have been developed. Although there are several different techniques for manufacturing large members from small timber laminates, this report will concentrate only on glue-laminated (glulam) timber beam and timber deck construction. The are two reasons for this limitation. First, glulam beams have been in use since the mid-1940's and the design methodology for this type of member is recognized by AASHTO. Secondly, systems such as laminated veneer lumber (LVL), while showing much potential for future use, have not been used in actual bridge construction. No formal specifications have been developed for LVL beam bridges at this time. Additional information on the use of LVL for bridges can be found in (Refs. 5, 77, 78, 98).

Glue-laminated panel bridge decks, which were developed at the USDA Forest Products Laboratory in the 1970's, are the most common type of timber deck in use today. The panels are normally 5 to 8 in. thick and 3 to 5 ft wide. Glulam decks are much stiffer and stronger than conventional nail-laminated decks because of the rigid bond between laminations.

Glue-laminated (glulam) timber beam bridges (see Fig. 3.37) are constructed essentially the same as an ordinary sawn lumber beam bridge with the exception of the beams themselves. Glulam beams are manufactured from 1-1/2 or 1-3/8 in. thick timber lamintes which are bonded together on their wide faces with waterproof structural adhesives. They are available in a number of standard widths from 3 to 14 1/4 in., while the depth of a glulam beam is limited only by the size of the pressure treating facility and transportation problems. Although glulam timber beams can be fabricated in essentially any shape, the most economical shape is a standard size beam which is available from a number of fabricators (66).

Glulam timber beams offer several advantages over ordinary sawn lumber beams. Because the depth of glulam beams are greater, a given bridge will require fewer beams. Also the glulam beams are able to span greater distances than sawn lumber beams. Glulam beams are able to span more than 140 ft, but are more commonly used for span lengths of 20 to 100 ft.

Glulam timber bridge beams are fabricated with horizontally laminated bending combinations given in Table 1 of AITC 117 - Design (4). These combinations provide the most efficient beam section where primary loading is applied perpendicular to the wide face of the laminations (66).

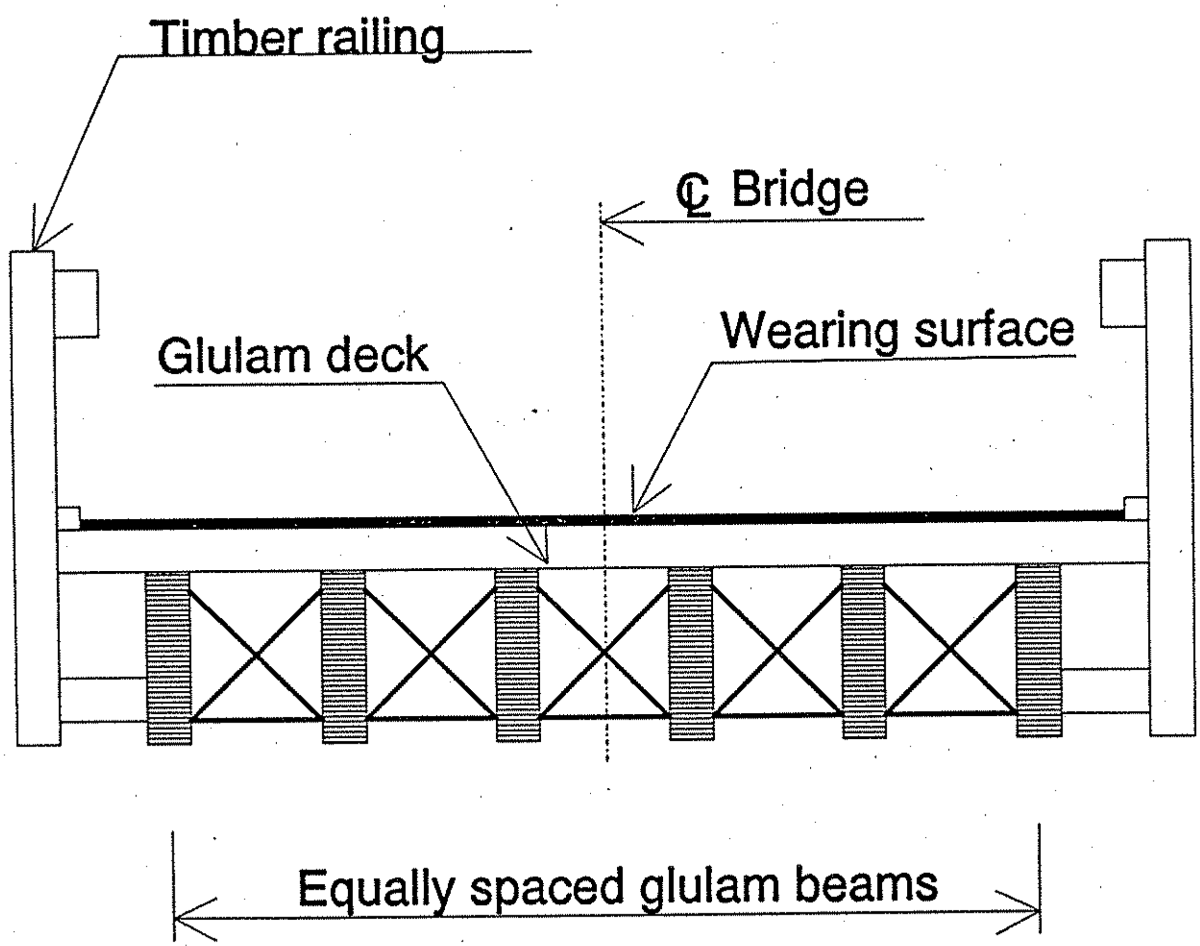


Fig. 3.37. Typical section of glulam timber beam bridge.

3.7.10.2. Design criteria

The design material presented in this section is based on the *AASHTO Standard Specifications for Highway Bridges* (2). Much of the material presented in this section is based on work presented by Ritter (66).

The glulam timber beam and glulam timber deck design procedures described in this manual are applicable only to AASHTO Group I loads. In this case, it is assumed that the design will be controlled by a combination of dead load and standard AASHTO truck loads. Dead load is assumed to include the weight of the timber itself, plus a 3 in. asphalt wearing surface and any guardrails or other attachments.

Material properties for various sawn lumber species are taken from the 1986 edition of the National Forest Products Association's *Design Values for Wood Construction* (51). Although this particular discussion applies only to Southern Pine and Douglas Fir, the basic design principles could be applied to a number of species. Combination symbols for glulam timber are taken from AITC 117 - Design (4). Timber components used in this procedure are assumed to be pressure treated with an oil-borne preservative.

The AASHTO specifications do not specify live load deflection limitations for either glulam beam or deck design. Deflection guidelines which follow are based on common design practice and experience.

Perhaps the most influential factor in the economic design of a glulam timber beam bridge is the beam configuration. The number and spacing of beams affects the size and strength requirements for the beam and deck elements which in turn significantly influence the cost of materials, fabrication and construction. Three primary factors influence the beam configuration in glulam bridges: site restrictions, deck thickness, and live load distribution to the beams. Each of these factors will be discussed briefly in the following paragraphs.

Site restrictions. The most efficient beam is a deep, relatively narrow section. In some cases, however, this cross-section may be impractical due to overhead clearance limitations. In such cases, shallower beams are employed which in turn requires the number of beams to be increased for the desired load capacity. These shallow beams are generally arranged in several closely spaced groups. In most cases, though, a deck-type structure (See Secs. 3.79 and 3.7.11) would be more economical for sites with clearance problems.

Deck thickness. As beam spacing increases, deck flexural stresses and deflections increase, requiring either a thicker deck, or one with a greater flexural stiffness. Glulam decks are available in standard member dimensions that increase in 1½ or 2 in. increments.

Live load distribution. The magnitude of the vehicle live load supported by each beam is directly proportional to the distribution factor (DF) for that beam. The DF provides an indication of the relative beam size and grade requirements for different configurations.

The ability of a bridge to distribute loads laterally depends on the transverse stiffness of the structure and the number, size and spacing of beams. Although load distribution is influenced by the type and spacing of beam bracing or diaphragms, these factors are not considered in the load distribution factor because of their minimal effect.

The AASHTO specifications provide an empirical method for determining the lateral distribution of wheel loads. The fractional portion of the total vehicle load distributed to each beam is computed as a distribution factor, DF, expressed in wheel lines per beam. The design force, moment, shear or reaction, is computed by multiplying the maximum design force for one wheel line of the design vehicle by the appropriate distribution factors. Tables of maximum vehicle live load moments can be found in Appendix A, AASHTO (2).

Distribution for moment. AASHTO specifications assume that wheel loads act as point loads for the computation of bending moments. The lateral distribution factor is determined based on the position of the beam relative to the roadway. Although different criteria are used for interior and exterior beams, exterior beams should not be designed for moments smaller than those used in the design of interior beams.

The distribution factor for moment in exterior beams is determined by assuming the deck acts as a simple span between beams and then computing the reaction of the wheel lines on the exterior beam. Wheel lines in the outside lane are positioned laterally to produce the maximum reaction on the beam, however, the wheel line cannot be placed closer than 2 ft from the curb. The distribution factor for moment in interior beams is computed from empirical formulas which relate deck thickness, beam spacing and the number of traffic lanes.

Table 3.29 presents the AASHTO distribution factors based on beam spacing, S, and the number of design traffic lanes. Note that for a one lane bridge with $S > 6$ ft and for a multi-lane bridge with $S > 7.5$ ft, the distribution factor for moment should be taken as the reaction of the wheel lines, assuming the deck to act as a simple span between longitudinal beams.

Table 3.29. AASHTO live load distribution factors.

Nominal deck thickness (in.)	DF for moment (wheel lines/beam)	
	One lane	Two or more lanes
4	S/4.5	S/4.0
≥ 6	S/6.0	S/5.0

Distribution for shear. AASHTO specifications require that horizontal shear in glulam beams be based on the maximum vertical shear which occurs at a distance 3 times the beam depth, 3d, from the support or at the quarter point of the span, L/4, whichever is less. Lateral shear distribution at this point is computed as one half the sum of 60 percent of the shear from the undistributed wheel lines and the shear from the wheel lines distributed laterally for moment. For undistributed wheel lines, one wheel line is assumed to be carried by one beam. The live load shear can be expressed as:

$$V_{LL} = 0.5[(0.6\ V_{LU}) + V_{LD}]$$

where:

V_{LL} = distributed live load vertical shear used to compute horizontal shear (lb).

V_{LU} = maximum vertical shear from an undistributed wheel line (lb).

V_{LD} = maximum vertical shear from the vehicle wheel lines distributed laterally as specified for moment (lb).

Distribution for reactions. The live load distribution for reactions is computed assuming there is no transverse distribution of wheel loads to adjacent beams. The distribution factor, DF, for both interior and exterior beams is computed as the reaction of the wheel lines at the beam, again assuming the deck acts as a simple span between longitudinal beams.

Exclusive of site restrictions, beam configurations should be based on economic and performance considerations for both the beam and deck components. These considerations will vary depending on material prices, availability, and transportation and construction costs. Table 3.30 provides a general guideline for the number and spacing of glulam beams.

Several modification factors have been developed to account for the behavior of different timber species for environmental and loading conditions. Note that the equations developed in this section do not include the duration of load factor, C_D, or the modification factors for temperature effect, C_t, and fire-retardant treatment, C_R.

To reduced fabrication costs, glulam beams should be developed using standard dimensions. Table 3.31 is provided for determining standard size glulam beams.

Table 3.30. Guidelines for number and spacing of glulam beams.

Roadway Width (ft)	Number of beams	Beam spacing (ft)	Deck overhang (ft)	Moment DF
24	5	5.0	2.0	1.00
26	5	5.5	2.0	1.10
28	5	6.0	2.0	1.20
34	6	6.0	2.0	1.20

Table 3.31. Standard glulam beam dimensions.

Nominal width (in.)	Net finished width (in.)	
	Western Species	Southern Pine
4	3-1/8	3
6	5-1/8	5
8	6-3/4	6-3/4
10	8-3/4	8-1/2
12	10-3/4	10-1/2
14	12-1/4	-
16	14-1/4	-

3.7.10.3. Design Procedure

The design procedure described in this section is for a glue laminated timber beam bridge. There are several types of bridge decks which are available commercially, thus no provision for deck design is provided. Spreadsheet input parameters refer to the spreadsheet in Fig. 3.38.

1. **Define basic configuration and design criteria.** Several dimensions must be determined:
   - Span length, L, from c-to-c of bearing **(Input A)**.
   - Roadway width, W, from inside of curbs **(Input B)**.

     Note: in this spreadsheet, the additional width of curbs and/or railing, etc., has been ignored.
   - Number of traffic lanes.
   - Number and spacing of beams (computed automatically by the spreadsheet based on roadway width).
   - Deck and railing/curb configuration.
   - Design live load vehicle **(Input C)**.

2. **Select beam combination symbol and species.** A preliminary beam combination symbol should be selected from the AITC 117 - Design manual (4) **(Input D)**. Commonly used combination symbols for glulam beam bridges are presented in Table 3.32.

   Based on the combination symbol, the tabulated design properties of the member can be determined from AITC 117. The allowable bending stress, F_{bx}, allowable compressive stress perpendicular to grain, F_{cp}, allowable horizontal shear stress, F_{vx}, and Young's modulus, E_x, should be recorded for later use. These tabulated design properties must be reduced for wet-

Design of Glue-Laminated Timber Beam Bridges

Input deck geometric requirements:

Input bridge length, L =	94.00	ft	A
Input roadway width, W =	24.00	ft	B

Select design live loading:

1) HS 20-44
2) HS 15-44

Please enter number of your choice:	1	C

Select beam combination symbol and species:

1) 24F-V3 Western species
2) 24F-V4 Western species
3) 24F-V2 Southern pine
4) 24F-V3 Southern pine
5) 24F-V6 Southern pine

Please enter number of your choice:	2	D

Computed design properties (adjusted for wet-use conditions):

Allowable bending stress, Fbx =	1920	psi
Compression perp. to grain, Fcp =	345	psi
Allowable shear stress, Fvx =	144	psi
Young's modulus, Ex =	1.499E+06	psi

Select type and thickness of wearing surface:

1) No wearing surface
2) Ashphalt wearing surface

Please enter number of your choice:	2		E
Thickness of a/c wearing surface (if any):	3.00	in.	F

Computed dead load moment (Assume 5 1/8" timber deck):

Dead load of deck panels =	21.35	lb/ft^2
Dead load of wearing surface =	37.50	lb/ft^2
Total uniform dead load =	58.85	lb/ft^2

Fig. 3.38. Glulam timber beam spreadsheet, input parameters, and example problem.

Beam spacing (based on roadway width) =	5.00	ft
Deck overhang (outside beam to face of rail) =	2.00	ft

DL moment for interior beams:

Uniform dead load, WDLI =	294.27	lb/ft
Estimated wt. of beam =	319.60	lb/ft
Dead load moment, MDLI =	678.02	ft-kip

DL moment for exterior beams:

Uniform dead load, WDLE =	339.27	lb/ft
Estimated wt. of beam =	319.60	lb/ft
Dead load moment, MDLE =	727.72	ft-kip

Computed live load moment:

Distribution factor, DFM =	1.00	
Moment due to one wheel line =	708.09	ft-kip
Live load moment, MLL =	708.09	ft-kip

Computed beam size based on bending stress:

Preliminary allowable stress, Fb' = (Doesn't include CF)	1920	psi
Design moment for interior beams, MTI =	1,386.1	ft-kip
Design moment for exterior beams, MTE =	1,435.8	ft-kip
DESIGN ALL BEAMS FOR DESIGN MOMENT =	1,435.8	ft-kip
Required section modulus, Sx CF =	8,974	in^3

Western species - lightest alternative:

Press <ALT> W to select the lightest beam section.

Fig. 3.38. Continued.

Computed lightest section which meets requirements:

Beam width, b =	12.25 in.
Beam depth, d =	73.5 in.
Section modulus, Sx =	11,030 in^3
Adjusted section modulus, SxCF =	9,018 in^3
Cross section area, A =	900.38 in^2
Moment of inertia, Ix =	405,338 in^4
Self-weight of beam =	312.63 lb/ft

Actual weight of beam < estimated - OK!

Actual bending moment, Mactual =	1,428.1 ft-kip
Bending stress on beam, fb =	1,553.8 psi
Allowable bending stress, Fb' =	1,569.8 psi

BEAM IS SATISFACTORY FOR FLEXURE.

Computed check of beam for lateral stability:

Distance between lateral support, lu =	23.50 ft.	
Length-to-depth ratio, lu/d =	3.84	
Effective length, le =	680.16 in.	
Beam slenderness factor, Cs =	18.25	
Intermediate beam factor, Ck =	26.72	Intermediate beam
Lateral stability factor, CL =	0.927	

CL > CF so strength controls design - OK.

Computed check of live load deflection:

Live load deflection, DELTALL =	1.68 in.
Allowable LL deflection, L/360 =	3.13 in.

BRIDGE IS SUFFICIENT FOR LL DEFLECTION.

Fig. 3.38. Continued.

Computed check of horizontal shear:

Uniform dead load on beam =	651.90 lb/ft	
Dead load vertical shear, VDL =	26,646 lb.	
LL vertical shear computed at distance =		18.38 ft.
Max. shear due to one wheel line, VLU =		25,388 lb.
Vert. shear distributed laterally, VLD =		25,388 lb.
Live load vertical shear, VLL =		20,311 lb.
Total vertical shear, V = VDL + VLL =		46,957 lb.
Horizontal shear stress, fv =	78.23 lb/in^2	
Allowable shear stress, Fvx =	144 lb/in^2	

BEAM IS SUFFICIENT FOR SHEAR.

Computed bearing length and bearing stress (wet-use conditions):

Allowable compressive stress, Fcp =		344.5 lb/in^2		
Reaction due to uniform load, RDL =		30,639 lb.		
Distribution factor for reaction =		1.00		
Reaction due to one wheel line, Rwheel =		32,430 lb.		
Total reaction force, Rtotal =		63,069 lb.		
Required bearing length =	14.9 in.	Use	18.00 in.	
Recompute DL reaction, RDLtotal =		31,128 lb.		
Total compressive stress, fcp =	286.03 lb/in^2			

BEAM IS SUFFICIENT FOR BEARING.

Computed design camber:

Uniform dead load, wunif =		651.90 lb/ft
Dead load deflection, DELTADL =		1.88 in.
Minimum design camber =	3.80 in.	

Fig. 3.38. Continued.

Summary of Design Values

Geometry and design loading:

Bridge Length, L =	94.00	ft.
Roadway width, W =	24.00	ft.
Design live load:	AASHTO HS20-44	
Beam combination symbol:	24F-V4 Western species	
Type of wearing surface:	Asphalt	3.00 in.

Beam dimensions and properties:

Beam width, b =	12.25	in.
Beam depth, d =	73.5	in.
Beam spacing, S =	5.00	ft.
Beam section modulus, Sx =	11,030	in^3
Beam cross sectional area, A =	900.375	in^2
Beam moment of inertia, Ix =	405,338	in^4
Self weight of beam, wbeam =	312.630	lb/ft

Stresses and deflections:

Actual bending stress, fb =	1,554	lb/in^2
Allowable bending stress, Fb' =	1,570	lb/in^2
Actual shear stress, fv =	78	lb/in^2
Allowable shear stress, Fvx =	144	lb/in^2
Actual bearing stress, fcp =	286	lb/in^2
Allowable bearing stress, Fcp' =	345	lb/in^2
Live load deflection, DELTALL =	1.68	in.
Allowable LL deflection, L/360 =	3.13	in.
Dead load deflection, DELTADL =	1.88	in.
Design camber =	3.80	in.

Fig. 3.38. Continued.

use conditions unless a watertight deck is used. The computer spreadsheet in this manual is based on wet-use conditions. The spreadsheet automatically calculates the allowable stresses based upon the combination symbol and species selected.

Table 3.32. Commonly used glulam beam combination symbols.

Bridge type	Western Species	Southern Pine
Simple span	24F-V3 24F-V4	24F-V2 24F-V3 24F-V6
Continuous spans	24F-V8	24F-V5

3. **Determine deck dead load and dead load moments.** The deck dead load supported by each member should be computed. This dead load should include such things as the weight of the deck, railing, wearing surface and hardware. If there is no better estimate available, a preliminary deck thickness of 6-3/4 in. may be assumed. The difference between the estimated and the actual member weights is normally insignificant, but should be verified. The existence and type of wearing surface should be input as **E** and **F** for spreadsheet users. The spreadsheet automatically computes the dead load moment based on these input parameters.

   The dead load moment at any position along the span, M_{DL}, for a uniformly distributed load can be computed as:

$$M_{DL} = \frac{w_{DL}\,x}{2}(L - x)$$

4. **Determine live load moment.** Live load moments are computed for both interior and exterior beams by multiplying the maximum moment for one wheel line of the design vehicle (whether based on the design vehicle or the equivalent lane loading) by the appropriate distribution factors. Tabulated values of maximum live load moments can be found in Appendix A, AASHTO (2).

   The spreadsheet performs this calculation automatically based upon the span length and the live load design vehicle chosen.

5. **Determine beam size based on bending.** The allowable bending stress in glulam timber beams is controlled by either the size factor, C_F, which limits bending stress in the tension zone, or the lateral stability of beams factor, C_L, which limits bending stress in the compression zone. Under

normal circumstances, the allowable bending stress in bridge beams is controlled by C_F, however, both values should be checked.

The adjusted allowable bending stress in a glulam bridge beam can be computed from:

$$S_x\, C_F = \frac{M}{F_b'}$$

where:

$S_x\, C_F$ = required beam section modulus adjusted by size factor, C_F (in^3).

M = applied dead and live load bending moment (in-lb).

$F_b' = F_{bx}\, C_M$ (psi).

C_M = moisture content factor for bending = 0.80.

After an initial beam size has been determined, the dead load moment due to beam self weight can be computed and the design moment revised. This iterative process should be continued until a satisfactory beam size has been determined.

The computer spreadsheet performs this iteration automatically, using three cycles to determine the beam which provides the required flexural capacity. To initiate the iteration process using the spreadsheet, press <ALT> W when using western species, and <ALT> S when using Southern Pine. Spreadsheet users perform **(Input G)**.

The actual applied bending stress can be computed based on the total moment and compared to the allowable bending stress as:

$$f_b = \frac{M}{S_x} \leq F_b' = F_{bx}\, C_M\, C_F$$

In addition to satisfying allowable bending stresses, the proposed beam must be checked for lateral stability. The allowable bending stress, based on lateral stability, depends on the slenderness of the proposed beam. The slenderness factor, C_S, can be computed as:

$$C_S = \sqrt{\frac{l_e\, d}{b^2}}$$

where:

l_e = effective beam length for a single span beam with a uniformly distributed load, in. = 1.63 l_u + 3 d

l_u = unsupported beam length, in.

d = beam depth, in.

b = beam width, in.

Glue laminated timber beams are classified as short, intermediate or long; allowable bending stresses are based on this classification.

Beams with a C_S of 10 or less are classified as short. The capacity of these short members is controlled by the strength of the wood, rather than stability, and can be computed from the equation above.

Intermediate beams are those with a C_S of between 10 and C_k, where C_k is given by:

$$C_k = 0.956 \sqrt{\frac{E'}{F_b''}}$$

and:

$E' = E\, C_M$, psi

$F_b'' = F_b\, C_M$, psi

It should be noted that the above equation for C_k is based on a modified NDS equation which takes into account the reduced variability of glulam timber beams (52).

Intermediate beams can fail by either an overstress in bending, or by torsional buckling from lateral instability. The controlling mode is indicated by the lateral instability of beams factor, C_L, which is given by:

$$C_L = [1 - \frac{1}{3}(\frac{C_S}{C_k})^4]$$

If C_L is less than C_F, the bending stress is controlled by stability, and C_L is the controlling modification factor. The allowable bending stress is then computed using the following relation:

$$F_b' = F_b\, C_M\, C_L$$

Long beams are those with a slenderness ratio in the range $C_k \leq C_S \leq 50$. Lateral stability, rather than strength, controls the design of long beams. As with the intermediate beams, the allowable bending stress for long beams is a modified NDS equation which takes into account the reduced variability of glulam timber beams and is computed by:

$$F_b' = \frac{0.609\, E'}{C_S^2}$$

6. **Check live load deflection.** The live load deflection of a glulam beam bridge can be computed by several methods of analysis. For a uniformly loaded simple span, the maximum deflection occurs at midspan and is computed as:

$$\Delta_{LL} = \frac{5\, w_{LL}\, L^4}{384\, E'\, I}$$

A reasonable limit on live load deflection is L/360. A lower value of deflection should be considered if the bridge supports a pedestrian walkway or will be covered with an asphalt riding surface. The spreadsheet automatically computes the live load deflection and compares it to the allowable value.

7. **Check horizontal shear.** Dead load horizontal shear is based on the maximum vertical shear which occurs a distance equal to the beam depth, d, from the support. For a uniform dead load, the dead load shear, V_{DL}, can be computed as:

$$V_{DL} = w_{DL}\left(\frac{L}{2} - d\right)$$

where:
V_{DL} = vertical dead load shear at a distance d from the support, lb.
w_{DL} = uniform dead load supported by the beam, lb/in.

Live load vertical shear is computed at a distance from the support of 3d or L/4, whichever is less. The horizontal shear stress due to applied loads, f_v, which is 1.5V/A for a rectangular cross section, must not exceed the allowable stress, which is given by:

$$f_v = \frac{1.5\, V}{A} \leq F_v' = F_{vx}\, C_M$$

where:
$V = V_{DL} + V_{LL}$, lb.
A = cross-sectional area of beam, in.^{2}
C_M = moisture content factor for shear = 0.875.
It should be noted that the allowable shear stress may be increased by a factor of 1.33 for overloads in AASHTO Load Group 1B. The spreadsheet performs this calculation for the user.

8. **Check lateral and longitudinal loads.** The magnitude and appropriate lateral and longitudinal loads, such as wind load and centrifugal force will vary among structures. It is the designer's responsibility to compute and apply the necessary loads in accordance with AASHTO load groups. The spreadsheet does not consider this step in the design process.

9. **Determine bearing length and stress.** The bearing area at beam reactions must be large enough to limit bearing stresses to an acceptable value. The dead load reaction, R_{DL}, due to the beam, deck, wearing surface, railing and hardware should be computed from statics. The live load reaction at each beam, R_{LL}, can be computed by multiplying the maximum reaction at one wheel line by the appropriate distribution factor for reactions as computed previously.

The required bearing length, L_{bear}, should be no less than the following:

$$L_{bear} = \frac{R_{DL} + R_{LL}}{b\,F_{c\perp}}$$

where:

R_{DL} = dead load reaction, lb.

R_{LL} = distributed live load reaction, lb.

b = beam width, in.

$F_{c\perp}$ = allowable compressive stress perpendicular to grain, psi

= $F_{c\perp x}\,C_M$

The actual applied bearing stress can be computed from:

$$F_{c\perp} = \frac{R_{DL} + R_{LL}}{A}$$

where A is the bearing area in square inches.

The spreadsheet automatically computes the required bearing length and also computes a "rounded value" to the next largest 6 in. increment for design purposes.

10. **Determine camber.** Camber is provided to offset the effect of long-term dead load deflection. The amount of camber to build into a glulam beam bridge is a decision of the designer. There are two "rules of thumb" which can be used. For spans less than 50 ft, camber should generally be 1.5 to 2.0 times the dead load deflection plus 0.5 times the vehicle live load deflection. For spans greater than 50 ft, camber can be estimated as 1.5 to 2.0 times the dead load deflection. The spreadsheet performs this calculation automatically.

3.7.10.4. Example

The spreadsheet input and ouput for this example is shown in Fig. 3.38. In this example, the following bridge will be used:

Length, L = 94 ft from c-to-c of bearings

Roadway width, W = 24 ft from inside of curbs

AASHTO HS20-44 live load (Group I loads only)

3 in. asphalt wearing surface

Visually graded western species

1. Define basic configuration and design criteria.

L = 94 ft

W = 24 ft

HS20-44 design live loading

3 in. asphalt wearing surface

(Input A, B, C, E and F respectively.)

2. Select beam combination symbol and species.

Select 24F-V4, Western species combination symbol **(Input D).**

The design properties are:

F_{bx} = 2400 psi C_M = 0.80

F_{cp} = 650 psi C_M = 0.53

F_{vx} = 165 psi C_M = 0.87

E_x = 1.800 x 10^6 psi C_M = 0.83

The allowable stresses are computed as:

$F_b' = F_{bx}C_MC_F$

$F_{cp}' = F_{cp}C_M = 345$ psi

$F_v' = F_{vx}C_M = 144$ psi

$E' = E_xC_M = 1.499 \times 10^6$ psi

3. Determine deck dead load and dead load moments.

Dead load of the deck panels and wearing surface can be computed as:

$$DL = \frac{(5.125\ in.)(50 lb/ft^3) + (3\ in.)(150\ lb/ft^3)}{12\ in./ft} = 58.85\ lb/ft^2$$

The dead load supported by each beam is proportional to the tributory area. For this example, interior beams support 5 ft of deck width and exterior beams support 5 ft of deck plus 45 lb/ft of curb/rail load.

As a "rule of thumb" for a glulam beam bridge, the estimated weight of the beam itself can be computed as 3.4 times the length of the bridge:

$$W_{beam} = 3.4(94 \text{ ft}) = 319.6 \text{ lb/ft}$$

For interior beams:

$$W_{DL} = (5.0\,ft)(58.85\,lb/ft) + 319.6\,lb/ft = 294.27 + 319.6 = 613.85\,lb/ft$$

$$M_{DL_I} = \frac{(613.85\,lb/ft)(94\,ft)^2}{(8)(1000)} = 678.02\,ft\text{-}k$$

For exterior beams:

$$W_{DL} = 294.27\,lb/ft + 45\,lb/ft + 319.6\,lb/ft = 339.27 + 319.6 = 658.87\,lb/ft$$

$$M_{DL_E} = \frac{(658.87\,lb/ft)(94\,ft)^2}{(8)(1000)} = 727.72\,ft\text{-}k$$

4. **Determine live load moment.** The distribution factor for moment, DFM, is computed as:

$$DFM = \frac{S}{5} = \frac{5.0}{5} = 1.00$$

The live load moment due to one wheel line can be found in Appendix A, AASHTO (2). For a 94 ft span:

$$M_{wheel\ line} = 708.09 \text{ ft-k}$$

The live load moment distributed to one beam is found as:

$$M_{LL} = (708.09 \text{ ft-k})(1.00) = 708.09 \text{ ft-k}$$

5. **Determine beam size based on flexure.** The AASHTO specifications require that the exterior beam be at least as large as the interior beams. For simplicity, one beam size will be designed for the maximum design moment in the bridge. Conservatively, the maximum dead load moment will be added to the maximum live load moment, even though these moments do not occur at the same location in the span.

For this example:

$$M_T = 727.72 + 708.09 = 1435.81 \text{ ft-k}$$

The size factor, C_F, can not be determined until a beam size has been determined. In the iterative process, the S_xC_F terms will be used; after a preliminary beam size has been determined, C_F will be calculated.

Based on an allowable bending stress, $F_b' = 1920$ lb/in², the required section modulus can be computed as:

$$S_x\,C_F = \frac{M_T}{F_b'} = \frac{(1435.81\ ft\text{-}k)(12{,}000)}{1920\ psi} = 8973.83\ in^3$$

For each standard beam width, a beam depth can be determined which will provide the required S_x. The section modulus for this width and depth is computed, and reduced by the factor C_F. A new beam size is then determined by this iterative process until the required S_xC_F is obtained.

CYCLE 1:

Using a 12¼ in. wide section, a depth of 66.30 in. is required to provide the required S_xC_F. A standard depth of 67½ in. is used.

For this section:

$$C_F = \left(\frac{12}{d}\right)^{1/9} = \left(\frac{12}{67.5}\right)^{1/9} = 0.825$$

$$S_x\,C_F = \frac{C_F\,b\,h^2}{6} = \frac{(0.825)(12.25\ in.)(67.5\ in.)^2}{6} = 7678.0\ in^3$$

This S_xC_F is less than 8973 in³, thus we must recycle.

CYCLE 2:

Based on the C_F from the first cycle, an approximation for S_x can be found as:

$$\frac{S_x}{C_{F_1}} = \frac{8973.83\ in^3}{0.825} = 10{,}872.4\ in^3$$

For this section, a depth of 72.97 in. would be required. Again, a standard depth of 73½ in. is used.

For this section:

$$C_F = \left(\frac{12}{d}\right)^{1/9} = \left(\frac{12}{73.5}\right)^{1/9} = 0.818$$

$$S_x\,C_F = \frac{C_F\,b\,h^2}{6} = \frac{(0.818)(12.25\ in.)(73.5\ in.)^2}{6} = 9017.9\ in^3 > 8973\ in^3$$

A third cycle produces the same section. For a 12¼ in. width, the required depth is 73.5 in.

The computer spreadsheet performs this iterative process for each standard beam width and determines the lightest possible section which satisfies the required S_xC_F criteria.

Spreadsheet users should press <ALT> W to perform this iterative procedure and determine the lightest section which meets the criteria.

For this example, the lightest section is:

Width, b = 12¼ in.

Depth, d = 73½ in.

Section modulus, $S_x = \dfrac{(12.25\ in.)(73.5\ in.)^2}{6} = 11{,}029.6\ in^3$

Adjusted section modulus, S_xC_F = 9017.9 in.^{3}

Moment of inertia, $I_x = \dfrac{(12.25\ in.)(73.5\ in.)^3}{12} = 405{,}338\ in^4$

Cross section area, A = (12.25 in.)(73.5 in.) = 900.38 in.^{2}

Beam weight, $W_{beam} = \dfrac{(900.38\ in^2)(50\ lb/ft)}{144} = 312.63\ lb/ft$

In this example, the estimated beam weight is greater than the actual weight, thus the beam cross section obtained is conservative.

The actual bending moment on the beam can be found as:

$$M_{actual} = \frac{(339.27 + 312.63)(94\ ft)^2}{(8)(1000)} + 708.09\ ft\text{-}k = 1428.11\ ft\text{-}k$$

The actual bending stress on the beam is:

$$f_b = \frac{M_{actual}}{S_x} = \frac{(1428.11\ ft\text{-}k)(12{,}000)}{11{,}029.6\ in^3} = 1553.7\ psi$$

The allowable bending stress is:

$$F_b' = F_{bx}\, C_M\, C_F = (2400\ psi)(0.800)(0.818) = 1569.8\ psi$$

Since $f_b < F_b'$, the beam is satisfactory for flexure. However, the beam must also be checked for lateral stability.

<u>Check of lateral stability:</u>

The distance between points of lateral support, l_u, is assumed to be L/4 = 23.5 ft.

The length-to-depth ratio for this configuration is:

$$\frac{l_u}{d} = \frac{(23.50\ ft)(12)}{73.5\ in.} = 3.84$$

The effective length, l_e, is:

$$l_e = 1.63 l_u + 3d = 1.63(23.5\ ft)(12) + 3(73.5\ in.) = 680.16\ in.$$

The beam slenderness factor, C_S, is found as:

$$C_s = \sqrt{\frac{l_e d}{b^2}} < 50 = \sqrt{\frac{(680.16)(73.5)}{(12.25)^2}} = 18.25$$

The intermediate beam factor, C_k, is the largest value of C_S for which the intermediate beam equation applies. C_k is calculated as:

$$C_k = 0.956\sqrt{\frac{E'}{F_b''}} = 0.956\sqrt{\frac{1.499 \times 10^6}{1920}} = 26.72$$

Since $C_S < C_k$, the beam is classified as an intermediate beam.

The lateral stability factor, C_L, for an intermediate beam is given by:

$$C_L = [1 - \frac{1}{3}(\frac{C_s}{C_k})^4] = [1 - \frac{1}{3}(\frac{18.25}{26.72})^4] = 0.927$$

For this example, $C_L < C_F$, so strength, rather than lateral stability controls the design. Therefore, no changes are required in the previous calculations.

6. **Check live load deflection.** The live load deflection due to an HS20-44 truck can be computed as:

$$\Delta_{LL} = \frac{\textit{Deflection coeff.}}{E' I_x} = \frac{1.02 \times 10^{12}}{(1.499 \times 10^6\ psi)(405{,}338\ in^4)} = 1.68\ in.$$

The allowable live load deflection is:

$$\Delta_{LL_{allow}} = \frac{L}{360} = \frac{(94\ ft)(12)}{360} = 3.13\ in.$$

Since Δ_{LL} < the allowable value, the bridge is adequate with respect to live load deflection.

7. **Check horizontal shear.** The dead load vertical shear is based on the loading shown in Fig. 3.39.

The dead load vertical shear, V_{DL}, is computed as:

$$V_{DL} = W_{DL}(\frac{L}{2}-d) = 651.9\ lb/ft\ (\frac{94\ ft}{2} - \frac{73.5\ in.}{12}) = 26{,}646\ lbs.$$

Live load vertical shear is computed from the maximum vertical shear occurring at the lesser of 3d or L/4 from the support.

For this example,

3d = 3(73.5 in.) = 220.5 in. = 18.38 ft —> controls

L/4 = 94 ft/4 = 23.5 ft

The maximum shear (see Fig. 3.40) for one wheel line of an HS20-44 truck, V_{LU}, is computed as:

$$V_{LU} = R_L = \frac{(47.62\ ft)(4k)+(61.62\ ft)(16k)+(75.62\ ft)(16k)}{94\ ft} = 25{,}388\ lbs.$$

The distribution factor for this example is 1.00. The shear distributed to the exterior girder is computed as:

$$V_{LD} = 1.00\ (25{,}388) = 25{,}388 \text{ lbs.}$$

The maximum live load shear is then:

$$V_{LL} = 0.50[0.60V_{LU}+V_{LD}] = 0.50[0.60(25{,}388)+25{,}388] = 20{,}311\ lbs.$$

The total vertical shear, V, is computed as:

$$V = V_{DL} + V_{LL} = 26{,}646 + 20{,}311 = 46{,}957\ lbs.$$

The horizontal shear stress for a rectangular cross section is computed from the following:

$$f_v = \frac{1.5\ V}{A} = \frac{1.5(46{,}957)}{900.38} = 78.23\ psi$$

The allowable shear stress, F_{vx}', is found as:

$$F_{vx}' = F_{vx}\ C_M = 165\ psi(0.87) = 144\ psi$$

The actual shear stress is less than the allowable value, so the beam is adequate with respect to horizontal shear.

8. **Check lateral and longitudinal loads.** Not applicable for this example.

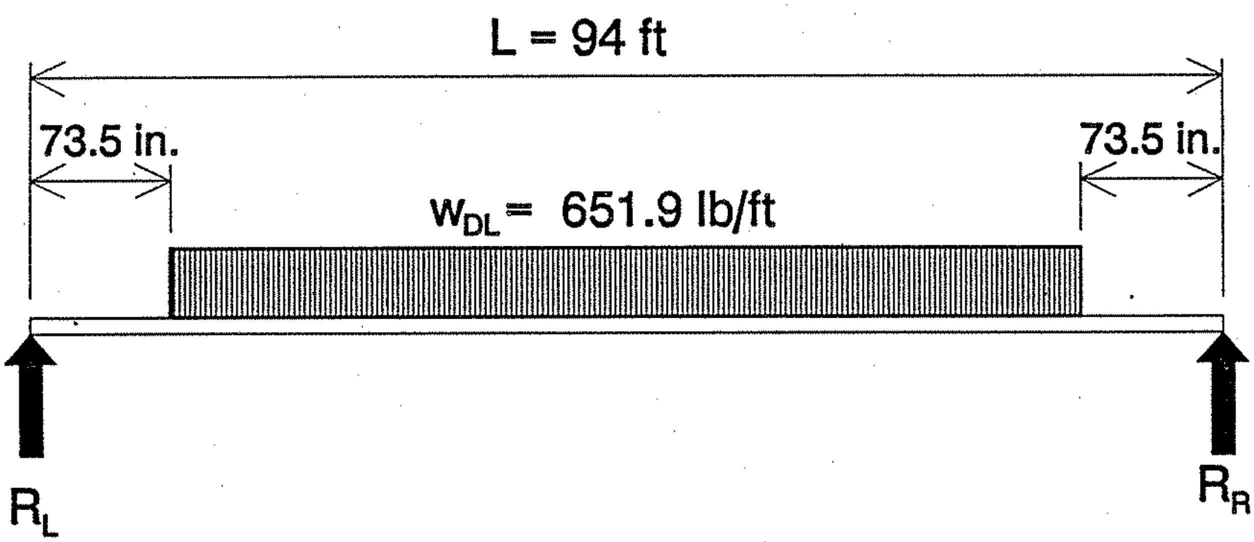


Fig. 3.39. Dead load vertical shear load configuration.

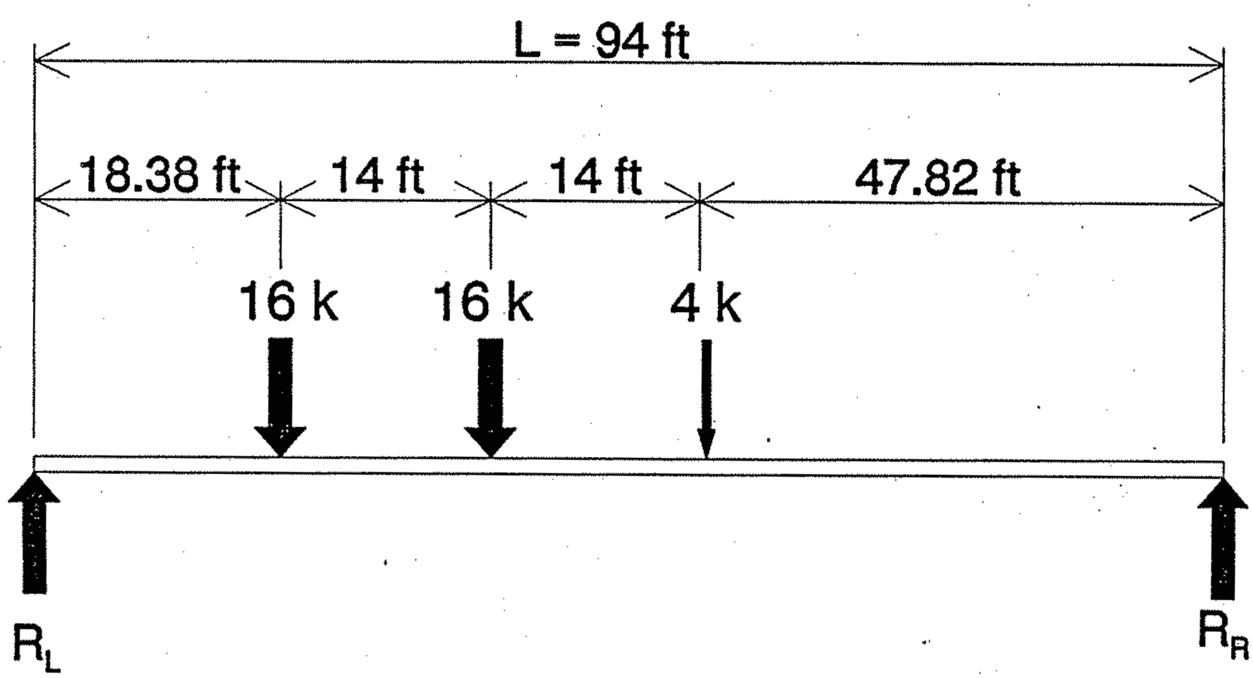


Fig. 3.40. Live load vertical shear load configuration.

9. **Determine bearing length and bearing stress.** The allowable compressive stress perpendicular to the grain, $F_{c\perp}'$, is computed as:

$$F_{c\perp}' = F_{c\perp}\, C_M = 650 \text{ psi } (0.53) = 344.5 \text{ psi}$$

The reaction due to a uniform dead load is:

$$R_{DL} = \frac{w_{DL}\, L}{2} = \frac{(651.9\ lb/ft)(94\ ft)}{2} = 30{,}639 lbs.$$

The distribution factor for reactions is 1.00.

The reaction due to one wheel line may be found in Appendix A of the AASHTO specifications (2).

For a 94 ft span multiplied by the distribution factor:

$$R_{LL} = (1.00)(32{,}430 \text{ lb.}) = 32{,}430 \text{ lbs.}$$

The total reaction is then:

$$R_{total} = R_{DL} + R_{LL} = 30{,}639 + 32{,}430 = 63{,}069 \text{ lbs.}$$

The required bearing length is computed as:

$$L_{bear_{req'd}} = \frac{R_{total}}{b\, F_{c\perp}} = \frac{63{,}069\ lb.}{(12.25\ in.)(344.5\ lb/in^2)} = 14.94\ in.$$

For ease of construction, use a bearing length of 18 in.

The reaction force should be recomputed based on the length of bearing:

$$R_{DL} = \frac{(651.9\ lb/ft)(94\ ft + \frac{18\ in./12}{2})}{2} = 31{,}128\ lbs.$$

The total reaction must also be recomputed:

$$R_{total} = R_{DL} + R_{LL} = 31{,}128 + 32{,}430 = 63{,}558 \text{ lbs.}$$

The total compressive stress at the bearing is computed:

$$f_{c\perp} = \frac{63{,}558\ lb}{(12.25\ in.)(18\ in.)} = 286.03\ psi$$

The actual compressive stress is less than the allowable value, so the bridge is adequate with respect to bearing.

10. **Determine camber.** The dead load deflection is computed from:

$$\Delta_{DL} = \frac{5\, w\, L^4}{384\, E'\, I_x}$$

$$\Delta_{DL} = \frac{5(651.90)[(94\ ft)(12\ in/ft)]^4}{(384)(1.499\ x\ 10^6\ psi)(405{,}338\ in^4)(12\ in/ft)} = 1.88\ in.$$

For a span length greater than 50 ft, use a camber of 2.0 times Δ_{DL}.

Camber = 2.0(1.88) = 3.76 in. —> use 3.80 in.

Summary of Design Values

Geometry and design loadings:

Bridge length, L = 94.0 ft

Roadway width, W, = 24.0 ft

Design live load = HS20-44

Beam combination symbol: 24F-V4, Western species

Type of wearing surface: 3.00 in. asphalt

Beam dimensions and properties:

Beam width, b = 12¼ in.

Beam depth, d = 73½ in.

Beam spacing, s = 5.00 ft

Beam section modulus, S_x = 11,030 in.^{3}

Beam cross sectional area, A = 900.37 in.^{2}

Beam moment of inertia, I_x = 405,338 in.^{4}

Self weight of beam, w_{beam} = 312.63 lb/ft

Stresses and deflections:

Actual bending stress, f_b = 1554 psi

Allowable bending stress, F_b' = 1570 psi

Actual shear stress, f_v = 78 psi

Allowable shear stress, F_{vx} = 144 psi

Actual bearing stress, $f_{c\perp}$ = 286 psi

Allowable bearing stress, $F_{c\perp}'$ = 345 psi

Live load deflection, Δ_{LL} = 1.68 in.

Allowable live load deflection = 3.13 in.

Dead load deflection, Δ_{DL} = 1.88 in.

Design camber = 3.75 in.

3.7.11. Glue Laminated Panel Deck Bridge

3.7.11.1. Background

Longitudinal glulam timber deck bridges consist of a series of glulam panels spanning in the direction of traffic and placed edge to edge across the deck width (see Fig. 3.41). A glulam panel deck is

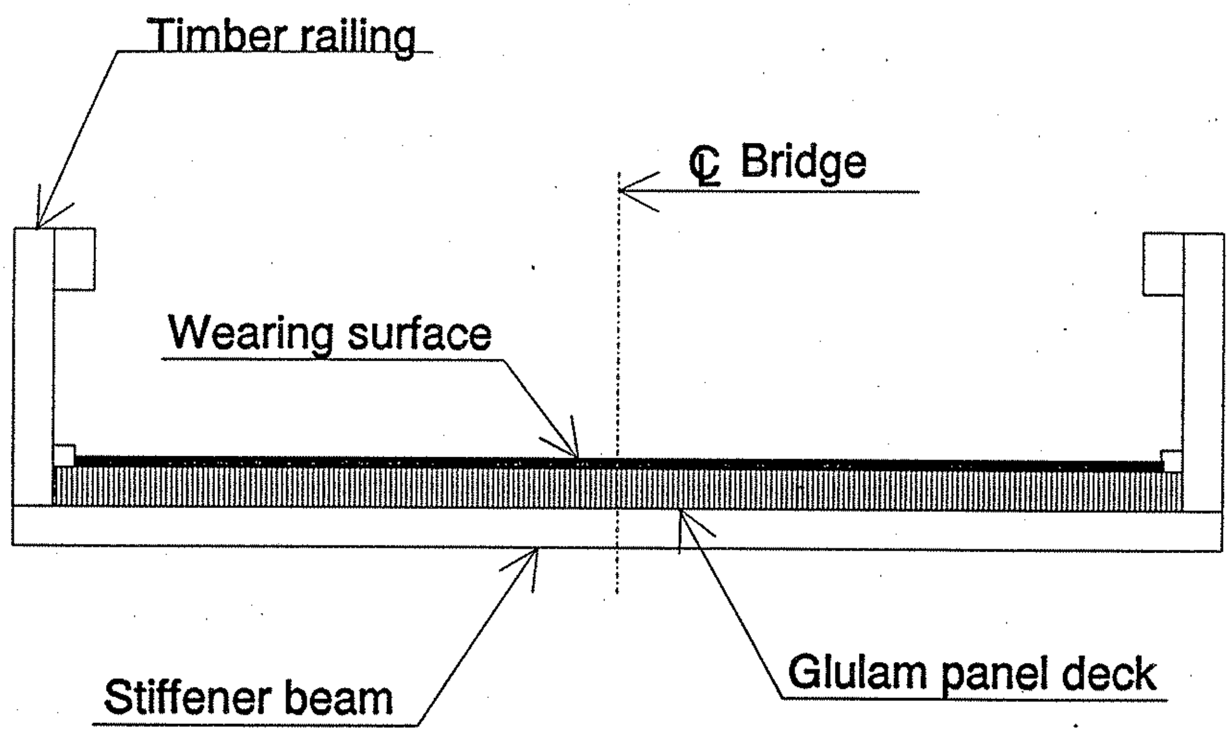


Fig. 3.41. Typical section of glulam timber deck bridge.

able to span up to 35 ft and can be used for either single- or multiple-lane bridges. The panels are usually not connected, however, transverse stiffener beams are bolted below the deck to transfer loads between panels (66).

Deck panels for longitudinal glulam bridges are designed as individual rectangular glulam beams. The portion of the design vehicle live load distribution to each panel is computed as a wheel load fraction (WLF), which is very similar to the distribution factor used for beam design. It is assumed that the entire deck panel is effective in resisting applied loads.

The prefabrication of these panels allows for the pressure treatment of all laminates, which significantly extends the service life of the bridge, and greatly simplifies construction

3.7.11.2. Design Criteria

The design procedures discussed in this section are based on the AASHTO standard specifications (2) and on research performed at ISU (69, 96, 97). The material and engineering properties for the glulam deck bridge are from AITC 117 - Design (4).

3.7.11.3. Design Procedure

Spreadsheet input parameters refer to the spreadsheet in Fig. 3.42.

1. **Define deck geometric requirements and design loads.** The effective span length, L, measured from center-to-center of bearings, and bridge deck width, W, measured from curb-to-curb plus any additional width required for railings, etc., must be determined. The design live loading and timber species must also be specified **(Input A, B, C and D).** If an asphalt wearing surface is to be used, it must be specified and thickness given **(Input E and F).**

2. **Estimate panel dimensions and compute section properties (Input G and H).** Initially, an estimate of the deck thickness and panel width must be made. For economy, the designer should use a standard glulam dimension for deck thickness whenever possible. Panel widths, w_p, are usually 42 to 54 in. in multiples of 1½ in. for western species and 1⅜ in. for Southern Pine. Table 3.33 may be used in estimating the required deck thickness for HS20-44 live loading. The user must input the necessary parameters **(Input G and H)** and the spreadsheet automatically performs the remaining calculations.

Design of Glue-Laminated Timber Deck Bridges

Input deck geometric requirements:

Input bridge length, L =	20.00	ft	A
Input bridge width, W =	29.00	ft	B

Select design live loading:

1) HS 20-44
2) HS 15-44

Please enter number of your choice:	1		C

Select species of timber for use in laminates:

5) Western species
6) Southern Pine

Please enter number of your choice:	5		D

Select type and thickness of wearing surface:

1) No wearing surface
2) Ashphalt wearing surface

Please enter number of your choice:	2		E
Thickness of a/c wearing surface (if any):	3.00	in.	F

Fig. 3.42. Glulam timber deck spreadsheet, input parameters, and example problem.

Input deck panel dimensions and computed section properties:

Initial est. of deck thickness, td =	10.750	in.	G	(based on span length)
Input panel width, wp =	49.50	in.	H	
Panel area, A =	532.13	in^2		
Panel section modulus, Sy =	953.39	in^3		
Panel moment of inertia, Iy =	5124.47	in^4		

Computed panel dead load and dead load moment:

Dead load of timber deck =	184.8	lbs/ft
Dead load of wearing surface =	154.7	lbs/ft
Railing load =	20.0	lbs/ft
Stiffener beam and hardware load =	8.0	lbs/ft
Total design dead load =	367.5	lbs/ft
Design dead load moment, MDL =	18.37	ft-kips

Computed wheel load fraction and live load moment:

Wheel load fraction, WLF =	0.924	
Max. moment due to one wheel line =	80.00	ft-kips
Design live load moment, MLL =	73.92	ft-kips

Computed design moment, MT = (MDL+MLL) =	92.29	ft-kips
Computed design deck bending stress, fb =	1.16	ksi

Select deck combination symbol and computed allowable stress:

1) #2 - western species
2) #48 - southern pine

Please enter number of your choice:	1	I
Tabulated allowable bending stress, Fb =	1800	psi

Fig. 3.42. Continued.

Size factor for depth of members, CF =	1.010	
Allowable Bending stress, Fb' =	1.45	ksi

Deck is sufficient in flexure.

Computed check of live load deflection:

Adjusted modulus of elasticity, E' =	1.42E+06	ksi
Wheel line deflection, DELTAWL =	0.64	in.
Adjusted live load deflection, DELTALL =	0.59	in.
Allowable live load deflection = L/400 =	0.6	in.
Check DELTAL < allowable deflection ?	**Deck is sufficient for deflection.**	

Computed check of horizontal shear:

Dead load vertical shear, VDL =	3345	lb.
Compute LL vertical shear at distance =	2.69	ft.
Vertical shear due to one wheel line =	16,500	lb.
Live load vertical shear, VLL =	15,246	lb.
Horizontal shear, V =	18,591	lb.
Horizontal shear stress, fv =	52.41	psi
Allowable shear stress, Fv =	126.88	psi

Deck is sufficient for shear.

Fig. 3.42. Continued.

Input stiffener material and computed configuration:

Select stiffener material:

1) Steel rolled section
2) Douglas fir-larch (sawn)
3) Hem-fir (north) (sawn)
4) Red pine (sawn)
5) Eastern white pine (sawn)
6) Southern pine (sawn)

Please enter number of your choice: 6 J

Preliminary stiffener spacing =	6.67	ft.
Young's modulus, Estiff =	1,700	ksi
Moment of inertia req'd =	47.06	in^4

Input bearing width and computed bearing stress:

Input width of abutments, in: 12 in. K

WLF for reactions, WLFR =	1.031	
LL reaction/wheel line, RWL =	20.80	kips
Live load reaction, RLL =	21.45	kips
Dead load reaction, RDL =	3.86	kips
Bearing stress, fcp =	42.61	psi
Allowable bearing stress, Fcp' =	296.8	psi

Deck is sufficient for bearing.

Fig. 3.42. Continued.

Summary of Design Values

Geometry and dimensions:

Length, L =	20.00	ft
Width, W =	29.00	ft
Design live loading:	AASHTO	HS 20
Timber species:	Western species	
Deck combination symbol:	#2 - Western species	
Thickness of deck, td =	10.75	in.
Wearing surface:	Asphalt	
Thickness of wearing surface :	3.00	in.

Stresses and deflections:

Bending stress, fb =	1.16	ksi
Allowable bending stress, Fb' =	1.45	ksi
Horizontal shear stress, fv =	52.41	psi
Allowable shear stress, Fv =	126.88	psi
Bearing stress at abutment =	42.61	psi
Allowable bearing stress, Fcp' =	296.8	psi
Live load deflection, deltaL =	0.59	in.
Allowable LL deflection, L/400 =	0.60	in.

Fig. 3.42. Continued.

Table 3.33. Glulam timber deck thicknesses and span lengths.

Deck thickness (in.)	Simple span (ft)	Continuous span (ft)
5 or 5⅛	6	7
6¾	10	12
8½ or 8¾	15	18
10½ or 10¾	21	23
12¼	24	27
14¼	27	31

Based on an estimated panel size, the section properties for the section can be computed:

w_p = panel width (in.)

t_d = panel thickness (in.)

A = panel cross sectional area (in^2)= $w_p t$

S_y = section modulus of panel (in^3) = $\dfrac{w_p t_d^2}{6}$

I_y = moment of inertia of panel (in^4) = $\dfrac{w_p t_d^3}{12}$

3. **Compute panel dead load.** The uniform panel dead load of the deck and wearing surface can be computed using the following unit weights:
   - timber (treated or untreated) = 50 lb/ft^3
   - asphalt or concrete = 150 lb/ft^3

   The spreadsheet performs this calculation automatically using input parameters **E** and **F** which were input in design step 1.

   Table 3.34 may be used for estimating the dead load of the timber deck plus a 3 in. asphalt or concrete wearing surface.

Table 3.34. Dead load of glulam panel deck and wearing surface.

w_p (in.)	DL of deck plus 3 in. wearing surface, (lb/ft³)					
	Thickness of deck, t_p, (in.)					
	5⅛	6¾	8¾	10¾	12¼	14¼
42.0	206.0	229.7	258.9	288.0	309.9	339.1
43.5	213.3	237.9	268.1	298.3	321.0	351.2
45.0	220.7	246.1	277.3	308.6	332.0	363.3
46.5	228.1	254.3	286.6	318.9	343.1	375.4
48.0	235.4	262.5	295.8	329.2	354.2	387.5
49.5	242.8	270.7	305.1	339.5	365.2	399.6
51.0	250.1	278.9	314.3	349.7	376.3	411.7
52.5	257.5	287.1	323.6	360.0	387.4	423.8
54.0	264.8	295.3	332.8	370.3	398.4	435.9

4. **Determine wheel load fraction for live load distribution.** Longitudinal glulam panels are designed as individual members; as no transverse load distribution is assumed and wheel loads are assumed to act as point loads. Lateral load distribution is based on the wheel load factor (WLF), which is based on the panel length and width, and the number of lanes in a given bridge.

For single lane bridges, the WLF is computed as the greater of:

$$WLF = \frac{W_p}{4.25 + \frac{L}{28}} \quad or \quad WLF = \frac{W_p}{5.50}$$

where:

WLF = portion of maximum force or deflection produced by one wheel line that is supported by one deck panel.

W_p = panel width, ft.

L = length of span for simple span decks measured center to center of bearings, ft.

For bridges of two or more traffic lanes, the WLF is the greater of:

$$WLF = \frac{W_p}{3.75 + \frac{L}{28}} \quad or \quad WLF = \frac{W_p}{5.00}$$

The spreadsheet performs the calculations automatically.

5. **Determine dead load and live load moment.** The panel dead load moment can be computed based on the uniform dead load, w_{DL}, previously determined:

$$M_{DL} = \frac{w_{DL} L^2}{8}$$

The live load moment, M_{LL}, is computed by multiplying the maximum moment for one wheel line of the design vehicle by the WLF:

$$M_{LL} = M_{WL}\, WLF$$

where:

M_{LL} = live load moment applied to one panel, in-lb.

M_{WL} = maximum moment produced by one wheel line of the design vehicle, in-lb.

Conservatively, the maximum dead load moment and maximum live load moment can be added, even though the occur at different positions along the span.

The total design moment, M_T, is the sum of the maximum live and dead load moments:

$$M_T = M_{LL} + M_{DL}$$

The spreadsheet computes this step automatically based on the span length and design vehicle chosen.

6. **Compute bending stress and select combination symbol.** The deck bending stress, f_b, computed as the design moment divided by the panel section modulus, S_y, cannot exceed the allowable bending stress adjusted for wet use and size factors:

$$f_b = \frac{M_T}{S_y} \le F_b'$$

where:

F_b' = $F_{by}\, C_M\, C_F$

F_{by} = tabulated bending stress for species of interest

= 1800 lb/in.^{2} for Western Species

= 1750 lb/in.^{2} for Southern Pine

C_M = wet use factor for glulam = 0.80

C_F = size factor for thin panels (see Table 3.35)

If the calculated flexural stress exceeds the allowable value computed above, either the deck thickness, t_d, must be increased, or a higher grade of lumber (one with better material properties) must be utilized. If f_b is significantly less than the allowable stress value, a thinner deck or a lower-grade material may be used. No changes should be made until the live load deflection is reviewed.

The user must select a combination symbol **(Input I)** and the spreadsheet will perform the remaining calculations.

7. **Check live load deflection.** The live load deflection, Δ_{LL}, is a function of the panel moment of inertia and is produced by one wheel line of the design vehicle times the WLF:

$$\Delta_{LL} = \Delta_{WL}\, WLF$$

where:

Δ_{LL} = live load panel deflection, in.

Δ_{WL} = maximum live load deflection produced by one wheel line of the design vehicle, in.

Table 3.35. Size factor for glulam timber deck panels.

t (in.)	C_F
5 or 5⅛	1.10
6¾	1.07
8½ or 8¾	1.04
10½ or 10¾	1.01

The deck live load deflection can be computed by standard elastic analyses, using the glulam modulus of elasticity adjusted for wet-use conditions. For a uniform load, the live load deflection can be found from:

$$\Delta_{LL} = \frac{5\, w_{LL}\, L^4}{384\, E'\, I}$$

AASHTO specifications do not limit live load deflection for glulam timber decks, thus the allowable deflection is left to the designer. Recommended practice limits maximum deflection to L/360.

The spreadsheet performs this calculation and compares the result to the allowable value.

8. **Check horizontal shear.** Because of the relatively large panel area, horizontal shear is rarely a controlling factor; however, it should be checked. Horizontal shear (i.e. vertical shear) due to dead and live load is assumed to be resisted by the total area of the deck panel.

The dead load vertical shear is computed at a distance from the support equal to the thickness, t, and is given by:

$$V_{DL} = w_{DL}\left(\frac{L}{2} - t\right)$$

where:

V_{DL} = dead load vertical shear (lb)

w_{DL} = uniform panel dead load (lb/ft)

The live load vertical shear is computed at a distance from the support equal to the lesser of 3 times the deck thickness (3t) or the span quarter point (L/4), and is equal to the maximum shear due to one design wheel line times the WLF.

$$V_{LL} = V_{WL}\, WLF$$

where:

V_{LL} = live load vertical shear (lb)

V_{WL} = max. vertical shear produced by one wheel line (lb)

The applied shear stress must not exceed the allowable shear stress for the deck combination symbol, given by:

$$f_v = \frac{1.5\,V}{A} \le F_v' = F_{vy}\, C_M$$

where:

V = $V_{DL} + V_{LL}$, lb.

A = panel cross-sectional area, in^2.

C_M = wet-use factor for shear = 0.875.

The maximum dead and live load shear do not occur at the same position along the span, can be combined for simplicity. When $f_v > F_v'$, the only alternatives for the designer are to increase the deck thickness or panel width. In either case, the design procedure must be recycled. The spreadsheet performs this step automatically.

9. **Determine stiffener spacing and configuration.** Transverse stiffener beams typically consist of horizontal glulam beams or shallow rolled steel sections, and are intended to distribute loads to adjacent panels. The design criteria for transverse stiffener beams is based on research done at ISU. Stiffener beams are often used for guardrail post attachment, so rail loads and connection details must also be considered.

   AASHTO specifications require that a transverse stiffener beam be placed at midspan for all deck span lengths, and at intermediate spacings of ≤ 10 ft. An intermediate spacing of 8 ft, recommended by AITC, will be used in the subsequent design.

   The empirical stiffener design requires that the transverse stiffener has a bending stiffness, E'I, of ≥ 80,000 k-in^2. Note that transverse load distribution between panels is influenced more by stiffener spacing than by the bending stiffness, E'I.

   The connection between the stiffener and the deck panels depends on the type of stiffener beams used. A bolt may be placed through the deck and stiffener for both glulam beams and steel channel sections. Deck brackets or steel plates may be used for glulam beams and C-type clips may be used to connect the top flange of a rolled section.

10. **Determine bearing configuration and check bearing stress.** For longitudinal deck bridges, the required bearing area is usually controlled by the required configuration, rather than compressive stresses. A bearing length of 10 to 12 in. is recommended for stability and simplicity.

    The dead load reaction for a glulam deck bridge can be computed using the unit dead load of the panel. The live load reaction is based on a wheel load factor of $W_p/4$, but not less than 1.0. The live load reaction distributed to each panel is the maximum reaction for the design vehicle, multiplied by the WLF.

    The bearing stress due to R_{DL} and R_{LL} must not exceed the allowable value for the panel being used. This may be expressed as:

$$f_{c\perp} = \frac{R_{DL} + R_{LL}}{w_p\, l_b} \le F'_{c\perp} = F_{c\perp}\, C_M$$

where l_b is the length of the panel bearing in inches.

3.7.11.4. Example

An existing bridge is to be replaced with a longitudinal glulam deck bridge. The replacement bridge must have a 20 ft span from center-to-center of bearings, a roadway width of 26 ft, and must support two lanes of AASHTO HS20 live loading. The spreadsheet input and output for this example is shown in Fig. 3.42.

1. **Define deck geometry and design loads.** The bridge span length, L, must be determined. **(Input A).** Bridge width must consider the roadway width, 26 ft, and the curb and railing on each side:

   W = 26 ft + 2 ft + 2(6 in.) = 29 ft. **(Input B)**

   Design live load = one HS-20 wheel line. **(Input C)**

   The designer specifies the species of timber to be used in the laminates. Most common for glulam deck bridges are western species and Southern Pine. For this example western species are selected. **(Input D)**

   The designer must specify if a wearing surface is going to be used for the proposed bridge; if used its thickness needs to be given. For this example, a 3.0 in. asphalt wearing surface is selected. **(Input E and F)**

2. **Estimate panel size and compute section properties.** From Table 3.33, select an initial deck thickness of 10¾ in. **(Input G)** The panel width must be 42 to 54 in., in 1½ in. increments. For this example, select two 51 in. wide panels, and five panels 49½ in. wide, for a total width of 29 ft-1½ in. **(Input H).** The section properties for the 49 1/2 in. panel will be conservatively used for all panels.

   Section properties for this panel can be computed as:

$$t = 10.75 \text{ in.}$$

$$w_p = 49.5 \text{ in.}$$

$$A = w_p\, t = 10.75(49.5) = 532.13 \text{ in}^2$$

$$S_y = \frac{w_p t^2}{6} = \frac{(49.5\ in.)(10.75\ in.)^2}{6} = 953.39\ in^3$$

$$I_y = \frac{w_p t^3}{12} = \frac{(49.5\ in.)(10.75\ in.)^3}{12} = 5124.47\ in^4$$

3. **Compute panel dead load and dead load moment.** From Table 3.34, the dead load of a 49.5 in. panel with a 3 in. asphalt wearing surface can be determined as:

$$w_{DL} = 339.5 \text{ lb/ft.}$$

The dead load due to the railing is assumed to be distributed equally over the entire deck. The dead load due to the railing is:

$$w_{railing} = 2(55 \text{ lb/ft})/7 \text{ panels} = 15.7 \text{ lb/ft.}$$

A conservative estimate of 20 lb/ft has been assumed for the railing load in the remaining calculations. An additional 8 lb/ft should be added to account for stiffener beams and railing hardware.

$$w_{DL/panel} = 339.5 + 20 + 8 = 367.5 \text{ lb/ft.}$$

The dead load moment is computed by:

$$M_{DL} = \frac{(367.5\ lb/ft/1000)(20\ ft)^2}{8} = 18.37\ ft\text{-}k$$

4. **Determine wheel load fraction for live load distribution.** For a two lane bridge, the wheel load fraction can be determined as the larger of the following:

$$WLF = \frac{49.5\ in./12}{3.75 + (20\ ft/28\ )} = 0.924\ WL\ per\ panel$$

$$WLF = \frac{49.5\ in./12}{5.00} = 0.83\ WL\ per\ panel$$

For this example, use WLF = 0.924 WL/panel.

5. **Determine dead load and live load moment.** The maximum live load moment due to one wheel line can be found in Appendix A, AASHTO (2). For the 20 ft span of this example, $M_{wheel\ line}$ = 80,000 ft-lb. The live load moment distributed to each panel is given by:

$$M_{LL} = (0.924 \text{ WL/panel})(80 \text{ ft k}) = 73.92 \text{ ft-k}$$

The total design moment, M_T can be computed as:

$$M_T = M_{DL} + M_{LL} = 18.37 + 73.92 = 92.29 \text{ ft-k}$$

Note, although the maximum dead load and live load moments do not occur at the same point along the span, they can conservatively be added for simplicity.

6. **Compute bending stress and select combination symbol.** The bending stress in the deck is:

$$f_b = \frac{92.29\ ft\text{-}k\ (12)}{953.39\ in^3} = 1.16\ ksi$$

The deck combination symbol chosen for this example is a No. 2 **(Input I).**

The design properties can be found in AITC 117 (4):

F_{by} = 1800 psi	C_M = 0.80
F_{vy} = 145 psi	C_M = 0.875
$F_{c\perp}$ = 560 psi	C_M = 0.53
E = 1.7×10^6 psi	C_M = 0.833

For the deck thickness which has been chosen, the size factor, C_F = 1.01.

The allowable bending stress for the chosen combination symbol is given by:

$F_b' = F_{by}\, C_M\, C_F$ = (1800 psi)(1.01)(0.80) = 1.45 ksi

The actual bending stress is less than the allowable value, so the deck is adequate in flexure.

7. **Check live load deflection.** The deflection coefficient for one line of HS20 wheels can be obtained from simple engineering mechanics or a tabulated value can be used (Table 16-8, Ref. 66). For the 20 ft span in this example the deflection coefficient = 4.61×10^9.

$$\Delta_{WL} = \frac{4.61 \times 10^9}{E' I_y}$$

$E' = E\, C_M = (1.7 \times 10^6)(0.833) = 1.416 \times 10^6$ lb/in.^{2}

$$\Delta_{WL} = \frac{4.61 \times 10^9}{(1.416 \times 10^6)(5124.47)} = 0.64\ in.$$

The deck deflection can then be computed as:

$$\Delta_{LL} = \Delta_{WL}\ \text{WLF} = (0.64\ \text{in.})(0.93) = 0.60\ \text{in.}$$

The allowable live load deflection is L/400 = 0.60 in. The deck is thus sufficient for live load deflection.

8. **Check horizontal shear.** The dead load vertical shear is computed at a distance t from the support:

$$V_{DL} = (367.5\ lb/ft)\left(\frac{20 ft}{2} - \frac{10.75 ft}{12}\right) = 3345\ lb.$$

The live load vertical shear is computed at a distance of 3t or L/4, which ever is less. For this example:

$$3t = 3(10.75\ \text{in.})/12 = 2.69\ \text{ft}$$

$$L/4 = 20\ \text{ft}/4 = 5\ \text{ft}$$

The maximum live load shear for one wheel line of the HS20 vehicle can be found by computing the maximum reaction in the given beam:

$$V_{WL} = R_L = \frac{(16{,}000\ lb)(3.31\ ft + 17.31\ ft)}{20\ ft} = 16{,}500\ lb$$

The live load shear can then be computed as:

$$V_{LL} = V_{WL}\ WLF = (16{,}500\ lb)(0.93) = 15{,}246\ lb.$$

The total horizontal shear can then be found by:

$$V = V_{DL} + V_{LL} = 3345 + 15{,}246 = 18{,}591\ \text{lb}.$$

The horizontal shear stress, f_v, is:

$$f_v = \frac{1.5\ V}{A} = \frac{1.5(18{,}591\ lb)}{532.13\ in.^2} = 52.41\ psi$$

The horizontal shear stress must not exceed the allowable shear stress, which is computed as the tabulated value of F_{vy} times the moisture factor, C_M.

$$F_v' = F_{vy}\ C_M = (145\ \text{psi})(0.875) = 126.88\ \text{psi}$$

The computed shear stress is less than the allowable, thus the deck is adequate for shear.

9. **Determine stiffener spacing and configuration.** It is convenient to choose a stiffener spacing of equal fractions of the span length, as long as the spacing does not exceed 8 ft. For this example, choose a spacing of L/3, or 8 ft-6 in. which is close to the 8 ft limit. The stiffener must have an E'I of greater than 80,000 k-in.^{2}. The designer should determine the stiffener material and compute its adjusted Young's modulus. For the present example, choose a sawn lumber Southern pine stiffener with an adjusted E' value of 1.7×10^6 lb/in.^{2}. **(Input J).**

   The required moment of inertia for the stiffener can be computed as:

$$I_{req'd} = \frac{80{,}000\ k\ in^2}{1.7 \times 10^3\ ksi} = 47.06\ in^4$$

   The selection of a cross section to satisfy this requirement is left to the designer.

10. **Determine bearing configuration and check bearing stress.** The designer must determine the bearing length for the proposed bridge **(Input K)**. For the current example, assume an l_b of 12 in. The reaction due to a uniform dead load is determined as follows:

$$R_{DL} = \frac{(367.5\ lb/ft)(20 + 1\ ft)}{(2)(1000)} = 3.86\ kips$$

The live load reaction is computed as the maximum reaction due to one wheel line of the design vehicle times the WLF.

$$WLF_{reaction} = \frac{W_p}{4} = \frac{4.13\,ft}{4} = 1.031\ WL/panel$$

The maximum reaction for a 20 ft span due to an HS20 loading, R_{WL}, is 20,800 lbs. (Appendix A, AASHTO (4)). The live load reaction is then:

$$R_{LL} = R_{WL}\ WLF = (20.80\ \text{kips})(1.031) = 21.45\ \text{kips}$$

For a bearing length of 12 in., the compressive stress perpendicular to the grain, $F_{c\perp}$, is computed as:

$$f_{c\perp} = \frac{3.86 + 21.45}{(1000)(49.5)(12)} = 42.61\ psi$$

The actual bearing stress must be less than the allowable value, which is computed by:

$$F_{c\perp}' = F_{c\perp}\ C_M = (560\ \text{psi})(0.53) = 296.8\ \text{psi}$$

The stress is less than the allowable value, so the deck is adequate for bearing.

Summary of design values:

Geometry and loading:

Length, L = 20.00 ft

Width of roadway, W = 26.00 ft

Design live load: AASHTO HS20

Timber species: Western Species

Deck combination symbol: No. 2 - Western Species

Deck thickness, t_d = 10¾ in.

Wearing surface: 3 in. asphalt overlay

Stresses and deflections:

Bending stress, f_b = 1.16 ksi

Allowable bending stress, F_b' = 1.45 ksi

Horizontal shear stress, f_v = 52.41 psi

Allowable shear stress, F_v = 126.88 psi

Bearing stress at abutment, $f_{c\perp}$ = 42.61 psi

Allowable bearing stress, $F_{c\perp}'$ = 296.8 psi

Live load deflection, Δ_{LL} = 0.60 in.

Allowable live load deflection, L/400 = 0.60 in.

4. SUMMARY AND CONCLUSIONS

4.1. Summary

The major emphasis of this study was to develop a manual to assist the county engineer in making cost-effective bridge strengthening and replacement decisions. The study was performed in two phases; 1) the determination and prioritization of critical problems on Iowa's secondary bridge system and 2) development of solutions for the problems identified in Phase 1.

The two bridge types with the greatest needs and greatest potential for strengthening are FHWA Type 302 (steel stringer) and FHWA Type 702 (timber stringer) bridges. Methods presented for strengthening the steel stringer bridges include (1) replacement of damaged stringers, (2) respacing existing stringers and adding stringers, (3) increasing the section modulus of existing stringers, (4) developing composite action, (5) replacing existing deck with a lightweight deck, and (6) post-tensioning. For timber stringer bridges, the following methods are included: (1) respacing existing stringers and adding timber stringers, and (2) adding new steel stringers. Each of these procedures are explained and, where applicable, examples are provided. Microcomputer spreadsheets have been developed for items (1), (2), (3) and (6) above for steel stringer bridges and for item (1) above for timber stringer bridges.

Replacement methods for low volume road applications have also been included in the manual. Some of the methods presented are proprietary. Where appropriate, design examples and cost information has been provided. The following bridge replacement types are included (1) precast culvert/bridge, (2) air formed arch culvert, (3) welded steel truss bridge, (4) prestressed concrete beam bridge, (5) Inverset bridge system, (6) precast multiple tee beam bridge, (7) low water stream crossing, (8) corrugated metal pipe culvert, (9) stress laminated timber bridge, (10) glue-laminated timber beam bridge, and (11) glue-laminated panel deck bridge. Microcomputer spreadsheets have been developed for items (9), (10) and (11) above.

For determining the cost effectiveness of each of the methods of strengthening and replacement, a method of economic analysis (equivalent uniform annual cost) as well as cost data have been presented. This allows the engineer to select the most appropriate alternative: (1) replacing the existing bridge or (2) strengthening the existing bridge. The economic model developed allows each alternative to be quantified so that each can be compared in a rational manner.

Information related to inspection of bridges is included in this manual. Before an accurate evaluation of a bridge can be performed, it is imperative that a thorough inspection be performed. Fundamentals related to evaluation of a bridge are also included in the manual.

4.2. Conclusions

1. Steel stringer and timber stringer bridges are the bridge types on the secondary road system in Iowa with greatest potential for cost-effective strengthening methods.
2. County engineers' requests for design aids to assist them in evaluation, strengthening and replacement decisions have been provided.

3. Numerous strengthening procedures have been provided for the two types of bridges (steel stringer and timber stringer) with the greatest potential for strengthening.
4. In situations where strengthening is not cost effective, information has been provided on numerous replacement structures.

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APPENDIX A

Iowa Legal Trucks

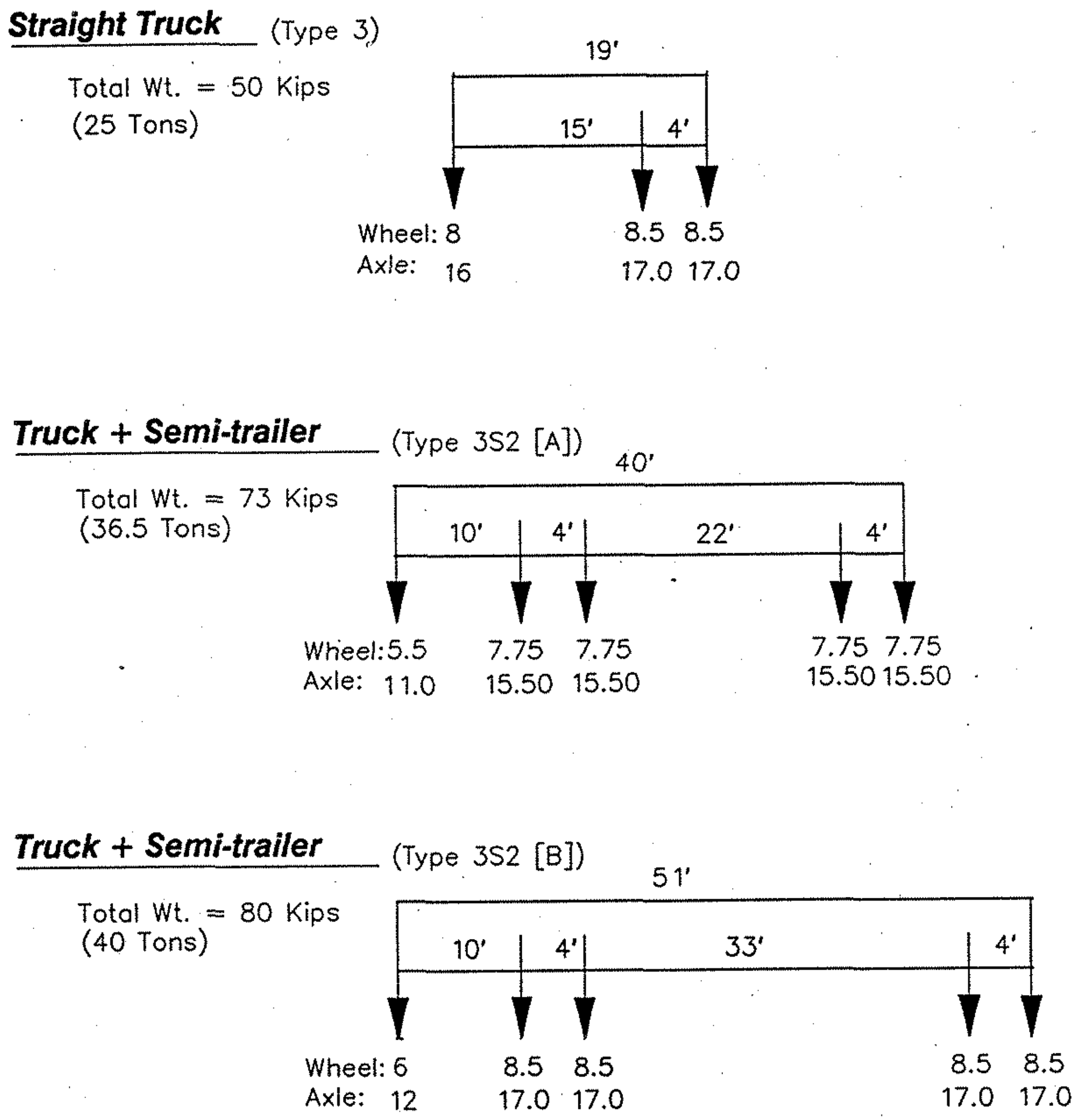
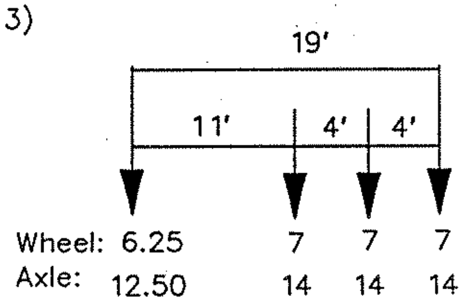


Fig. A.1: Iowa Department of Transportation legal dual axle truck loads. (Wheel and zxle loads are shown in Kips.)

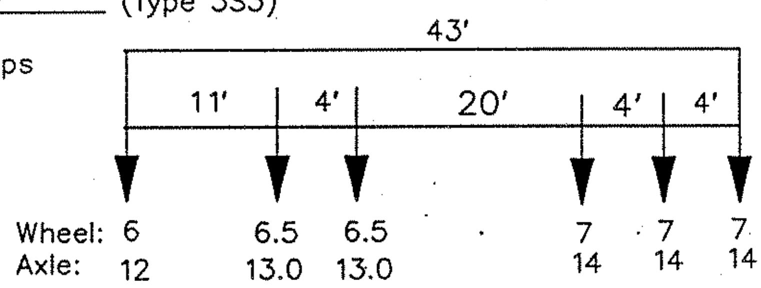
Straight Truck (Type 3)

Total Wt. = 54.5 Kips
(27.25 Tons)



Truck + Semi-trailer (Type 3S3)

Total Wt. = 80 Kips
(40 Tons)



Truck + Semi-trailer (Type 3–3)

Total Wt. = 80 Kips
(40 Tons)

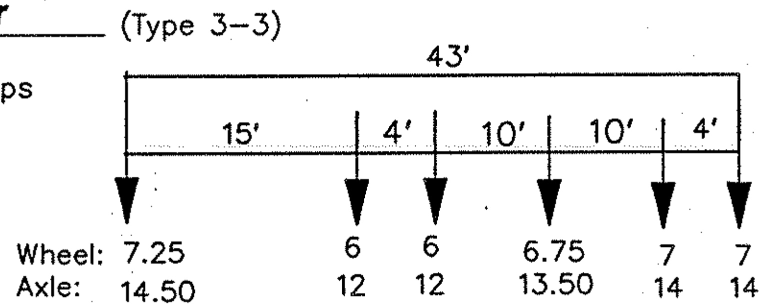


Fig. A.1: Continued.

APPENDIX B

Live Load Moments

LIVE LOAD MOMENTS - LONGITUDINAL BEAMS FOOT-KIPS PER WHEEL LINE						
WITHOUT IMPACT			SPAN FT	WITH IMPACT		
TYPE OF TRUCK*				TYPE OF TRUCK*		
3	3S2(A)	3S2(B)		3	3S2(A)	3S2(B)
27.20	24.80	27.20	10.00	35.36	32.24	35.36
31.30	28.53	31.30	11.00	40.68	37.09	40.68
35.42	32.29	35.42	12.00	46.04	41.98	46.04
39.56	36.07	39.56	13.00	51.42	46.89	51.42
43.71	39.86	43.71	14.00	56.83	51.81	56.83
47.88	43.66	47.88	15.00	62.25	56.76	62.25
52.06	47.47	52.06	16.00	67.68	61.71	67.68
56.25	51.29	56.25	17.00	73.12	66.67	73.12
60.44	55.11	60.44	18.00	78.58	71.64	78.58
64.64	58.94	64.64	19.00	84.04	76.62	84.04
68.85	62.77	68.85	20.00	89.50	81.61	89.50
73.06	67.39	73.89	21.00	94.98	87.61	96.06
77.27	72.64	79.64	22.00	100.45	94.43	103.53
81.49	77.88	85.38	23.00	105.94	101.24	110.99
85.71	83.12	91.12	24.00	111.42	108.06	118.46
89.93	88.37	96.87	25.00	116.91	114.88	125.93
94.15	93.62	102.62	26.00	122.40	121.70	133.40
98.38	98.86	108.36	27.00	127.89	128.52	140.87
102.61	104.11	114.11	28.00	133.39	135.34	148.34
106.84	109.35	119.85	29.00	138.89	142.16	155.81
112.90	114.60	125.60	30.00	146.77	148.98	163.28
119.07	119.85	131.35	31.00	154.79	155.80	170.75
125.25	125.09	137.09	32.00	162.82	162.62	178.22
131.43	130.34	142.84	33.00	170.86	169.44	185.69
137.62	135.59	148.59	34.00	178.90	176.26	193.16
143.81	140.84	154.34	35.00	186.95	183.09	200.64
150.00	146.08	160.08	36.00	195.00	189.91	208.11
156.20	151.33	165.83	37.00	203.05	196.73	215.58
162.39	156.58	171.58	38.00	211.11	203.55	223.05
168.60	161.83	177.33	39.00	219.17	210.37	230.52
174.80	167.07	183.07	40.00	227.24	217.20	238.00

* See Appendix A for weights and configurations of trucks.

LIVE LOAD MOMENTS - LONGITUDINAL BEAMS FOOT-KIPS PER WHEEL LINE						
WITHOUT IMPACT			SPAN FT	WITH IMPACT		
TYPE OF TRUCK*				TYPE OF TRUCK*		
3	3S2(A)	3S2(B)		3	3S2(A)	3S2(B)
181.01	172.32	188.82	41.00	235.31	224.02	245.47
187.21	177.57	194.57	42.00	243.27	230.74	252.83
193.42	182.82	200.32	43.00	250.99	237.23	259.94
199.64	188.07	206.07	44.00	258.70	243.71	267.04
205.85	193.32	211.82	45.00	266.39	250.17	274.12
212.07	198.57	217.57	46.00	274.07	256.63	281.18
218.28	203.81	223.31	47.00	281.74	263.06	288.23
224.50	209.06	229.06	48.00	289.38	269.49	295.27
230.72	216.76	234.81	49.00	297.02	279.04	302.29
236.94	225.69	240.56	50.00	304.64	290.17	309.29
243.16	234.63	246.31	51.00	312.24	301.29	316.28
249.38	243.58	252.06	52.00	319.83	312.38	323.26
255.61	252.53	257.81	53.00	327.41	323.47	330.22
261.83	261.49	263.56	54.00	334.97	334.53	337.17
268.06	270.46	269.30	55.00	342.52	345.58	344.11
274.29	279.43	275.05	56.00	350.06	356.62	351.04
280.51	288.41	280.80	57.00	357.58	367.64	357.95
286.74	297.39	286.55	58.00	365.09	378.64	364.84
292.97	306.37	292.30	59.00	372.58	389.83	371.73
299.20	315.37	298.05	60.00	380.06	400.60	378.60
305.43	324.36	303.80	61.00	387.54	411.56	385.47
311.66	333.36	309.55	62.00	394.99	422.50	392.32
317.89	342.37	315.30	63.00	402.44	433.42	399.15
324.13	351.38	321.05	64.00	409.87	444.33	405.98
330.36	360.39	326.80	65.00	417.29	455.23	412.80
336.59	369.40	332.55	66.00	424.70	466.10	419.60
342.82	378.42	338.29	67.00	432.10	476.97	426.39
349.06	387.44	344.04	68.00	439.49	487.81	433.17
355.29	396.47	353.57	69.00	446.86	498.65	444.70
361.53	405.49	363.29	70.00	454.23	509.47	456.44

* See Appendix A for weights and configurations of trucks.

LIVE LOAD MOMENTS - LONGITUDINAL BEAMS FOOT-KIPS PER WHEEL LINE						
WITHOUT IMPACT			SPAN FT	WITH IMPACT		
TYPE OF TRUCK*				TYPE OF TRUCK*		
3	3S2(A)	3S2(B)		3	3S2(A)	3S2(B)
367.76	414.52	373.01	71.00	461.58	520.27	468.16
374.00	423.56	382.74	72.00	468.92	531.06	479.88
380.24	432.59	392.47	73.00	476.26	541.83	491.58
386.47	441.63	402.22	74.00	483.58	552.59	503.28
392.71	450.67	411.97	75.00	490.89	563.34	514.96
398.95	459.71	421.72	76.00	498.19	574.07	526.63
405.19	468.75	431.49	77.00	505.48	584.78	538.29
411.42	477.90	441.26	78.00	512.76	595.49	549.94
417.66	486.85	451.03	79.00	520.03	606.18	561.58
423.90	495.90	460.81	80.00	527.29	616.85	573.21
430.14	504.95	470.60	81.00	534.54	627.51	584.82
436.38	514.01	480.39	82.00	541.78	638.16	596.43
442.62	523.06	490.19	83.00	549.02	648.80	608.02
448.86	532.12	499.99	84.00	556.24	659.42	619.60
455.10	541.18	509.79	85.00	563.45	670.03	631.17
461.34	550.24	519.60	86.00	570.66	680.63	642.73
467.58	559.30	529.42	87.00	577.86	691.21	654.28
473.82	568.36	539.24	88.00	585.04	701.78	665.82
480.06	577.43	549.06	89.00	592.22	712.34	677.35
486.30	586.49	558.89	90.00	599.39	722.89	688.86
492.54	595.56	568.72	91.00	606.56	733.42	700.37
498.78	604.63	578.55	92.00	613.71	743.95	711.86
505.02	613.70	588.39	93.00	620.86	754.46	723.34
511.27	622.77	598.23	94.00	627.99	764.96	734.82
517.51	631.84	608.08	95.00	635.12	775.44	746.28
523.75	640.92	617.93	96.00	642.25	785.92	757.73
529.99	649.99	627.78	97.00	649.36	796.39	769.17
536.23	659.07	637.63	98.00	656.47	806.84	780.60
542.48	668.14	647.49	99.00	663.57	817.28	792.02
548.72	677.22	657.35	100.00	670.66	827.71	803.43

* See Appendix A for weights and configurations of trucks.

LIVE LOAD MOMENTS - LONGITUDINAL BEAMS
FOOT-KIPS PER WHEEL LINE

WITHOUT IMPACT				SPAN FT	WITH IMPACT			
TYPE OF TRUCK*					TYPE OF TRUCK*			
4	3S3	3-3	HS 20		4	3S3	3-3	HS 20
24.50	24.50	22.40	40.00	10.00	31.85	31.85	29.12	52.00
29.75	29.75	25.77	44.00	11.00	38.67	38.67	33.50	57.20
35.00	35.00	29.17	48.00	12.00	45.50	45.50	37.92	62.40
40.25	40.25	32.58	52.00	13.00	52.32	52.32	42.35	67.60
45.50	45.50	36.00	56.00	14.00	59.15	59.15	46.80	72.80
50.75	50.75	39.43	60.00	15.00	65.97	65.97	51.26	78.00
56.00	56.00	42.88	64.00	16.00	72.80	72.80	55.74	83.20
61.25	61.25	46.32	68.00	17.00	79.62	79.62	60.22	88.40
66.50	66.50	49.78	72.00	18.00	86.45	86.45	64.71	93.60
71.75	71.75	53.24	76.00	19.00	93.27	93.27	69.21	98.80
77.00	77.00	56.94	80.00	20.00	100.10	100.10	74.02	104.00
82.25	82.25	62.08	84.00	21.00	106.92	106.92	80.70	109.20
87.50	87.50	67.23	88.00	22.00	113.75	113.75	87.40	114.40
92.75	92.75	72.38	92.00	23.00	120.57	120.57	94.09	119.60
98.00	98.00	77.53	96.00	24.00	127.40	127.40	100.79	124.80
103.25	103.25	82.69	103.36	25.00	134.22	134.22	107.49	134.37
108.50	108.50	87.86	110.77	26.00	141.05	141.05	114.20	144.00
113.75	113.75	93.01	118.22	27.00	147.87	147.87	120.91	153.69
119.00	119.00	98.17	125.71	28.00	154.70	154.70	127.62	163.43
125.39	124.25	103.33	133.24	29.00	163.01	161.52	134.33	173.21
132.12	129.50	108.50	140.30	30.00	171.75	168.35	141.05	183.04
138.84	134.75	113.67	148.39	31.00	180.50	175.17	147.77	192.90
145.58	140.00	118.84	156.00	32.00	189.25	182.00	154.49	202.80
152.32	145.25	124.01	163.64	33.00	198.01	188.82	161.21	212.73
159.06	150.50	129.18	171.65	34.00	206.78	195.65	167.93	223.14
165.81	155.75	134.35	180.49	35.00	215.55	202.47	174.65	234.63
172.56	161.00	139.60	189.33	36.00	224.32	209.30	181.49	246.13
179.31	166.25	146.94	198.19	37.00	233.10	216.12	191.02	257.65
186.07	171.50	155.13	207.05	38.00	241.89	222.95	201.66	269.17
192.83	176.75	163.31	215.92	39.00	250.67	229.77	212.31	280.70
199.59	182.00	171.50	224.80	40.00	259.46	236.60	222.95	292.24

* See Appendix A for weights and configurations of trucks.

LIVE LOAD MOMENTS - LONGITUDINAL BEAMS FOOT-KIPS PER WHEEL LINE								
WITHOUT IMPACT				SPAN FT	WITH IMPACT			
TYPE OF TRUCK*					TYPE OF TRUCK*			
4	3S3	3-3	HS 20		4	3S3	3-3	HS 20
206.35	187.25	179.69	233.68	41.00	268.26	243.42	233.59	303.79
213.12	192.50	187.88	242.57	42.00	276.93	250.13	244.12	315.20
219.89	197.75	196.06	251.47	43.00	285.33	256.60	254.41	326.31
226.66	203.00	204.25	260.36	44.00	293.72	263.06	264.68	337.39
233.43	208.27	212.44	269.27	45.00	302.09	269.53	274.92	348.46
240.21	215.04	220.63	278.17	46.00	310.44	277.92	285.14	359.51
246.98	221.82	228.81	287.09	47.00	318.78	286.30	295.33	370.54
253.76	229.25	237.00	296.00	48.00	327.10	295.51	305.50	381.55
260.54	237.62	245.19	304.92	49.00	335.41	305.90	315.64	392.54
267.32	246.00	253.38	313.84	50.00	353.70	316.29	325.77	403.51
274.10	254.38	261.56	322.76	51.00	351.97	326.65	335.87	414.46
280.88	262.77	269.75	331.69	52.00	360.23	337.00	345.95	425.39
287.67	271.16	277.94	340.62	53.00	368.47	347.33	356.01	436.30
294.45	279.56	286.13	349.56	54.00	376.70	357.64	366.05	447.20
301.24	287.95	294.31	358.49	55.00	384.92	367.94	376.07	458.07
308.03	296.36	302.67	367.43	56.00	393.12	378.22	386.28	468.93
314.81	304.76	312.60	376.37	57.00	401.30	388.49	398.48	479.77
321.60	313.17	322.54	385.31	58.00	409.47	398.74	410.66	490.59
328.39	321.58	332.48	394.25	59.00	417.63	408.97	422.82	501.39
335.18	330.00	342.42	403.20	60.00	425.77	419.19	434.96	512.17
341.97	338.42	352.36	412.15	61.00	433.90	429.39	447.08	522.94
348.77	347.10	362.30	421.10	62.00	442.02	439.90	459.17	533.69
355.56	356.83	372.25	430.05	63.00	450.12	451.73	471.25	544.42
362.35	366.56	382.20	439.00	64.00	458.21	463.54	483.31	555.14
369.15	376.31	392.14	447.95	65.00	466.29	475.34	495.34	565.84
375.94	386.06	402.09	456.91	66.00	474.35	487.12	507.36	576.52
382.73	395.82	412.05	465.87	67.00	482.40	498.90	519.35	587.18
389.53	405.59	422.00	474.82	68.00	490.44	510.66	531.33	597.83
396.33	415.36	431.95	483.78	69.00	498.47	522.41	543.28	608.47
403.12	425.14	441.91	492.74	70.00	506.49	534.15	555.22	619.09

* See Appendix A for weights and configurations of trucks.

LIVE LOAD MOMENTS - LONGITUDINAL BEAMS FOOT-KIPS PER WHEEL LINE								
WITHOUT IMPACT				SPAN FT	WITH IMPACT			
TYPE OF TRUCK*					TYPE OF TRUCK*			
4	3S3	3-3	HS 20		4	3S3	3-3	HS 20
409.92	434.93	451.87	501.70	71.00	514.49	545.88	567.14	629.69
416.72	444.72	461.83	510.67	72.00	522.48	557.60	579.04	640.28
423.51	454.52	471.79	519.63	73.00	530.46	569.30	590.92	650.85
430.31	464.32	481.75	528.59	74.00	538.43	580.99	602.79	661.41
437.11	474.13	491.71	537.56	75.00	546.39	592.67	614.64	671.95
443.91	483.95	501.67	546.63	76.00	554.33	604.33	626.46	682.48
450.71	493.77	511.63	555.49	77.00	562.27	615.99	638.28	692.99
457.51	503.59	521.60	564.46	78.00	570.19	627.63	650.07	703.49
464.31	513.42	531.56	573.43	79.00	578.11	639.26	661.85	713.98
471.11	523.25	541.53	582.40	80.00	586.01	650.87	673.61	724.45
477.91	533.09	551.50	591.37	81.00	593.90	662.48	685.36	734.91
484.71	542.93	561.47	600.34	82.00	601.79	674.07	697.09	745.35
491.51	552.77	571.44	609.31	83.00	609.66	685.65	708.80	755.78
498.31	562.62	581.40	618.29	84.00	617.52	697.22	720.50	766.20
505.11	572.47	591.37	627.26	85.00	625.38	708.77	732.18	776.61
511.91	582.33	601.35	636.23	86.00	633.22	720.02	743.84	787.00
518.71	592.18	611.32	645.21	87.00	641.05	731.85	755.50	797.38
525.52	602.05	621.29	654.18	88.00	648.88	743.37	767.13	807.75
532.32	611.91	631.26	663.16	89.00	656.69	754.88	778.75	818.10
539.12	621.78	641.24	672.13	90.00	664.50	766.38	790.36	828.44
545.93	631.65	651.21	681.11	91.00	672.30	777.86	801.95	838.77
552.73	641.52	661.18	690.09	92.00	680.09	789.34	813.53	849.09
559.53	651.40	671.16	699.06	93.00	687.86	800.80	825.10	859.40
566.34	661.28	681.14	708.04	94.00	695.64	812.25	836.65	869.70
573.14	671.16	691.11	717.02	95.00	703.40	823.69	848.18	879.98
579.94	681.04	701.09	726.00	96.00	711.15	835.12	859.71	890.25
586.75	690.93	711.07	734.98	97.00	718.90	846.54	871.22	900.52
593.55	700.82	721.04	743.96	98.00	726.63	857.95	882.71	910.77
600.36	710.71	731.02	752.94	99.00	734.36	869.35	894.20	921.01
607.16	720.60	741.00	761.92	100.00	742.08	880.73	905.67	931.24

* See Appendix A for weights and configurations of trucks.

APPENDIX C

Questionnaire Document

Iowa Department of Transportation
Highway Division
Research Project HR - 323

Strengthening/Rehabilitation of Low Volume Highway Bridges

Name of Respondent ______________________________

Organization ______________________________

Address ______________________________

____

____

The purpose of this questionnaire is to determine your experience and practice in the strengthening and/or rehabilitating of low volume highway bridges. For this investigation, bridges with 400 ADT or less are considered low volume. If you wish to comment on any questions or qualify your answers, please use the margins or a separate sheet of paper.

SECTION 1

1. Do you (or your county) have any experience with bridge strengthening?

   Yes _____ No _____

   with bridge rehabilitation (including replacement)?

   Yes _____ No _____

   If yes, please complete Section 2 of this questionnaire.

2. If you answered no to both parts of question 1, the reason bridge strengthening and/or rehabilitation has not been used is:

   - lack of financial resources _____
   - lack of useful guidelines for decision-making _____
   - lack of trained manpower _____
   - other (please explain) ______________________________

   Note: Check all reasons that are applicable.

____

SECTION 2

1. Recognizing that engineering judgement must be used to make many decisions regarding bridge management, do you typically use more formal methodologies for making management decisions (e.g. benefit/cost analysis, equivalent annual cost method, etc.

   Yes ______ No ______

   If yes, which one(s) ? ____________________________________

   ____

   ____

2. Have you developed any design aids, nomographs, computer software etc., that are useful in making bridge rehabilitation decisions? If so, please describe them.

   Yes ______ No ______

   ____

   ____

   ____

   Would you be willing to share them with others?

   Yes ______ No ______

3. Does your county hire a consulting engineer(s) to perform any bridge related structural engineering work?

   Yes ______ No ______

   If so, which firm(s) have you employed? ____________________

   ____

   ____

4. If a consulting engineer is hired, what type of service is most commonly performed by a consulting engineer?

Check all that apply.

Structural analysis ______

Construction inspection ______

Strengthening or rehabilitation ______

Biannual bridge inspection ______

Other (please describe)? ________________________________

____

5. Could your county benefit from some sort of decision making tools or design aids for the rehabilitation or strengthening of existing bridges?

Yes ______ No ______

What sort of tools would be most helpful to your county?

Computer software ______

Nomographs ______

Flow charts ______

Other (please describe)? ________________________________

____

____

____

____

6. If plans or in-house reports are available for any of the strengthening or rehabilitation methods implemented, please indicate who we should contact to obtain copies.

Name/Title: ________________________________

Organization: ________________________________

Address: ________________________________

7. With which types of strengthening procedures does your county have experience?

Bridge type (FHWA #)	Lightweight deck replacement	Provide composite action	Increase transverse stiffness	Strengthen existing members	Add or replace members	Post-tensioning various members	Strengthen critical connections	Develop continuity	Other (please describe):		
Timber stringer (multi beam) (702)											
Steel stringer (multi beam) (302)											
Steel pony (380) or thru (310) truss											
Steel girder & floor beam system (303)											
Other (please describe):											

The following question refers to the structural and cost effectiveness of various strengthening methods. The ratings requested are intended to be a subjective evaluation of the given methods. If a method has met your strengthening objectives, a high rating should be given. Likewise, if a particular method has been relatively inexpensive to perform, a high cost effectiveness rating should be given.

8. Based on your experience, please rate the strengthening methods you have employed. Use a scale of 1-10 (10 being the best.)

Strengthening Method	Cost Effectiveness	Structural Effectiveness
Lightweight Deck Replacement		
Provide composite action		
Increase transverse stiffness		
Strengthen existing members		
Add or replace members		
Post-tension various members		
Strengthen critical connections		
Develop continuity		
Other (please describe):		

9. If it is not possible to make an existing bridge structurally adequate to carry legal loads, but it is possible to strengthen it to carry an increased load, what load would you desire it to carry?

________ Tons

Optionally, rather than specifying a weight which type of vehicle should the bridge be able to support?

Dump truck _____

Dump truck with pup _____ Garbage truck _____

Farm vehicle _____ School bus _____

Type of farm vehicle _________________

Other (please describe) _________________________

10. Do you know of anyone who might be able to supply additional information regarding the rehabilitation and/or strengthening of low volume bridges (e.g. consulting engineers, highway officials, etc.)?

Name: ____

Organization: ____

Address: ____

Name/Title: ____

Organization: ____

Address: ____

11. Have you used, or are you familiar with the National Cooperative Highway Research Program Report #293, Methods of Strengthening Existing Highway Bridges?

Yes _____ No _____

12. Previous studies have determined that the four bridge types listed below account for over 93% of the structurally deficient bridges on the secondary highway system in Iowa.

Please complete the table below.

Bridge type (FHWA #)	Indicate the bridge type for which you would most like to see strengthening methods developed.	Indicate which bridge types would most benefit from a combination of strengthening and posted weight/speed restrictions.	Indicate which bridge types would be least likely to benefit from strengthening or rehabilitation methods.
Timber stringer (multi beam) (702)			
Steel stringer (multi beam) (302)			
Steel pony (380) or thru (310) truss			
Steel girder & floor beam system (303)			
Other (please describe):			

Please return completed questionnaire
in the enclosed envelope by June 25, 1990 to:

Dr. T. J. Wipf
Dept. of Civil and Construction Engineering
420 Town Engineering
Iowa State University
Ames, IA 50011