National Cooperative Highway Research Program

NCHRP Synthesis 249

Methods for Increasing Live Load Capacity of Existing Highway Bridges

A Synthesis of Highway Practice

Transportation Research Board
National Research Council

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Methods for Increasing Live Load Capacity of Existing Highway Bridges

ROGER A. DORTON, Ph.D., P. Eng.
Buckland & Taylor Ltd.
and
RANJIT REEL, P. Eng.
Ministry of Transportation, Canada

Topic Panel

JOHN J. AHLSKOG, *Bridge Consultant, Corrales, New Mexico*
MARK J. BLOSCHOCK, *Texas Department of Transportation*
D.W. (BILL) DEARASAUGH, *Transportation Research Board*
ROBERT P. DI BARTOLO, *New Jersey Department of Transportation*
JAMES J. HILL, *Minnesota Department of Transportation*
HARRY LEE JAMES, *Mississippi Department of Transportation*
DENNIS R. MERTZ, *University of Delaware*
ROBERT MIHALEK, *Federal Highway Administration*

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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

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PREFACE

A vast storehouse of information exists on nearly every subject of concern to highway administrators and engineers. Much of this information has resulted from both research and the successful application of solutions to the problems faced by practitioners in their daily work. Because previously there has been no systematic means for compiling such useful information and making it available to the entire community, the American Association of State Highway and Transportation Officials has, through the mechanism of the National Cooperative Highway Research Program, authorized the Transportation Research Board to undertake a continuing project to search out and synthesize useful knowledge from all available sources and to prepare documented reports on current practices in the subject areas of concern.

This synthesis series reports on various practices, making specific recommendations where appropriate but without the detailed directions usually found in handbooks or design manuals. Nonetheless, these documents can serve similar purposes, for each is a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems. The extent to which these reports are useful will be tempered by the user's knowledge and experience in the particular problem area.

FOREWORD

By Staff
Transportation Research Board

This synthesis will be of interest to state DOT bridge design and structural engineers, bridge consultants, and others involved in applied and research methods for increasing the live load capacity of existing highway bridges. The synthesis describes the current state of the practice for the various methods used to increase the live load capacity of existing highway bridges. This is done predominantly for bridges in the short- to medium-span range. Information on the more common bridge material types (e.g., steel, reinforced concrete, prestressed concrete and wood) is presented. There is an emphasis on superstructure rather than substructure strengthening.

Administrators, engineers, and researchers are continually faced with highway problems on which much information exists, either in the form of reports or in terms of undocumented experience and practice. Unfortunately, this information often is scattered and unevaluated and, as a consequence, in seeking solutions, full information on what has been learned about a problem frequently is not assembled. Costly research findings may go unused, valuable experience may be overlooked, and full consideration may not be given to available practices for solving or alleviating the problem. In an effort to correct this situation, a continuing NCHRP project, carried out by the Transportation Research Board as the research agency, has the objective of reporting on common highway problems and synthesizing available information. The synthesis reports from this endeavor constitute an NCHRP publication series in which various forms of relevant information are assembled into single, concise documents pertaining to specific highway problems or sets of closely related problems.

This report of the Transportation Research Board presents information on the methods used during the past 10 years to increase the live load capacity of existing highway bridges. The information of North American agencies was mostly collected from a survey of transportation agencies; information on international practices was collected through a literature search. A specific focus on the physical methods to increase capacity, and the selection of the most appropriate methods is included. In addition, the extent of bridge management systems used in identifying needs and making selections between strengthening and replacement is reported. Finally, several project case studies and profiles of innovation using emerging technology are presented.

To develop this synthesis in a comprehensive manner and to ensure inclusion of significant knowledge, the Board analyzed available information assembled from numerous sources, including a large number of state highway and transportation departments. A topic panel of experts in the subject area was established to guide the research in organizing and evaluating the collected data, and to review the final synthesis report.

This synthesis is an immediately useful document that records the practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As the processes of advancement continue, new knowledge can be expected to be added to that now at hand.

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This study was managed by Stephen F. Maher, P.E., Senior Program Officer, who worked with the consultant, the Topic Panel, and the Project 20-5 Committee in the development and review of the report. Assistance in Topic Panel selection and project scope development was provided by Sally D. Liff, Senior Program Officer. Linda S. Mason was responsible for editing and production, with assistance from Rebecca Heaton.

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METHODS FOR INCREASING LIVE LOAD CAPACITY OF EXISTING HIGHWAY BRIDGES

SUMMARY

The need to increase the live load capacity of existing highway bridges is becoming more important, as more trucks are using the highways and their weights frequently exceed the live loads for which older bridges were designed. Sixty-three percent of the North American transportation agencies responding to a survey for this synthesis expect this need to increase as the infrastructure ages, and capital for bridge replacement remains in short supply. The methods being used or developed to increase capacity need to be documented to help agencies address these challenges.

NCHRP Report 293: Methods of Strengthening Existing Highway Bridges, which was published in 1987 and which included a strengthening manual, is largely still valid for this synthesis study. Therefore, agencies and consultants were asked to report on their work during the past 10 years so that new and innovative methods could be identified.

To evaluate load carrying capacity, agencies in the United States generally use working stress or load factor methods, with little application of the load and resistance factor approach, which is particularly suited to evaluation and could raise calculated capacity. If simple analytical techniques produce low ratings, most agencies apply more sophisticated methods of analysis, which frequently produces a higher rating. The use of full-scale load testing to evaluate live load capacity appears to be on the increase.

Standard strengthening methods have not changed significantly in the past 10 years. The most frequently used methods are developing composite action, increasing the resistance of individual members, and adding post-tensioning. Agencies reported that 1,268 bridges have been strengthened for increased live load since 1987. This figure is questionable, however, because some agencies did not report totals, and a number of bridges were rehabilitated as a result of deterioration, not as a result of live load inadequacy.

Although few new methods have been devised and put into practice recently, some interesting applications of established techniques were reported or identified in the literature. These include the increased use of orthotropic decks and stress-laminated wood decks to reduce dead weight and, hence, increase live load capacity. The addition of external post-tensioning to steel box girders and segmental concrete boxes has been used in Europe for some years, and is now being applied to segmental concrete construction in North America. The epoxy bonding of steel plates to concrete bridges has been widely used in the United Kingdom, continental Europe, and Japan, but has not been adopted in North America, where there is considerable interest in developing advanced composite materials to replace the steel. There is concern in Europe and North America about the durability of the bonding interface, and the steel plates are heavy and difficult to apply in the field; therefore, the future of this technique appears limited.

Seismic retrofitting of piers for increased strength and ductility is a major program, particularly on the West Coast. In California, steel jacketing of columns is now an established procedure with the state department of transportation (CALTRANS), and wrapping columns with fiber-reinforced plastic sheets is becoming a viable alternative. Very few agencies

check the effect on seismic response when they strengthen the superstructure to increase live load capacity.

Most agencies have an operating Bridge Management System, but these systems are used infrequently for making decisions on bridge strengthening. These systems currently lack sufficient data for this purpose, and more information on first cost and life-cycle costs for a variety of bridge types, materials, and strengthening methods are needed if they are to be used more fully. A decision matrix containing relative assessments of suitability, first cost, and effects on traffic for a variety of bridge types, materials, and strengthening methods is presented in chapter 2. This matrix is shown for information only, but may be useful in making decisions on bridge strengthening.

There are some interesting new developments in lightweight decks and the bonding of fiber-reinforced plastic sheets for component strengthening. A new aluminum deck system was used on two projects in 1996, and a fiber-reinforced concrete deck, which has no reinforcing steel, has been used on two demonstration projects. The use of bonded fiber-reinforced plastic sheets to strengthen concrete components is in the early stages of field testing in North America, however, considerable research is being conducted. The preferred composite materials are carbon fibers with an epoxy resin. This research is a follow-up to European developments in this field, primarily those in Switzerland. More investigations of long-term durability, ductility at the ultimate limit state, resin embrittlement over time, and behavior at extreme temperatures are needed. With satisfactory answers to these concerns, this method of strengthening is likely to be used in the future.

To ensure that optimum decisions are made regarding increasing bridge capacity for live load, more performance and life-cycle cost data need to be collected and included in existing bridge management systems. Useful decision matrices incorporating these data can then be developed for these systems, which should eventually include data on fiber-reinforced plastics. Strengthening design using fiber-reinforced plastics is more complicated than designs using traditional materials, because of the many variables. There is a need for standards across the industry and for simpler design methods. For consistency and economy, there is a need for live load evaluation and strengthening design to be in a load and resistance factor format, similar to the AASHTO-LRFD Bridge Design Specification (1994). The upcoming NCHRP Project 12-46, Manual for Condition Evaluation and Load Rating of Highway Bridges Using Load and Resistance Factor Philosophy, represents an important step in this direction.

CHAPTER ONE

INTRODUCTION

BACKGROUND

This synthesis reports on the methods currently used to increase live load capacity of existing highway bridges. It covers methods used in the past 10 years by North American transportation agencies that responded to a survey, and internationally through a literature search, for bridges predominately in the short- to medium-span range. The more common bridge types and construction materials (e.g., steel, reinforced concrete, prestressed concrete, and wood) are addressed with emphasis on superstructure strengthening rather than substructure.

The need to increase live load capacity of many bridges has become increasingly important because of the maturing infrastructure, a reduction in available capital, and the demand for heavier truck loads, frequently in excess of the bridge design capacity. In the opinion of 63 percent of the North American agencies surveyed, the need for strengthening to increase live load capacity is increasing. In the past 20 years, concern over bridge deterioration has increased and all agencies have carried out major bridge rehabilitation programs. Bridge rehabilitation often results in an upgrading of capacity, but this synthesis concentrates on work specifically carried out to increase live load capacity above the original design load, although it is sometimes difficult to separate the two procedures.

The rated live load capacity of a bridge can often be increased by the use of evaluation techniques and analysis methods more sophisticated than the standard procedures. These methods, along with full-scale load testing, are reviewed. However, the main thrust of this synthesis is to cover physical methods of modifying to increase capacity, and the selection of the most appropriate methods. The extent to which bridge management systems are used to identify needs and choose between strengthening and replacement is also reported. There was insufficient cost data from the agencies to evaluate different strengthening methods on a cost basis, but perceived advantages and disadvantages of the various methods are shown in tabular form.

One of the important objectives was to identify and evaluate innovative methods being used or under development. Only seven of the 47 responding North American agencies reported the development of innovative methods, but several expressed interest in the use of advanced composite materials (ACM) such as carbon fiber-reinforced plastic (CFRP) sheets. Although ACM sheets are now used for seismic retrofit of columns, their use for superstructure strengthening in North America is still in the experimental stage. These materials are more widely used in Europe and Japan, therefore the literature and on-going research reported in this synthesis place particular emphasis on overseas work on ACM, along with the growing research effort in North America on this topic.

PREVIOUS WORK

In 1987 the National Cooperative Highway Research Program (NCHRP) issued *Report 293: Methods of Strengthening Existing Highway Bridges* (*1*), which was the culmination of Phase I research for NCHRP Project 12–28 of the same name, conducted by Iowa State University. That report detailed the results of a study of various methods of strengthening highway bridges, and included a thorough review of pertinent U.S. and international literature to determine the methods being used at that time and to discover innovative ideas being considered. Much of this work is still valid, and most of the strengthening methods identified are still commonly used. Report 293 has been used as the starting point for this synthesis, which in effect reviews and updates the methods used since the Iowa State University survey of 1986. The 379 references given in the bibliography of Report 293 are very comprehensive and are not repeated in this synthesis; the references herein cover only publications since 1986.

The major part of Report 293 consists of a 60-page strengthening manual for use by practicing engineers, which describes the most effective techniques for strengthening existing highway bridges, and includes a cost-effectiveness analysis. This manual will continue to be of practical use for some years to come. An updating or preparation of such a manual is not included in the scope of this synthesis.

NCHRP Project 12–28 originally envisaged a Phase II to pursue promising new techniques that were not fully developed. Most new techniques identified would not have had sufficiently widespread application to warrant conducting Phase II. However, the epoxy bonding of steel plates appeared to hold promise, particularly for concrete bridges, and a short report on overseas applications, on-going research, and adhesive problems was included as Appendix B of Report 293 (*1*). The use of bonded steel plates is considered further in this synthesis.

CURRENT SITUATION

Six methods for increasing live load capacity have been used for many years and can be considered standard methods. They were all identified in 1987 (*1*), have continued to be used since, and are expected to be used in the future. New techniques to achieve the desired results may be developed, but are still likely to fall within these six categories:

- Reduce dead load,
- Develop composite action,
- Increase transverse stiffness,
- Improve member strength,

TABLE 1

NUMBER OF NORTH AMERICAN AGENCIES STRENGTHENING BRIDGES FOR LIVE LOAD CAPACITY BETWEEN 1986 AND 1996, BY METHOD AND MATERIAL

	Superstructure Material					
Methods for Increasing Live Load Capacity	Steel	Reinforced Concrete	Pre-tensioned Concrete	Post-tensioned Concrete	Wood	Total Number
Reduce dead load	11	4	3	–	4	22
Develop composite action	22	3	2	–	–	27
Increase transverse stiffness	3	2	–	1	–	6
Improve member strength	26	4	4	–	3	37
Post–tension members	12	2	1	1	3	19
Develop continuity	7	1	2	–	–	10
Other methods	3	4	1	1	2	11

- Post-tension members, and
- Develop continuity.

A survey questionnaire (Appendix A) was sent to the state departments of transportation in the United States, and their provincial counterparts in Canada, and responses were received from 47. The number of these agencies that have carried out bridge strengthening for live load capacity since 1986 is shown in Table 1, categorized by strengthening method and superstructure material. Of the six standard methods, improving member strength is used by the most agencies, followed by developing composite action, reducing dead load, and post-tensioning members, with developing continuity and increasing transverse stiffness the least used. The Other Methods category includes such techniques as adding longitudinal beams or reducing span lengths by constructing extra piers. The agency interest in steel bridge superstructure strengthening exceeds the combination of all the other materials. This is probably because of the age of bridges needing upgrading and the relative ease with which steel structures can be strengthened. Masonry was included as a material in the survey, but as there was zero response, it was dropped from further consideration.

Only seven of the 47 responding agencies indicated that they had developed any innovative techniques for increasing live load capacity of bridges. No use of the bonding of steel plates for strengthening was reported. The expectation in 1987 (*1*) that this method would have considerable promise for use following further research has not been realized. The literature search identified only two research papers on the subject in the United States. The method has been used overseas, where 25 papers were noted, mostly from the United Kingdom. One paper (2) listed 31 bridges that have been strengthened by steel plate bonding in the United Kingdom between 1974 and 1994. Some of the concerns about the method have not been fully resolved however, including the need for high-quality workmanship and materials (*3*), anchorage at the ends of the plates (*4*), premature plate peeling (*5*), long-term performance of the adhesive (*6*), and possible plate corrosion (*7*). Practical problems arise because the heavy steel plates are difficult to handle and support, particularly during installation over traffic. These handling and weight problems are minimized by the use of ACM plates or sheets, which compensates, to some extent, for the much higher material costs of ACM. Much more research is now centered on composite materials such as CFRP, rather than bonded steel plates, with 62 papers located, including 17 from the United States. It now appears unlikely that bonded steel plates will find any significant use in North America, and the most likely new techniques for strengthening will involve fiber-reinforced plastic (FRP) plates and sheets. The situation regarding FRP in 1996 is very similar to that of bonded steel plates 10 years ago, but with a greater expectation of their successful application in North America.

The development of bridge management systems has been a major activity in recent years, and most agencies now have such a system in place. Ideally, a bridge management system should be able to identify the need to strengthen a bridge, include relevant cost data, and, through a decision matrix, be able to recommend whether to strengthen or replace the structure. The survey indicates that little use is being made of bridge management systems by agencies to decide these questions to date. Only four agencies stated that their bridge management systems had identified the need to increase capacity for the projects they reported on. Six agencies, or 13 percent of those replying, indicated that their systems have decision matrices for increasing capacity, the two systems mentioned being PONTIS and BRIDGIT (*8*).

If normal analytical methods show the need to increase capacity, the use of more sophisticated analytical methods can sometimes raise the predicted live load capacity and eliminate or reduce the need to upgrade. Forty-three percent of the agencies reported using more sophisticated analytical methods, with finite element methods (FEM) being the most commonly used.

Full-scale load testing of bridges has often revealed load capacity beyond the analytical prediction (*9*). To make load test data available more widely, and have it used for evaluation purposes where possible, NCHRP issued Report 306 (*10*) in 1988. The report also indicated when load testing may still be appropriate. The survey questionnaire indicates that 16 agencies have carried out load tests on individual bridges to upgrade their rating in the past 10 years. Load testing appears to be on the increase, a trend that should continue with the availability of an NCHRP manual to determine load rating by simple load testing techniques (*11*).

PROCEDURES

This synthesis was prepared following a detailed literature review, and after the return of survey questionnaires sent to highway agencies in the United States and Canada, and to selected consultants. A number of researchers were contacted directly in North America and Europe for the latest information on newly developing techniques. The latest CD-rom from the Ministry of Transportation of Ontario (MTO) was used for the initial search for papers since 1986. This CD includes the TRB-TRIS list and the IRRD-OECD list, to provide the needed North American and international coverage. In addition, the proceedings of several recent conferences (*12–15*) were obtained for review to ensure the currency of data, particularly in relation to ACM developments. Similarly, relevant journal volumes issued in 1996 were searched. The articles listed in the references are only those published in English since 1986.

The survey questionnaire (Appendix A) was sent to the highway agencies in 50 states, Puerto Rico, the District of Columbia, the FHWA, the Canadian Federal Government, and nine provinces. The questionnaire was also sent to a number of consulting firms in the United States and Canada. Replies were received from 76 percent of the agencies contacted, and from 27 consulting offices.

CHAPTER TWO

BRIDGE EVALUATION AND ANALYSIS

This chapter describes the procedures used to evaluate the load-carrying capacity of existing bridges. Inspection methods, condition surveys, analysis methods, bridge design, and evaluation codes and load testing used for evaluating the capacity of bridges are described. Methods to predict the future life of the bridges and decision criteria used to increase the load-carrying capacity are given. The results of the survey of current practices are summarized.

EVALUATION PROCESS

The evaluation process for calculating the live load capacity of a bridge consists of

- Determining the need for the evaluation;
- Conducting an inspection and condition rating to determine the condition of the bridge, its components, and material properties;
- Choosing the code, evaluation method, and analysis method to be used; and
- Calculating the live load capacity.

If the calculated live load capacity is too low, changes to the evaluation process may be made, including better assessment of the material properties, use of a different code, rating methodology, or refined methods of analysis. If the load-carrying capacity is still low, load testing the bridge may be selected to ascertain its behavior or proof load.

Need for Evaluation

The *Manual for Condition Inspection of Bridges* (*16*) recommends that load rating for live load capacity for all bridges should be reviewed and updated every 2 years based on the findings of the latest routine inspection. The need for an evaluation may be based on any of the following:

- Defects, deterioration, damage, or scour affecting the load-carrying capacity;
- A change in design or evaluation load specifications, road classification, or review of an existing load posting or previous evaluation;
- Rehabilitation affecting load-carrying capacity;
- Application for a permit to allow a controlled vehicle to use the bridge; and
- An unsatisfactory serviceability or fatigue performance.

Inspection Methods

Routine inspections must fully satisfy the requirements of the *National Bridge Inspection Standards* (*17*), and the Federal Highway Administration (FHWA) *Technical Advisory—Revisions to the National Bridge Inspection Standards* (*18*). These inspections are generally visual and are conducted from the deck, ground, water, and permanent walkway, if present. Each Canadian province sets its own standards and frequency for routine inspections; however, they generally follow the 2-year cycle for bridges under their jurisdiction. Detailed procedures for carrying out routine inspections are provided in the following:

- American Association of State Highway and Transportation Officials (AASHTO) *Manual for Bridge Maintenance (19)*
- FHWA, *Bridge Inspector's Manual for Movable Bridges (20)*
- FHWA, *Inspection of Fracture Critical Bridge Members (21)*
- FHWA, *Culvert Inspection Manual (22)*
- FHWA, *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* (*23*)
- FHWA, *Bridge Inspectors Training Manual 90 (24)*
- FHWA, *PONTIS Users' Manual (25)*
- Ministry of Transportation, Ontario (MTO), *Ontario Structure Inspection Manual (26).*

All the routine bridge inspection procedures require the rating of the condition of the bridge and/or its elements on a numerical scale of 0 to 9, 1 to 5, or 1 to 6, etc. PONTIS (*25*) rates the environment the element is exposed to, and the percentage of the element in any condition state. Routine inspections may identify the need for in-depth inspections needed to evaluate live load capacity, underwater investigations, maintenance, rehabilitation, and replacement programs.

In-depth Inspections and Condition Surveys

An in-depth inspection is a close-up, hands-on inspection of one or more members of the bridge. A condition survey is the measurement and recording of the extent of material and performance deficiencies and deterioration, and the establishment of the material properties of the component. In-depth inspections and condition surveys are carried out where routine inspections do not provide sufficient information about the extent of deficiency, deterioration, or material properties needed to evaluate the live load capacity of the bridge. They are also required for fatigue-prone details and fracture-critical members to identify any fatigue cracks. Condition surveys may involve destructive and nondestructive techniques. Detailed procedures for carrying out in-depth inspections and condition surveys are given in the following:

- FHWA, *Non-destructive Testing Methods for Steel Bridges (27)*
- FHWA, *Technical Advisory—Evaluating Scour at Bridges (28)*
- FHWA, *Underwater Inspection of Bridges* (*29*)
- MTO, *Ontario Highway Bridge Design Code* (OHBDC) (*30*).

Bridge Design and Evaluation Codes, Guides, and Technical Requirements

The codes, manuals, guides, and technical requirements used for the design and evaluation of existing bridges are listed below.

- AASHTO, *Standard Specification for Movable Highway Bridges (31)*
- AASHTO, *Guide Specification for Strength Evaluation of Existing Steel and Concrete Bridges (32)*
- AASHTO, *Guide Specification for Fatigue Evaluation of Existing Steel Bridges (33)*
- AASHTO, *Manual for Condition Evaluation of Bridges (34)*
- AASHTO, *LFRD* (Load and Resistance Factor Design) *Bridge Design Specifications, S.I First Edition*, with annual updated interim specifications *(35)*
- AASHTO, *Standard Specification for Highway Bridges*, with annual updated interim specifications *(36)*
- *NCHRP Report 292: Strength Evaluation of Existing Concrete Bridges (37)*
- *NCHRP Report 301: Load Capacity Evaluation of Existing Bridges (38)*
- *NCHRP Report 319: Recommended Guidelines for Redundancy Design and Rating of Two-Girder Steel Bridges* (*39*)
- *NCHRP Report 352: Inelastic Rating Procedures for Steel Beam and Girder Bridges (40)*
- Canadian Standards Association, CAN-CSA-S6, Supplement No. 1, *Existing Bridge Evaluation* (*41*)
- MTO, *Ontario Highway Bridge Design Code (30)*
- BD 46/92, *Technical Requirements for the Assessment and Strengthening Programme of Highway Structures, Stage 2—Modern Short Span Bridges (42)*
- BD 50/92, *Technical Requirements for the Assessment and Strengthening Programme of Highway Structures, Stage 3—Long Span Bridges (43)*

RATING LEVELS

In the United States, bridges are rated at two levels—inventory and operating (*34*). In Canada, OHBDC (*30*) allows two evaluation levels depending on how often the bridge is evaluated. CAN-CSA-6, Supplement 1 (*41*), allows a multiple number of evaluation levels based on a number of factors from which the engineer can choose.

Inventory Rating Level

For working stress design (WSD), the inventory rating level corresponds to allowable stresses that are similar to the design stresses. For load factor design (LFD), the inventory rating level has a live load factor of 2.17, which is similar to the design live load factor.

Operating Rating Level

For WSD, the operating rating level corresponds to stresses that are higher than the design stresses by approximately 30 percent. For LFD, the operating level has a live load factor of 1.3, which is lower than the design live load factor of 2.17. The bridge life may be reduced if the operating level becomes the norm (*34*).

ANALYSIS METHODS AND EVALUATION PROCEDURES

The analysis methods and evaluation procedures commonly used include the three types described below.

Linear elastic methods of analysis using one of the following evaluation procedures:

- Working stress design, also called allowable stress design (ASD), or service load design (SLD) (*34*);
- Load factor design, also called strength design (SD) (*34*);
- Load and resistance factor rating (LRFR) (*32*);
- Load and resistance factor design (LRFD) (*35*);
- Ultimate limit state (ULS), serviceability limit states (SLS), and fatigue limit state (FLS) (*30, 41*).

Inelastic methods of analysis:

- Alternate load factor design, also called autostress (*40*).

Plastic collapse methods of analysis:

- Yield line theory for deck slabs (*30,35*).

In the United States, the rating methods generally used are WSD or LFD. The LRFD method has not been calibrated for evaluation of live load capacity of existing bridges; it can be used for evaluations if the load factors and performance factors are considered to be the same as those for design or are modified rationally. The inelastic methods have not been used widely, even though the procedures are well established (*40*).

In Canada, evaluations are generally carried out at the ULS. However, as in the United States, concrete bridges that do not show any signs of distress may not need to be posted (*30*).

The load-carrying capacity of existing bridges can first be calculated by using simplified methods of analysis combined with simple live load distribution factors and evaluation procedures. If the live load capacity needs to be increased, more refined methods of analysis with more sophisticated evaluation procedures may be used. Examples of refined methods of analysis are grid analysis, orthotropic plate theory, finite element, finite strip, folded plate, and semi-continuum. Codified examples of more sophisticated evaluation methods are LRFD and ULS.

Most of the evaluation codes only allow linear elastic methods. However, some redistribution of moments over interior spans is allowed even when the analysis is linear. Nonlinear methods are permitted sometimes for the primary load-carrying members (*40*). Plastic system behavior is not generally allowed for primary members but is permitted for decks (*30,35*).

These analysis and evaluation methods can lead to very different live load capacities for the same bridge because of the inherent differences in the methods and the bridge design codes. The more sophisticated methods of analysis would usually lead to higher live load capacities. The effects of material and performance defects and deficiencies are incorporated in each of these methods to arrive at the rating factors (RF) and posting loads.

It is not possible to generalize how much gain in load-carrying capacity can be achieved by the refined methods of analysis and by the different evaluation methods as the gain is very site specific and is a function of the assumptions made in the original design. It may, however, make the difference between having to post or not post the bridge.

Calibration of the OHBDC (*44-46*) and the LRFD code (*47*) provides a comparison of bridges designed by previous AASHTO Standard Highway Specifications for Highway Bridges and by OHBDC and LRFD. A more detailed comparison is given in a study of the impact on the highway infrastructure of existing and alternative vehicle configurations and weight limits (*48*). For this study, reinforced concrete, prestressed concrete, steel, and wooden bridges for spans between 5 m and 40 m at 5-m intervals were designed to H15, H20, HS15, HS20, and OHBDC loads and compared with new truck loads being proposed for Ontario.

The analysis methods and evaluation procedures listed above are also applicable to culverts. However, distribution of the live load through the fill to the culvert must be taken into account. Culverts with more than 0.6 m (2.0 ft) of fill are not usually posted for load restrictions as most of the load is from the fill and not from the live load. Culverts that are showing signs of distress should be investigated for the required remedial measures. Guidelines for assessing soil-steel culverts are provided in two of the references (*49, 50*).

LOAD TESTING

If the engineer believes that the analytical evaluation procedures do not reflect the bridge's actual behavior or some of its components, load tests may be carried out (*11,30*) to ascertain structural behavior, to proof load, and to find out the ultimate load capacity.

The experiences gained from a number of load tests are reported in an ASCE journal (*9*) and NCHRP Report 306 (*10*). Considerable data about a bridge's structural behavior can be gained by instrumenting and load testing individual components or parts of the bridge. However, because of the inherent high safety risks involved, load tests should be carried out under the direction of an engineer with sufficient experience in monitoring the behavior of structures under load test conditions (*11,30*).

The bridge's load-carrying capacity is not directly related to test loads that the bridge is subjected to during controlled test conditions. Therefore, considerable judgment is required in calculating the load-carrying capacity, especially if load test results are to be extrapolated (*30, 11*).

FATIGUE

Bridges are not usually posted for load restrictions because of fatigue-related defects (*30*). Instead, it is preferable to rehabilitate fatigue-related defects (*51*) or accept a reduced life of the bridge based on an assessment of the remaining fatigue life (*38*).

FUTURE LIFE PREDICTION

The future life of a bridge is dependent on a number of interrelated criteria such as:

- Design details, materials, codes, and specifications;
- Fatigue details and defects;
- Construction practices, quality control, quality assurance, and initial defects;
- Protection systems used, e.g., coatings, water proofing, cathodic protection;
- Environmental and traffic exposure;
- The extent and location of material defects, deterioration, and distress;
- The extent of performance defects; and
- Maintenance practices and past history of the bridge.

Because of the nature of the interrelationships of these independent criteria, predicting the future life of a bridge is very site specific. Further details are covered in chapter 6.

DECISION CRITERIA TO INCREASE LOAD-CARRYING CAPACITY

The decision to increase the load-carrying capacity of a bridge is usually based on a careful study of alternative schemes influenced by a number of interrelated factors, such as the structural condition of the bridge, functional needs at the bridge, remaining life of the bridge, life-cycle costs to the agency and users, socioeconomic costs to the community, and the historical aspects of the bridge.

Structural Condition

Structural condition may be established through routine inspections, condition surveys, evaluations or load testing. Based on the structural condition of the bridge, it may be decided that the bridge does not need strengthening, that the bridge can be strengthened to full highway loadings, or that the bridge can be strengthened to partial highway loading and would need to be posted. Various other components, such as fatigue-prone details, may also need rehabilitation at the same time to improve their structural condition.

Functional Needs

The functional needs are the vehicular and pedestrian needs over the bridge; the size of opening related to the road, rail, utilities, or stream under the bridge; the horizontal and vertical clearance and alignments at the bridge. The functional needs may require that the bridge be widened or lengthened and may be addressed at the same time as the bridge is being strengthened.

Remaining Life Of The Bridge

The remaining life of the bridge should be established prior to strengthening. If the remaining life is less than 10 years for severe exposure, or 20 years for mild exposure conditions, it may be more cost effective to replace the bridge at the end of the remaining life rather than strengthen it. In the meantime, it may be necessary to increase the frequency of inspection and routine maintenance and rehabilitation of individual components may be necessary.

Life-Cycle Costing

Life-cycle costing should be carried out for alternative schemes that address the structural and functional needs at the bridge. Life-cycle cost analysis should take into account agency and user costs, including detour costs, and should be based over a period (usually 50 years) that includes at least one bridge replacement cycle. The analysis should be based on the appropriate discount rate and should account for the residual value of the different alternatives (*52, 53*).

Socioeconomic Costs

The socioeconomic costs during strengthening refer to costs to the community that the bridge serves, such as loss of commerce. These costs are not normally considered in the life-cycle cost analysis but may have to be in cases where they directly impact the decision to strengthen.

Historical Aspects

Historical aspects of the structure need to be considered during strengthening, especially if the structure is designated or is likely to be designated a historical structure or a cultural resource. The type of strengthening may be influenced by the need to maintain the structure's historical appearance (*54,55*).

Flow Chart

The flow chart in Figure 1 maps out some of the criteria considered during the decision-making process. The historical aspects, however, are not included as they impact on all the possible decisions and therefore should be considered in addition to the other criteria.

SURVEY OF CURRENT PRACTICES

The United States, FHWA, nine Canadian provinces, Public Works Canada (PWC), and a selected number of consultants in the United States and Canada were surveyed for their current practices for bridge evaluation and load testing. The number of respondents are as follows:

United States	*40 out of 50*
FHWA	*1*
Canadian provinces	*6 out of 9*
PWC	*1*
U.S. consultants	*21*
Canadian consultants	*5*

The following questions were asked.

Question 1—What analytical methods do you normally use to evaluate the live load capacity of bridges?

The answers are tabulated in Table 2. Most of the agencies use more than one analytical method. A number of respondents indicated the computer program or the bridge design code they use instead of the method.

Question 2—Have you used more sophisticated analytical methods in an attempt to raise the predicted live load capacity?

The answers are tabulated in Table 3. Two states mentioned LFD as being more sophisticated than WSD. Most of the agencies and consultants use grid analysis or a finite element method (FEM).

One state mentioned that it would use any method to avoid posting bridges. Two of the consultants used non-linear programs to improve the predicted live load capacity.

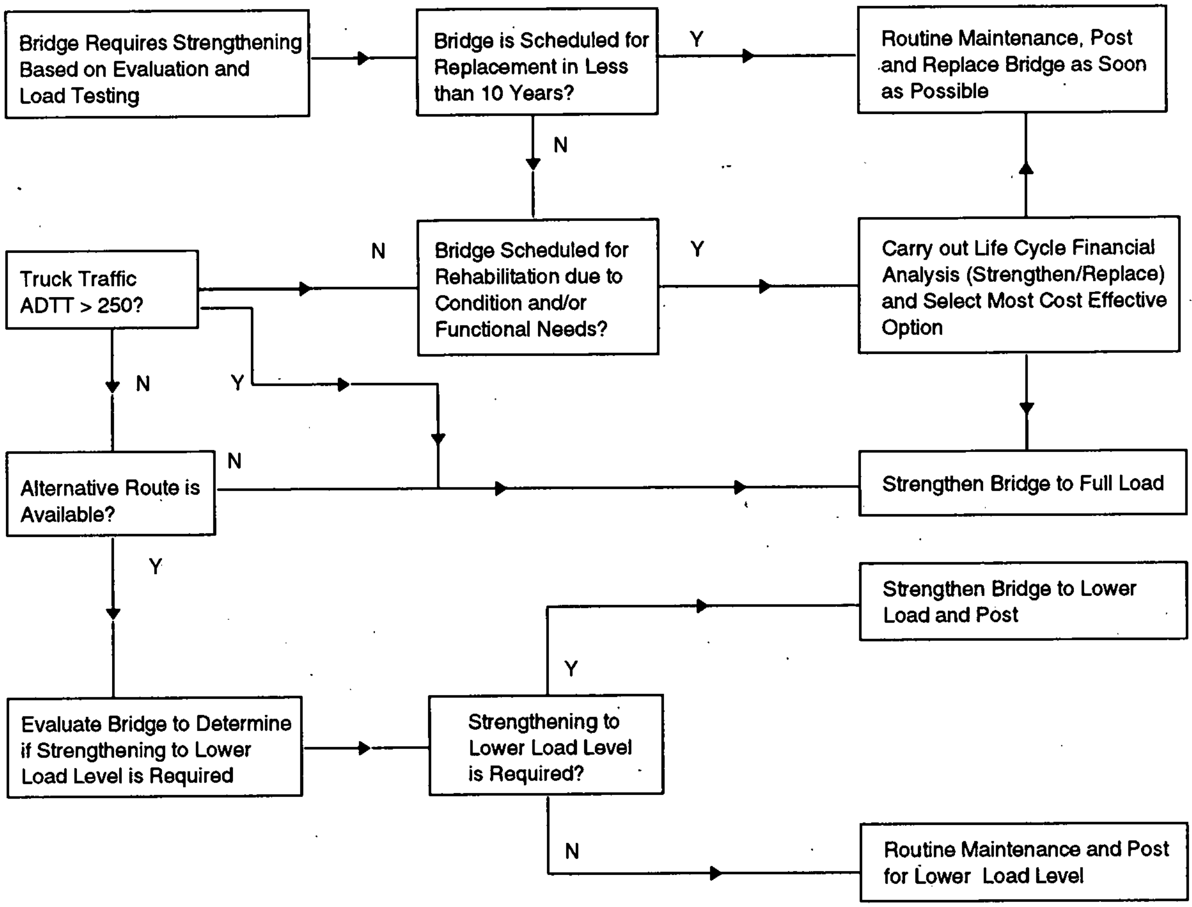


FIGURE 1 Flow chart for bridge strengthening.

TABLE 2
ANALYTICAL METHODS USED TO EVALUATE LIVE LOAD CAPACITY OF BRIDGES

Analytical Method	United States and FHWA	U.S. Consultants	Canadian Provinces and PWC	Canadian Consultants
Linear Method				
WSD	3	–	–	–
LFD	7	2	–	–
LRFD	–	2	–	–
WSD and LFD	8	2	–	–
LFD and LRFD	2	–	–	–
WSD, LFD, LRFD	–	1	–	–
AASHTO	2	1	–	–
ULS, CSA-S6, Supplement 1	–	–	5	–
ULS–OHBDC	–	–	1	2
Computer Program Used	17	11	1	3
Hand Calculations	1	–	–	–
No method indicated	1	2	–	–
Total	41	21	7	5

Question 3—Have you used full-scale load testing, directly or indirectly, to upgrade the live load rating of a bridge?

The answers are tabulated in Table 4. A number of agencies are starting to use load test methods to better understand how bridges behave under load, to proof load the bridges, and to upgrade the live load capacity of bridges.

Appendix B provides more detailed information on responses to these questions.

TABLE 3

MORE SOPHISTICATED ANALYTICAL METHODS USED TO RAISE THE PREDICTED LIVE LOAD CAPACITY

Analytical Method	United States and FHWA	U.S. Consultants	Canadian Provinces and PWC	Canadian Consultants
Linear Method				
LFD	2	–	–	–
LRFD	1	1	–	–
Computer programs used	15	11	4	5
Nonliner Methods				
Plastic	–	–	–	2
Not used	23	9	3	–

TABLE 4

FULL-SCALE LOAD TESTING USED TO UPGRADE THE LIVE LOAD RATING OF A BRIDGE

Load Test Used	United States and FHWA	U.S. Consultants	Canadian Provinces and PWC	Canadian Consultants
Yes	18	12	4	5
No	23	9	3	0

CHAPTER THREE

STANDARD METHODS

BACKGROUND

NCHRP Report 293 (*1*), published in 1987, identified the most widely used methods for increasing live load capacity. These methods were listed in a table in the questionnaire sent to transportation agencies for this synthesis. Agency responses, shown in Table 5, indicate the methods are still used, and no new method, developed since 1986, has any substantial application. These methods can be considered standard methods, and they will be reviewed briefly in this chapter.

In addition to agency and consultant responses, some measure of interest in the various standard methods can be obtained from the literature. The relevant papers have thus been categorized in this chapter by the standard method addressed for increasing capacity. This chapter covers actual physical modification methods for increasing the strength of bridge superstructures.

Chapter 4 covers methods by superstructure material, and chapter 7 covers new technologies, innovative methods, and ongoing research.

AGENCY RESPONSES

In the questionnaire for this synthesis, agencies were asked to identify which methods they used to increase live load capacity. Agency responses are provided in Table 1, and the total number of agencies using each of the standard methods, shown in the last column, gives a good measure of relative popularity of these methods. In addition, the agencies were asked for the number of bridges modified by each method, and the responses are provided in Table 5. The data are of questionable value, however, because, in many cases, data were not available or were difficult to compile because some agencies gave a "yes" or "no" answer rather than a number; for this synthesis, a "yes" counted as one bridge but in actuality, could have been many. A high number of bridges, however, does not necessarily represent widespread use of a method. For instance, the most popular method would appear to be "develop composite action," with a total of 658 bridges in the United States and Canada. However, 500 of the reported bridges are from Ohio, where decks were automatically made composite under a deck replacement program. This not only distorts the usefulness of the figure, but brings into question whether these decks have all been made composite to increase live load capacity or whether this condition is a by-product of a rehabilitation program.

As indicated in Table 5, North American agencies have reported the strengthening of 1,268 bridges for live load capacity since 1986. Several anomalies in the breakdown of methods used should be pointed out. Of the 264 bridges in the Improve Member Strength category, 73 are from one province, Alberta. Of the 143 bridges in the Post-tension Members category, 70 represent the post-tensioning of wood truss bridges in one province, New Brunswick. Of the 39 bridges in the Other Methods category, 30 are from one state, Oregon, and represent the adding of stringers or bents to deficient wood bridges.

Thirty-six agencies indicated that they had modified a bridge superstructure specifically to increase live load capacity since 1986, and 12 answered in the negative. However, only 29 of the agencies gave any numbers. For this, and other reasons given, it should not be assumed that the individual numbers, or the 1,268 bridge total, are reliable. The figures may be of some interest for comparative purposes, but the tabulated numbers do not accurately reflect the real numbers of bridges modified for live load by standard methods since 1986.

LITERATURE SEARCH

A review of literature for this synthesis identified 210 papers published since 1986 that relate to strengthening methods. Table 6 lists the number of papers by standard methods. Seventy-three of the papers deal with actual strengthening

TABLE 5

NUMBER OF NORTH AMERICAN BRIDGES STRENGTHENED FOR LIVE LOAD CAPACITY BETWEEN 1986 AND 1996, AS REPORTED BY AGENCIES

Methods for Increasing Live Load Capacity	United States	Canada	Total Number
Reduce Dead Load	89	19	108
Develop Composite Action	638	20	658
Increase Transverse Stiffness	7	5	12
Improve Member Strength	164	100	264
Post-Tension Members	53	90	143
Develop Continuity	43	1	44
Other Methods	37	2	39
Total Strengthened	1,031	237	1,268

carried out on bridges in service, while the others cover laboratory testing, theory, and research.

TABLE 6

NUMBER OF PAPERS PUBLISHED BETWEEN 1986 AND 1996 RELATED TO STRENGTHENING BRIDGES FOR LIVE LOAD CAPACITY, BY METHOD

Methods for Increasing Live Load Capacity	Total Number
Reduce Dead Load	9
Develop Composite Action	6
Increase Transverse Stiffness	6
Improve Member Strength	122
Post-Tension Members	43
Develop Continuity	6
Other Methods	18

By far the greatest interest for publication lies in two methods, improving member strength and post-tensioning of members, comprising 78 percent of the total. A significant portion of the Improve Member Strength category is accounted for by the increasing research and development effort on the use of ACM, particularly CFRP, to strengthen members. This is not yet a standard method, and is thus covered in chapter 7, Emerging Technologies. Of the 43 papers on post-tensioning, almost half refer to actual installations. International interest in this method is widespread with 17 papers from the United States, five from Canada, five from the United Kingdom, and 16 from other countries, including Switzerland, France, and Germany. In the Other Methods group, 15 of the 18 papers cover the addition of members to increase capacity.

The single digit numbers in Table 6 for each of the four categories might appear surprising, as three of the categories are shown to be commonly used methods (see Table 5). The explanation is probably that the methods have been used so much that they are no longer thought worth reporting on for project papers, and that they are not providing enough future promise to justify significant research and development effort.

The numbers in Table 6 do not reflect the relative popularity of the standard methods in practice. They do indicate, however, that improving member strength and post-tensioning members have the most promise for further development using new techniques.

METHODS

Chapter 2 of NCHRP Report 293 (*1*) gives a useful historic overview of the development of each of the standard methods considered herein. More detailed information on each method, including a description of use and limitations, plus information on basic cost is given in chapter 3 of that same report. This synthesis chapter reports on the use of standard methods during the past 10 years, based on information from agencies, consultants, and the literature.

Reduce Dead Load

Reducing dead load was the third most popular method reported by North American agencies (Table 1). It was similarly ranked when considering the number of bridges strengthened in the United States and fourth for the North American totals (Table 5).

The most effective way of reducing dead load to increase live load capacity is to replace a heavy deck for a slab on a girder bridge with a lighter one. This is only cost effective, however, if the deck needs replacing for other reasons. Frequently, the need to replace a concrete deck because of deterioration and delamination occurs at much the same time as the need to increase live load capacity, around 40 years or less. In these cases, deck replacement and live load upgrade can be carried out as part of a major rehabilitation program. If the beams were noncomposite, they could be made composite at the same time to further increase capacity.

Many options exist for providing a lighter deck. If there is a substantial thickness of asphalt, the removal of the asphalt and replacement with a thin wearing surface can raise the capacity, but this method is not likely to be very effective for short spans. Both New Brunswick and Public Works Canada (PWC) reported this method where deck protection was needed. The removal of asphalt only was reported by Arkansas and New Jersey. Replacing the full deck slab is the usual standard method, and the various replacement systems are now each considered.

Lightweight Concrete Decks

New Mexico, New York, and North Carolina reported using lightweight concrete decks, as did three consultants. Decks may be cast-in-place or precast. The former is usually lower in construction cost, but precast decks may be necessary to minimize lane-closure times if the bridge must be kept open to traffic. For precast panel construction, particular attention should be paid to joint details, prestressing, seating, and connection to girders. These issues are covered in two 1995 reports (*56,57*) in the *PCI Journal*. Selecting the lightweight aggregate and the mix design should be done with care to ensure durability (*58–60*).

The Massachusetts Highway Department provided data on the deck replacement in 1992 for a two-span steel stringer bridge. A required increase in live load capacity from H20-44 to HS20-44 was achieved by combining lightweight concrete deck, composite action, and bottom cover plates. The controlling moment capacity was increased by 22 percent. Lane closures were required, and traffic protection represented 17 percent of the contract cost. The wearing surface weight was minimized by use of a thin latex-modified concrete overlay.

Steel Grid Decks

Steel grids have been used for many years to provide lightweight decks, generally on steel stringers. No agencies reported on their use to increase live load capacity, but one consultant noted such an application. Grids can be left open, or filled with concrete with or without overfill.

In addition to their light weight, grids have the advantage of composite action if welded to the stringers, but these welds tend to be fatigue prone. Disadvantages with the open grids

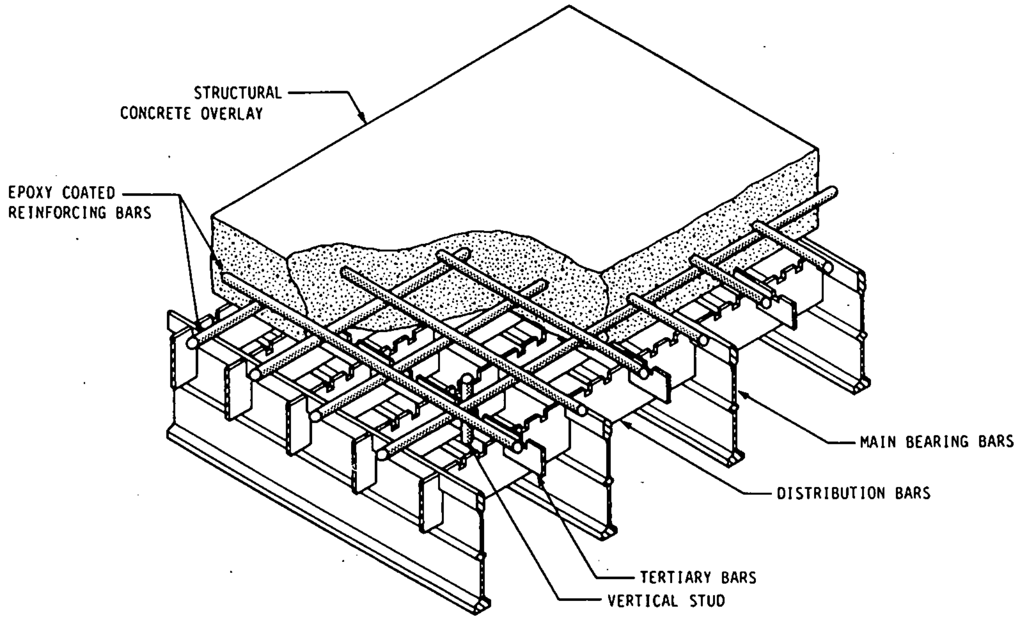


FIGURE 2 Exodermic deck system *(1)*.

are the need to protect the structure beneath the deck, poor skid resistance of the exposed steel, and high noise level. These problems can be overcome by concrete filling, but with a weight penalty. Grids concreted to the level of the top of the steel have had problems because of corroding of the vertical bars, causing longitudinal expansion and breaking of welds to the stringers. The problem can be alleviated by concreting over the top of the grid, which also improves skid frictional resistance but generally at further weight increase.

The Exodermic deck is a proprietary system that combines a thin, 75-mm (3-in.) minimum, concrete deck on top of a steel grid (Figure 2). The concrete can be precast to provide prefabricated modules for fast construction, or cast-in-place. First used in 1984 in New Jersey, the deck was identified by New York for recent use. In a 1992 application in St. Johnsville, N.Y., a reinforced concrete deck was replaced by the Exodermic deck, enabling the load rating to be upgraded to HS-20 *(61)*. The 1,730 m^2 (18,600 ft^2) deck, using cast-in-place concrete, had an in-place cost of \$365 per m^2 (\$34 per ft^2) including steel grid, rebar, concrete, and labor.

Orthotropic Decks

Steel orthotropic decks have been used primarily on new long-span steel bridges, such as suspension bridges, where reduced dead weight is important for economy (Figure 3). The deck consists of a wearing surface on a steel plate stiffened by welded longitudinal ribs, spanning between integral floorbeams. The ribs can be open or closed, and the closed trapezoidal rib has predominated in North America. Although expensive to construct, this deck type is attractive on long-span bridges because it reduces dead weight and acts integrally with the main structural system to increase the bridge's capacity (*62*). In Japan, where reinforced concrete decks were replaced, this method was cost-effective on some relatively

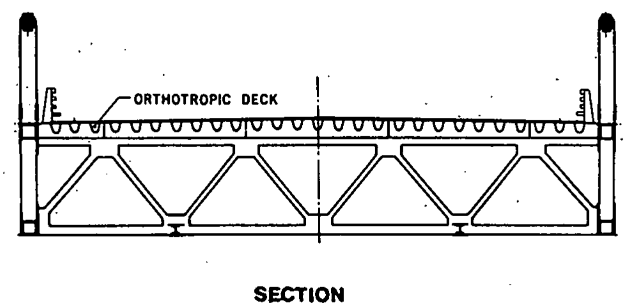


FIGURE 3 Orthotropic deck on a suspension bridge.

short spans, such as arch spans of 54 m (177 ft) and 74 m (243 ft) (*63,64*). In Canada, reinforced concrete decks on the main cantilever spans and four-deck truss approach spans for the Champlain Bridge over the St. Lawrence River were replaced by orthotropic decks in 1993 (*62*). The shorter deck truss spans are 77 m (252 ft), and the new decks act integrally with the truss top chords. Orthotropic deck panels can be paved prior to erection to minimize lane closure time. At deck joints, steel bulkheads can be added to contain the paving.

A number of orthotropic decks have suffered from failure of the wearing surface. It is essential that the bond between the deck plate and the wearing surface is maintained; this requires careful consideration of the specific environmental and loading conditions for each site before selecting the most appropriate system. In particular, interaction forces at the interface of the wearing surface and the underlying plate must be characterized throughout the entire expected range of temperature. A paper on six orthotropic redecking projects in North American (*62*) covers wearing surfaces and costs.

Develop Composite Action

Developing composite action was the second most popular method of increasing live load capacity among North American agencies (Table 1), but when considering the number of bridges strengthened (Table 5), it was the leading method.

In 1988 a review of full-scale load tests to assess the degree of unintended composite action in beam and slab highway bridges was reported (*10*). It was limited to bridges with reinforced concrete decks on steel beams with no mechanical horizontal shear connection between the slab and beams. The tests showed significant composite action from the natural bond at this interface, but because the values varied, they were not considered reliable enough to evaluate live load capacity, particularly at ultimate load levels. It would be possible to increase the capacity by breaking out some pockets in the slab over the beam, welding stud shear connectors in groups, and grouting the pockets. This method was not reported by any agencies, and the composite action is usually fully developed only when the slab is due for replacement from deterioration. At that time, the simple welding of studs provides a relatively inexpensive way of obtaining a substantial increase in capacity.

Although most cases reported for this synthesis were for steel beams, the method has also been applied to deck trusses. Both cast-in-place and precast decks have been used. The most common technique for precast decks is placing studs in groups to match block-outs in the precast slab, later grouted.

Developing composite action for concrete beams is a slow and more costly operation, generally requiring the drilling and grouting of dowels between the slab and beam. The number of agency applications reported was small, with only 14 for concrete beams compared with 644 for steel (Table 7).

Alaska DOT reported a major upgrading project to bring two bridges, designed to HS20, up to new HS25 loading to match new adjacent parallel structures. The construction took place in 1992 for $22 million on 14 spans between 48 m (156 ft) and 62 m (202 ft) and featured welding cover plate to beam flanges, welding shear studs, and adding a new composite concrete deck. If field welding is to be used in strengthening members, fatigue performance will need to be evaluated.

Increase Transverse Stiffness

For beam and slab bridges, increasing the transverse stiffness improves the load distribution to the beams and enables a higher live load to be applied. It is a relatively expensive way of increasing capacity, however, and the increase may be small (*65*), which may account for the few applications reported. Of the 1,268 bridges reported, only 12 used this method (Table 7), and only six agencies employed it (Table 1).

Although few cases of transverse strengthening were identified, it is common to seek an improvement in load distribution by analytical means. In reply to question two, 20 agencies reported using more sophisticated analytical methods to raise the predicted live load capacity. Such methods will frequently give improved load distribution with no change in transverse stiffness compared with the simplified distribution factor approach. If transverse strengthening is selected, by adding cross frames or diaphragms for instance, then a sophisticated analysis will be required to evaluate the modification's full benefit.

Increased transverse stiffness can be obtained by thickening the deck slab. Michigan DOT reported on casting a concrete slab over an existing slab on a concrete T-beam bridge to upgrade from the H15 design capacity. After testing the strengthened bridge, a 32 percent increase in live load was permitted. This could be an economical method for relatively short spans, as no special deck forms are needed. The Michigan project had spans of 12.5 m (42 ft.) or less, but longer spans would result in an increasing dead load penalty.

In Minnesota, a retrofit scheme to strengthen longitudinal nail-laminated timber decks was evaluated (*66*). The scheme consisted of laying a second transverse layer of timbers above the existing deck and casting a grout layer between the two. A number of longitudinal nail-laminated decks have been strengthened by transversely post-tensioning the laminates (*67*). This restores the full transverse stiffness of the wood deck, providing better load distribution and increased live load capacity. In Ontario, a second cycle of stressing is performed to compensate for creep and relaxation effects.

Improve Member Strength

Adding material to improve the strength of a deficient member is the most obvious way of increasing live load capacity. It is the method used by more agencies than any other (Table 1), and is second only to composite action in the number of

TABLE 7

BRIDGES STRENGTHENED FOR LIVE LOAD CAPACITY BY NORTH AMERICAN AGENCIES BETWEEN 1986 AND 1996, BY METHOD AND MATERIAL

Methods for Increasing Live Load Capacity	Superstructure Material				
	Steel	Reinforced Concrete	Pre-tensioned Concrete	Post-tensioned Concrete	Wood
Reduce Dead Load	34	49	3	–	22
Develop Composite Action	644	9	5	–	–
Increase Transverse Stiffness	6	5	–	1	–
Improve Member Strength	216	16	28	–	4
Post-Tension Members	57	11	1	1	73
Develop Continuity	40	2	2	–	–
Other Methods	3	4	1	1	30
Total All Methods	1,000	96	40	3	129

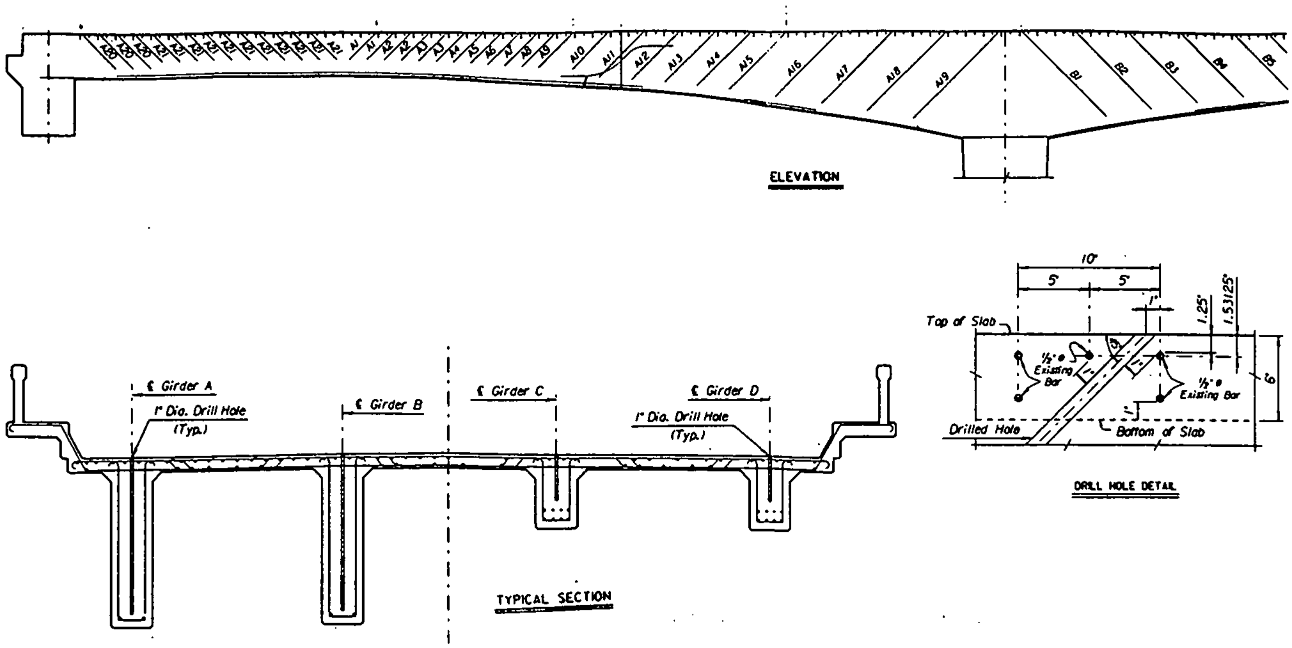


FIGURE 4 Rebar insertion to increase shear capacity (Kansas DOT).

actual applications reported (Table 5). Although the method is considered standard, there are many ways of carrying out strengthening, as illustrated by the high level of interest shown by the number of published papers (Table 6). The 122 papers on improving member strength is nearly three times the number in any other category.

The method can be relatively simple if additional plates or sections are attached to a member, whether for flexural or axial load deficiencies. If the dead load stress in a member is high, however, a large increase in section may be required to limit the dead plus live load stress in the parent material. At the same time, the stress in the added material, due to live load only, could be very low. A more effective method, requiring less strengthening material and giving more even stress distribution, is to reduce the dead load stress before strengthening. This can be done by jacking and supporting beams, or by using temporary members loaded to relieve the stress in axially loaded truss members, before carrying out the strengthening. For truss members, it is important to check the capacity of the connections if a member is to sustain an increased load.

Member strengthening can be effective in all bridge materials, and the standard techniques are presented in NCHRP Report 293 (*1*). These are covered in the following categories, and the references noted provide information on recent uses of the techniques.

- Addition of steel cover plates:
  - Steel stringer bridges (*68,69*);
  - Reinforced concrete bridges (*70*);
  - Timber stringer bridges;
  - Compression members in steel truss bridges;
  - Strengthening tension members of truss bridges (*71*).
- Shear reinforcement:
  - External shear reinforcement for concrete, steel, and timber beams (*72,73*);
  - Epoxy injection and rebar insertion.
- Jacketing of timber or concrete piles and pier columns (*74,75*).

Kansas' rebar insertion method to increase shear capacity in concrete beams is an interesting example of the development of an experimental method into a standard one. Kansas DOT reported that about 80 bridges have been strengthened this way, where bridges designed to HS20 loading have had shear capacity increased by up to 75 percent. Most of the work was done in the early 1980s, but Kansas still strengthens about three bridges a year this way. The technique as used in 1995 is illustrated in Figure 4. The cost of drilling the inclined holes, placing the rebars, and filling with epoxy is \$62 to \$75/m (\$19 to \$23/ft).

Post-Tension Members

For more than 30 years, post-tensioning of members to increase their capacity has been used for steel, concrete, and wood bridges. If a member is overstressed in tension, a concentric post-tension force will reduce the tension. If a member is overstressed in flexure, local post-tensioning or full-length longitudinal eccentric post-tensioning can counteract the condition. The bending effect can be magnified by having the tendons below the soffit in a king-post arrangement. Figure 5 shows these tendon arrangements for beams, but they are also applicable in a similar fashion to trusses. Shear deficiencies can be corrected by external vertical post-tensioning usually with rods rather than strands. Transverse post-tensioning can be applied to side-by-side box girders or laminated wood decks (*67*) to improve load distribution. The details of various techniques, and tables showing some 42 application examples before 1987, make NCHRP Report 293 (*1*) an important source for standard methods still in use.

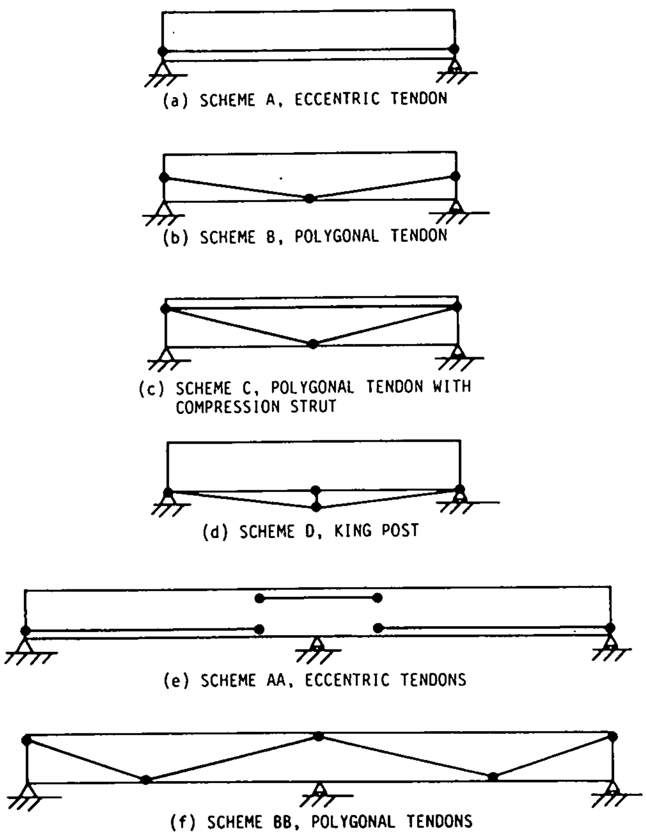


FIGURE 5 Tendon configurations for flexural post-tensioning of beams *(1)*.

The flexibility and wide potential for application ensures the popularity of the post-tensioning method, with 19 agencies reporting on its use since 1986. It is interesting to note that although post-tensioning is traditionally associated with concrete, 12 agencies have applied it to steel members, compared to only four agencies with concrete applications. *Prestressed Steel Bridges* (*76*), published in 1990, covers all aspects of theory and design, including detailing of tendons, anchorage, and corrosion protection, and has one chapter devoted to rehabilitation and strengthening. Post-tensioning to increase strength of reinforced concrete or prestressed concrete bridges will normally involve external tendons. This topic is covered in detail in an ACI publication, *External Prestressing of Bridges* (*77*), and by work at the University of Texas on segmented box girders (*78*). Post-tensioning is more complicated than the other methods considered for a number of reasons. From a design standpoint, the effect of post-tensioning on all parts of the structure must be considered, not just the element of direct concern. There may be secondary effects to consider, and the high local stresses at anchorages can be a problem. Post-tensioning can overcome an allowable stress problem at the serviceability level, but the ultimate strength condition needs checking and may not produce equal benefit. In the field, the construction requires care, particularly the stressing operation itself. Advantages of the method are considerable, however. By using high-strength wire or rod, the weight of added material is minimized and used efficiently. The operation can usually be carried out with little traffic disruption.

The 143 bridges strengthened by post-tensioning (Table 5) places the method in third place for number of applications, with the largest one coming from post-tensioning the bottom chords of wood trusses. This has become a standard method in New Brunswick, being applied to 70 bridges. Connecticut DOT supplied drawings for the 1987 post-tensioning of a precast concrete beam bridge, and some details are reproduced in Figure 6. In Indiana, a similar prestressing system has been protected against corrosion by enclosing in a polyethylene pipe. Information from Colorado DOT emphasizes the importance of design and detailing of the anchorage, particularly with steel beams. In the post-tensioning of steel wide flange girders in 1991 the anchorage to the bottom flange had to be modified during construction following the occurrence of local flange buckling.

Although post-tensioning is not new, the number of new and interesting applications is reflected in the many papers published on the subject (43 papers, Table 6), second only to improving member strength. Because prestressed concrete started in Europe well before its introduction to North America in the late 1950s, many European bridges have undergone strengthening by post-tensioning recently, and much of the literature on this method comes from that continent. Some review papers of general interest come from France (*79*), Austria (*80*), Germany (*81*), Italy (*82*), and Switzerland (*83*). Papers dealing with particular projects and nonstandard applications will be considered in chapter 4.

Develop Continuity

Changing simple spans into continuous spans reduces the positive moment and is an established method of increasing live load capacity. The agencies did not indicate frequent use of the method, however, with 10 agencies reporting 44 applications (Table 5), of which 22 were in Kansas, 10 in Ohio, and six in Colorado. Details of the means of providing continuity were provided only by Kansas, where all 22 bridges used girder encasement, 20 for steel girders and two for reinforced concrete beams.

A widely used method for new precast beam and slab type construction, is to make the simple beam spans continuous for live load by casting an encasing diaphragm at the piers, integrally with the deck, and reinforcing the deck for negative moments (*84*). The same method can be used for existing simple-span concrete structures, but would only be cost-effective if a new deck were required. In fact, only four concrete bridges were strengthened by developing continuity (Table 7). The method can be expensive for concrete bridges on which the deck slab and beams act compositely by the use of stirrups connecting the two elements, as slab removal while preserving the stirrup steel is difficult.

The same method of developing continuity can be used for steel girder bridges, with shear studs welded to the girders to engage the new diaphragm concrete. If the steel girders were originally noncomposite, the slab removal is relatively simple, and the combination of providing continuity and composite action could be cost-effective.

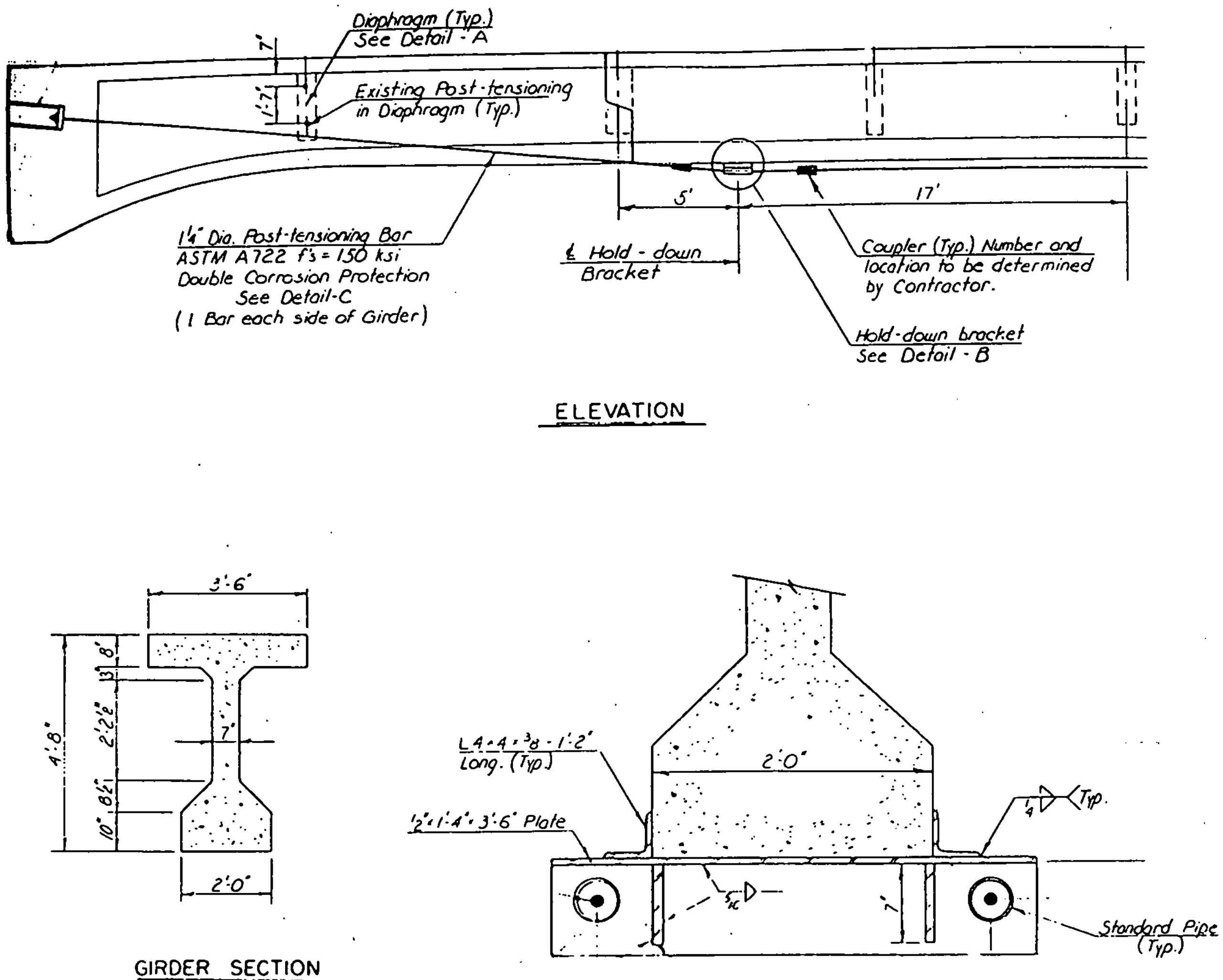


FIGURE 6 Post-tensioning of precast concrete beam (Connecticut DOT).

If the slab does not need replacing, noncomposite steel beams may be made continuous by splicing the webs and flanges at the piers. This can be done by welding without disrupting traffic, but is not very common and can present fatigue problems with field welding. If the top flange splice requires bolting, the deck slab has to be broken out locally, requiring temporary lane closures.

Continuity can be most easily developed on timber stringer bridges by bolting steel side plates to stringers at the piers, but no instance of this was reported. The literature search did not reveal any recent papers of significant interest on developing continuity.

Other Methods

Of the other methods reported, adding members, either beams or piers, to increase capacity could be considered standard. Thirty-nine applications were reported, with 30 of those from Colorado, all in timber structures (Table 7). Roughly half of the Colorado bridges were strengthened by doubling the longitudinal stringers, the other half by adding intermediate pier bents. In each case, the structures were designed for H-15 loading and needed to be brought up to current legal or annual permit loads.

For short-span timber bridges this is an effective method, as the members are not heavy to handle and timber lends itself to field modifications quite readily. In 1994-1995 in Colorado, adding a timber bent cost $10,000 and adding a timber stringer $1,000 or $9,000 per span for a two-lane bridge with spans around 6 m (20 ft).

Saskatchewan reported on a three-span flat slab-reinforced concrete overpass, with 10-m (33-ft) spans, that was deficient in flexure in the end spans. The addition of new lines of support near the points of maximum positive moment in the end spans raised the live load capacity.

The addition of a pier to raise the capacity of a larger bridge took place in 1995 on the Langleybury Lane Bridge in the United Kingdom (*85*). The overall length of the original three-span bridge (Figure 7(a)) was 72 m (236 ft), and construction was in reinforced concrete. Shear cracks were observed in the main span spine beams, and analysis showed a

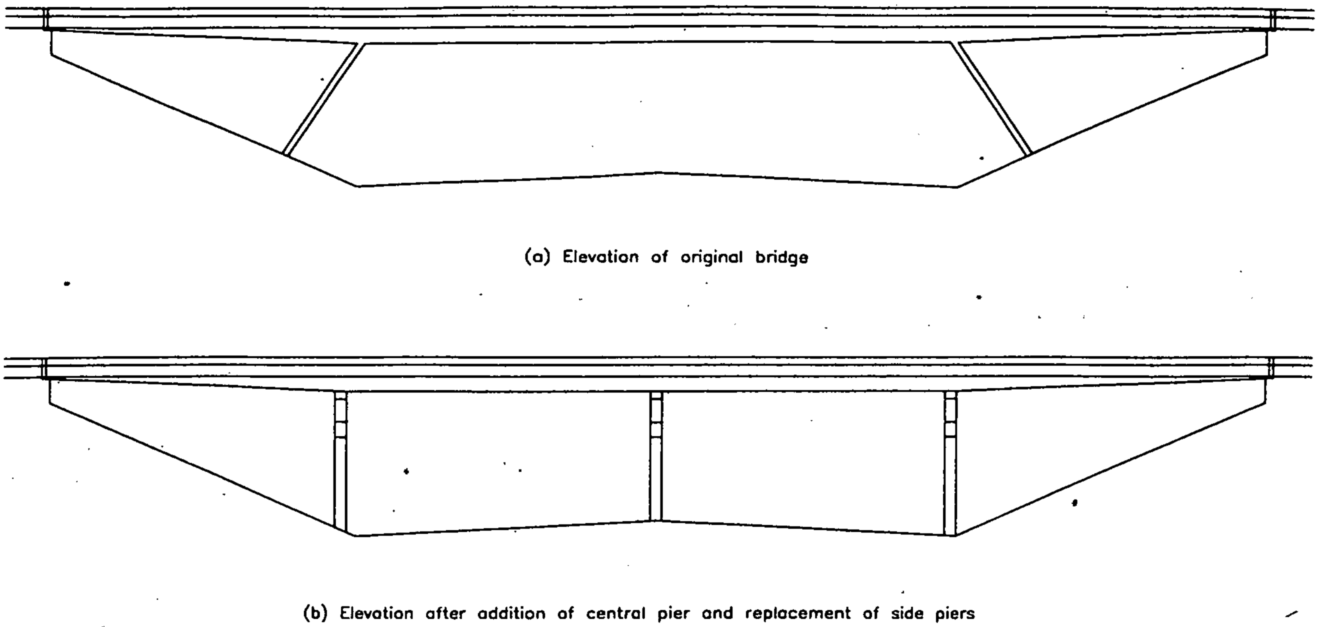


FIGURE 7 Change from 3-spans to 4-spans, Langleybury Lane Bridge *(85)*.

serious capacity deficiency. The change from a three-span to a four-span structure (Figure 7(b)), was carried out keeping one lane open to traffic, and enabled the shear-deficient bridge to be upgraded to full current code requirements. The contract cost was roughly half the estimated cost of a replacement structure.

CHAPTER FOUR

SUPERSTRUCTURE CASE STUDIES

This chapter presents case studies of methods to increase live load capacities of bridge superstructures since 1986. They include interesting or unusual applications of standard methods, as well as innovative methods, and are categorized by the main superstructure materials: steel, reinforced concrete, prestressed concrete, and wood. The cases cover practical and tested techniques, and do not include methods still under development, such as the use of advanced composite materials (ACM) for superstructures, which are covered in chapter 7. The projects have been selected from a review of the agency responses, consultant replies, and the literature search, and include a number of overseas applications that could be of interest to North American agencies. References are given for several relevant papers not selected for detailed review.

STEEL

Interesting applications of standard methods for steel bridges will be presented in the same order as used in chapter 3. Two new categories have been added to cover combined methods and other methods.

Reduce Dead Load

Wolchuk (*62*) reported on redecking of the following major North American bridges, using steel orthotropic decks to reduce the dead load:

- Lions' Gate Bridge, Vancouver;
- George Washington Bridge, New York;
- Golden Gate Bridge, San Francisco;
- Throgs Neck Bridge, New York;
- Ben Franklin Bridge, Philadelphia; and
- Champlain Bridge, Montreal.

Although the main spans of these bridges are all in the long-span range, some shorter approach spans were also redecked with steel orthotropic decks. The Lions' Gate Bridge approach spans were 13 m (42.6 ft) to 38 m (125 ft), the Throgs Neck viaducts 42 m (138 ft) to 58 m (190 ft), and the Champlain Bridge deck trusses 77 m (252 ft).

Another interesting example of this method comes from Japan (*64*) on the Showa Bridge, a stiffened steel arch with a span of 73 m (240 ft). The bridge was opened in 1959, and after 30 years, the reinforced concrete deck slab required replacement. At the same time, sidewalks needed to be added to the bridge, and the live load needed upgrading from Class 2 to Class 1, a 40 percent increase. The extra width increased the dead weight by 15 percent, but the weight was minimized by using a steel orthotropic deck composite with the stiffening girder (Figure 8). The steel deck also best suited the need to keep one lane open to traffic at all times and thus shortened the construction period. The increased dead and live load caused the overstressing of some members and was corrected by adding diagonals. This change to the structural system reduced the bending moments in the arch rib and stiffening girder satisfactorily, but increased the load in some verticals, which needed reinforcing with cover plates. The structure was load tested before and after reinforcement and the results showed a 50 percent reduction in deflections for the rehabilitated bridge, which was reopened in 1992.

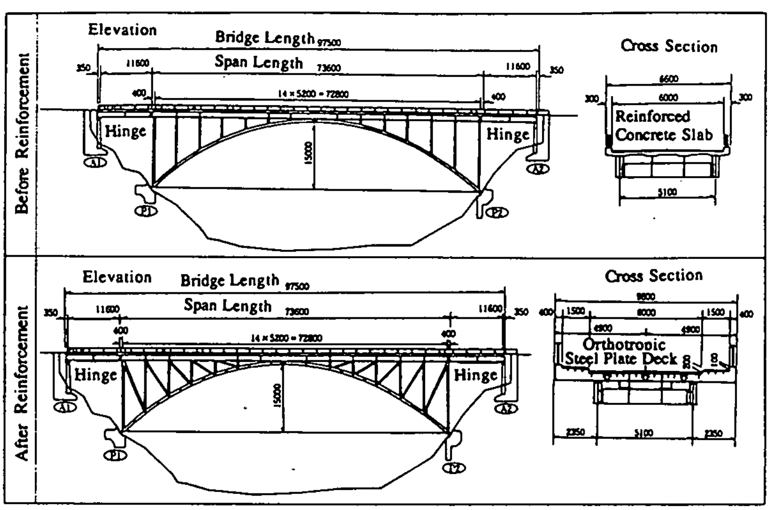


FIGURE 8 General view of the Showa Bridge, Japan (*64*).

Orthotropic decks can be made even lighter by fabricating with aluminum rather than steel. The aluminum industry contends that aluminum may be a viable economic alternative for lightweight decks, particularly if life-cycle costs are considered (*86*). Ten aluminum bridges have been built in North America and are expected to give satisfactory service for up to 50 years. Although aluminum decks are relatively new to this continent, about 40 have been built in the Nordic countries in the past decade. Following a recent testing and development program, orthotropic aluminum decks were used on two bridges in the United States in 1996. A demonstration project for a replacement deck on the Corbin suspension bridge in Pennsylvania, with a span of 91 m (300 ft), has tripled truck live load capacity. A second system (Figure 9) was used on the Route 58, Little Buffalo Creek Bridge in Virginia, weighing about 98 kg per m^2 (20 lbs. per ft^2).

Improve Member Strength

The Corpus Christi Harbor Bridge (Figure 10) was completed in 1959, and was the last major riveted structure built in Texas (58). The original lightweight concrete deck had deteriorated and the use of a replacement deck of normal weight

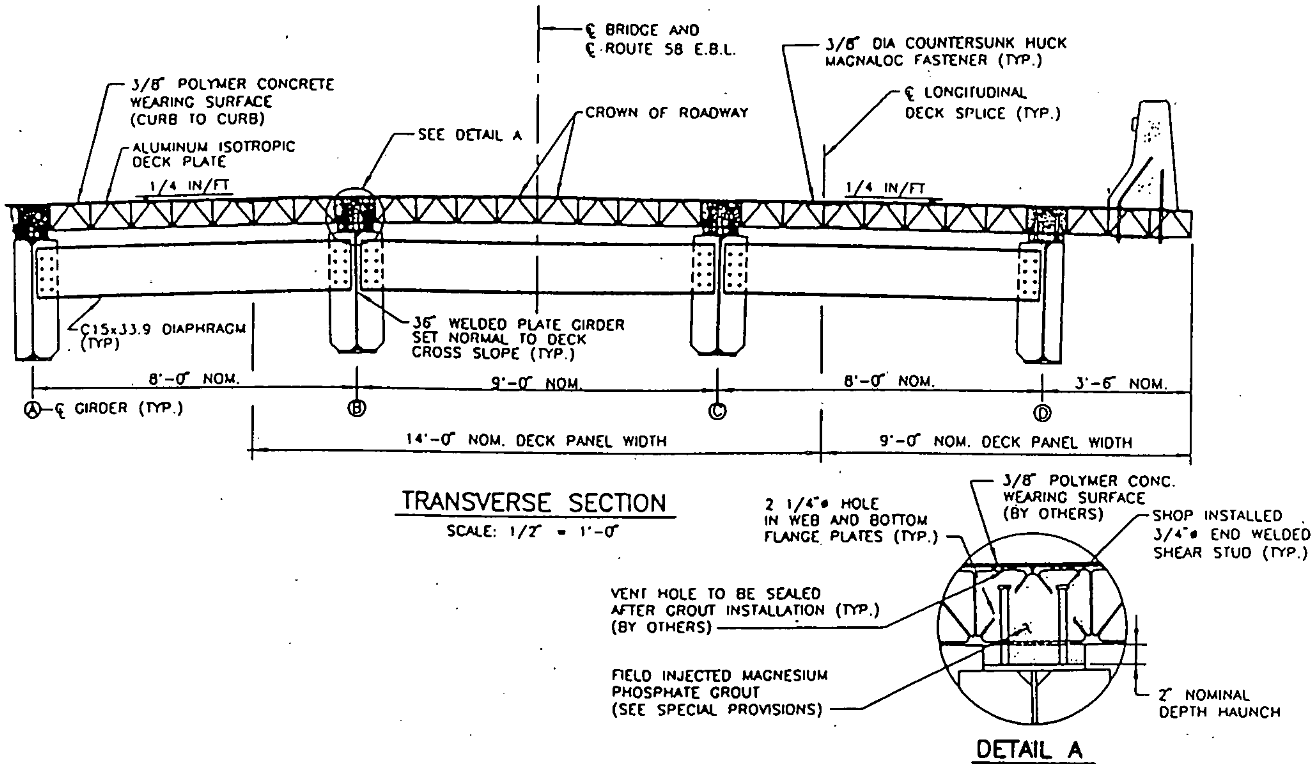


FIGURE 9 Aluminum deck, Little Buffalo Creek Bridge (Modjeski and Masters).

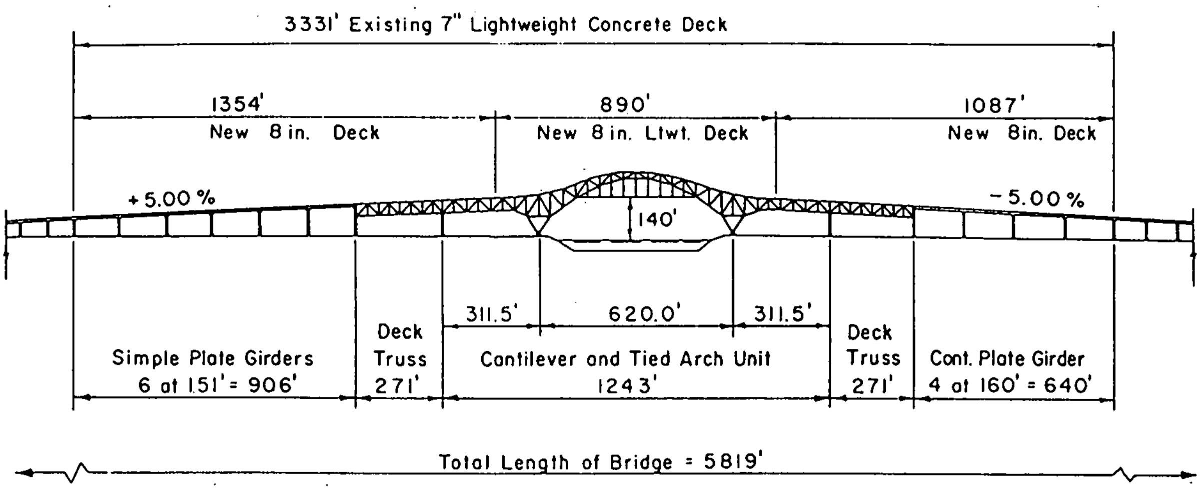


FIGURE 10 Corpus Christi Harbor Bridge *(58)*.

concrete was investigated. However, as this would have caused an overstress in many main span members, a thicker lightweight deck was selected for the central portion instead, with normal weight concrete elsewhere. The selection of normal weight concrete for the anchor arms reduced the number of compression members requiring strengthening. The original design was by working stress methods to the 1953 AASHO specification, whereas the rehabilitation design used load factor methods to the 1983 specifications. The two specifications employed different compression member design methods, and frame analysis also indicated some long compression members had K factors well above the recommended AASHTO value of 0.75. The combination showed serious overstress to the main V-struts (Figure 11), which was corrected by adding a system of triangular bracing in both planes. Some of the cantilever truss compression members, with lengths up to 24 m (78 ft), were strengthened by bolting side plates to the box-shaped members. Twenty-four approach truss compression members, 10 m (34 ft) long, were strengthened by bracing to the adjacent roadway stringers.

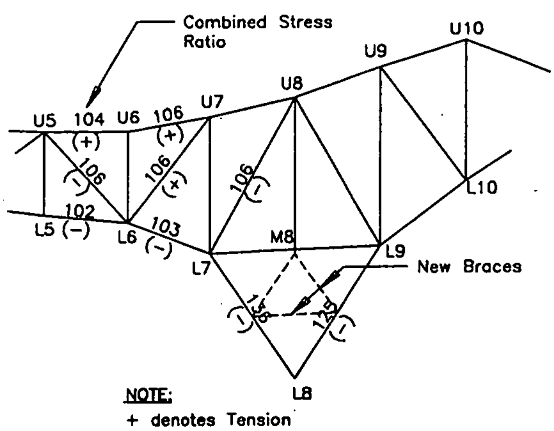


FIGURE 11 Added bracing at main pier *(58)*.

The Friarton Bridge is a major steel/concrete composite box girder bridge over the River Tay in Scotland. Built in 1978, the bridge was assessed for current BS5400 design standards and live loading in 1991 (*87*). The new live loading gave lane loadings, which vary with loaded length, as much as 90 percent above the original design values. The most serious overstress areas were at the main piers, where the tension flange required modification over a 41-m (135-ft) length and the compression flange over a 85-m (279-ft) length (Figure 12). The bridge was strengthened using a combination of post-tensioning, additional stiffeners and bracing. As it was not practical to prop the deck before strengthening, and as the dead loads were dominant, the addition of steel to the top flange would be an inefficient approach. However, as the overall buckling strength of the compression flange was also increased, it was more efficient to stiffen the bottom flange (Figure 13). The top flange was post-tensioned over the required length using a multi-strand system. The ring beams needed to distribute the loads at the tendon anchorage areas required additional bracing, also shown in Figure 13.

The South Muskoka River Bridge in Ontario was built in 1952, and by 1988 required deck replacement. The two-span deck trusses, with spans of 20.5 m (67 ft) and 30.8 m (101 ft), were evaluated for current loading and were found to be deficient (*88*). Most truss members in the two-truss system were overstressed, as were all of the floorbeams. Alternative approaches considered were replacing the bridge, strengthening members, post-tensioning members, and adding a third truss in the center. The third truss option was selected as it halved the floorbeam spans, which then did not require strengthening. A decision had also been made to detour the traffic off the bridge as lane closures were not acceptable. The bracing between the two original trusses had to be removed, and new bracing installed for the three-truss system. The preliminary estimates indicated this was the most cost-effective alternative, but final costs were higher than estimated. Replacing the superstructure may have been a better choice, if the value of reduced construction and detour time were built into the estimated cost.

The overseas strengthening project that has received the most publicity in North America is the Severn Crossing in the United Kingdom (89). The crossing opened in 1966 and comprises a four-lane motorway on a major suspension bridge, a cable-stayed bridge, and several approach viaduct structures. The suspension and cable-stayed structures featured revolutionary shallow box sections, aerodynamically shaped, consisting

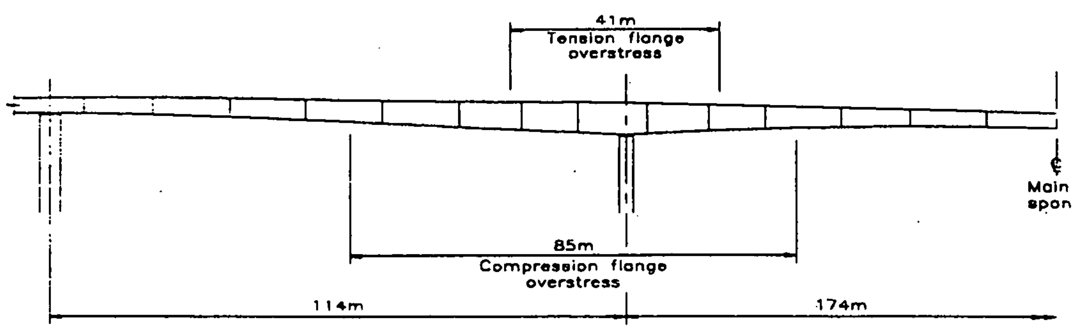


FIGURE 12 Friarton Bridge, showing area of overstress *(87)*.

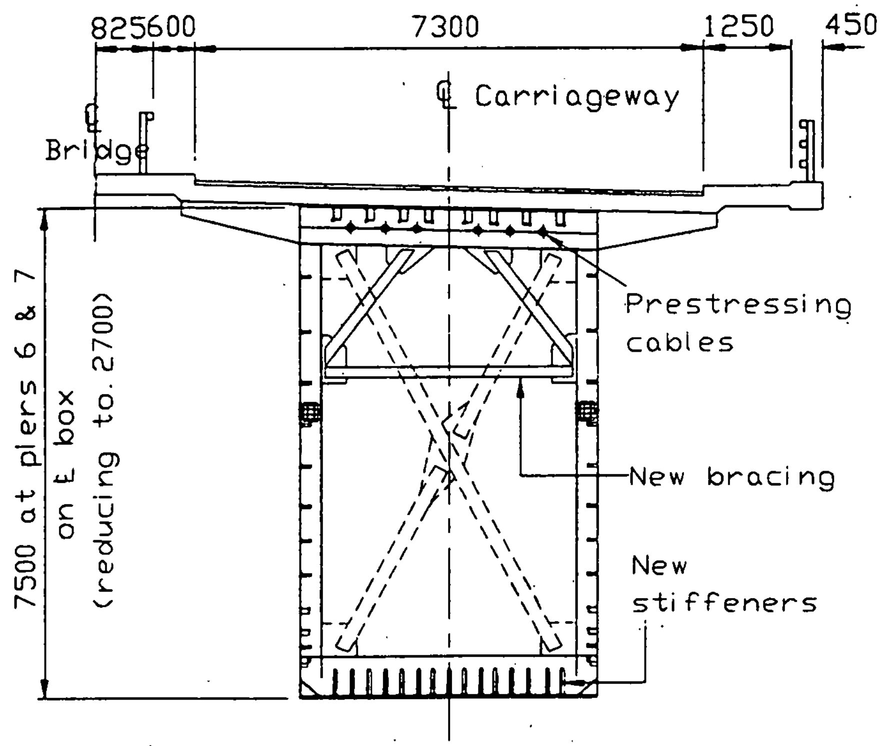


FIGURE 13 Friarton Bridge section, showing strengthening *(87)*.

of thin stiffened plate elements and an orthotropic deck. Problems with this structural form on other bridges led to publication of the Merrison Committee rules in 1975 (*90*), and some modifications to the Severn Bridge diaphragms were made in 1972. In 1976, a lengthy appraisal was initiated as a result of increased traffic loading that had taken place since the design criteria were set in 1960. A statistical analysis of observed vehicle weight and length distribution data indicated that lane loads were 20 to 160 percent higher than the design loading for some loaded lengths over 100 m (328 ft). This resulted in what is probably the largest strengthening project ever undertaken for perceived live load deficiencies, and various contracts between 1985 and 1991 upgraded the original live loading by a factor of three to take care of present and anticipated future commercial traffic (*89*). The need for the amount of strengthening done has been a source of controversy, however, and has been featured in the North American technical press (*91*). Other bridges are also being upgraded in Europe to take the 40-tonne (88-kip) trucks that will soon be adopted.

The requirement to keep all lanes open to traffic on the Severn Crossing during strengthening had interesting ramifications on the design criteria. Reinforcement was designed to participate in carrying full live load only if it was to be attached when traffic controls would prevent significant live load stress. Otherwise the reinforcement was considered effective for 67 percent of the live load only. To minimize the amount of strengthening, specific site conditions regarding traffic, wind, and temperature were built into the rehabilitation criteria. The design dead weight of the deck was lowered by reducing the usual dead load factor on the deck wearing surface; this was justified by carrying out site surveys and exercising extra control on the thickness during application.

Extensive strengthening of the suspension bridge deck box girders was required, mainly by welding additional internal plates, amounting to 3.3 percent of the original deck steel weight. Additional stiffening of the compression flanges at the piers of the approach viaducts was required. The components most effected by the increase in live load were the suspension bridge towers. The unique corrective measure chosen was to install and jack additional internal columns in each corner of the tower legs to relieve the existing towers of 22 percent of the dead load and 25 percent of the live load.

The most significant visual change from strengthening occurred on the Wye cable-stayed bridge, (Figure 14). To reduce the amount of deck box girder strengthening that would be required, the single-stay girder system was replaced by a twin-stay system, thus reducing the effective girder span lengths. The scheme entailed extending the towers and installing new stay cables and anchorages and local box girder strengthening at the anchorage locations. The additional steel amounted to 16 percent of the original steel weight. Full details of this £20-million project are given in four additional papers published by the Institution of Civil Engineers in 1992 (*92–95*).

Post-tension Members

Although post-tensioning of simple-span steel bridges is a standard method of strengthening, the method has been less frequently used for continuous bridges. Methods have been developed in Iowa and have been applied on two three-span continuous beam bridges for testing (*96-98*). The two-lane bridges were each 45.7-m (150-ft) long with four continuous beams. When subjected to legal live loads, both bridges were

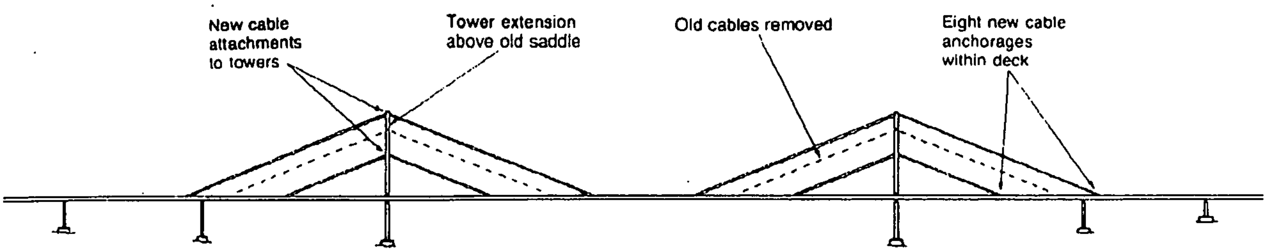


FIGURE 14 Wye Bridge, showing new cable system *(89)*.

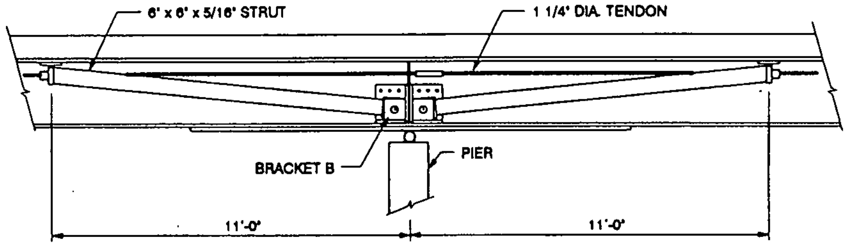


FIGURE 15 Superimposed truss system *(98)*.

overstressed in positive- and negative-moment regions. For Bridge 1, positive-moment regions were post-tensioned using anchorages attached to the bottom flanges. For Bridge 2, superimposed trusses over the piers of external beams (Figure 15) were employed in addition to post-tensioning in positive-moment regions. Both systems were considered cost-effective and practical, and have been incorporated in a design manual *(99)* for strengthening continuous-span composite bridges.

The Walnut Street bridge crosses the Tennessee River in Chattanooga with six pin-connected truss spans over 100 years old and 64 m (210 ft) and 98 m (321 ft) in length. As part of a major renovation program in 1991, the eye-bar bottom-chord members were post-tensioned to relieve them of some dead load and increase the truss live load capacity *(100)*. Reinforcing the eye bars was investigated but would have been difficult because of the close spacing of the bars. Thus, the bridge was strengthened by post-tensioning with straight and deflected strands. This method was not only more effective but also provided redundancy for the bottom-chord members, which proved advantageous as many of the eye-bars had existing flaws that could initiate cracks in these fracture-critical members.

The Burlington Skyway is a high-level crossing of the shipping channel to Hamilton Harbor, Ontario. With the opening of a second structure, the original four-lane structure was closed for upgrading with another lane added and increased live load design criteria. The main structure is a three-span arched through-truss with spans of 83.8 m (275 ft), 150.9 m (495 ft), and 83.8 m (275 ft); the deck is supported on floor trusses spanning 19.1 m (63 ft). Members in the floor trusses were significantly overloaded with the new design requirements, and post-tensioning was selected to strengthen the bottom chord *(101)*. The most effective end-anchorage location was in the concrete deck slab, which was to be replaced, enabling proper reinforcing for the anchorages to be placed (Figure 16). Twin prestressing bars were used, with a transfer plate added at the first node of the bottom chord, to anchor the bars and distribute prestress force to the members (Figure 17). All six bars on one truss were stressed simultaneously to minimize the chance of component buckling. The procedure proved to be cost-effective and structurally efficient.

A number of papers from the literature search deal with post-tensioning of steel structures *(76, 87, 102–106)*.

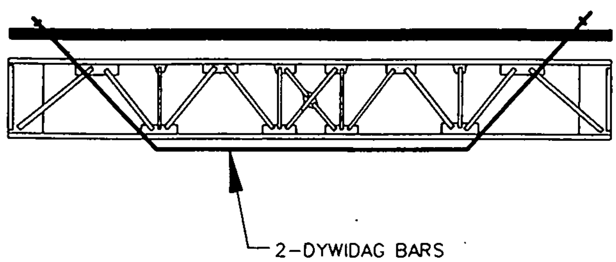


FIGURE 16 Burlington Skyway, floor truss post-tensioning *(101)*.

Combined Methods

Several agencies submitted bridge data sheets on bridges that had been strengthened using combinations of some of the standard methods. Three interesting strengthening projects designed by Modjeski and Masters are presented in this category.

The Worthington Bridge in Pennsylvania on U.S. 422 is a six-span continuous plate girder bridge, 18.3-m (60-ft) wide curb to curb, with pinned hanger hinges at two locations in the main girders. In 1992-93, the deck was replaced; the width was increased to 19.2 m (63 ft) and the load capacity was increased from HS20, as originally designed, to HS25. The span

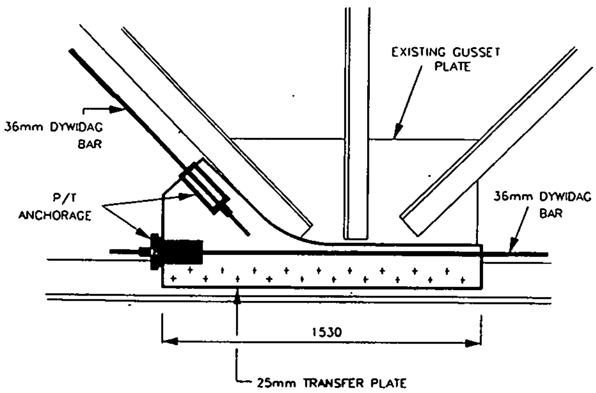


FIGURE 17 Burlington Skyway, post-tensioning transfer plate details *(101)*.

lengths varied between 15.2 m (50 ft) and 53.3 m (175 ft) and the contract cost was $7,390,000. The new deck was made composite with the stringers, and a new composite stringer was added over each plate girder (Figure 18) to increase girder capacity and reduce fatigue effects. By eliminating the hanger assemblies, the girders were made fully continuous (Figure 19), thereby increasing their strength and redundancy. In addition, tack welds used during fabrication were ground off to improve the fatigue capacity.

The Walt Whitman Bridge over the Delaware River in Philadelphia is being redecked in a series of contracts, and the original design capacity of HS20 is being upgraded to HS25 using the 1989 AASHTO specification. The bridge consists of a suspension main span, and approaches with beam spans from 15.2 m (50 ft) to 30.5 m (100 ft), girder spans from 32.3

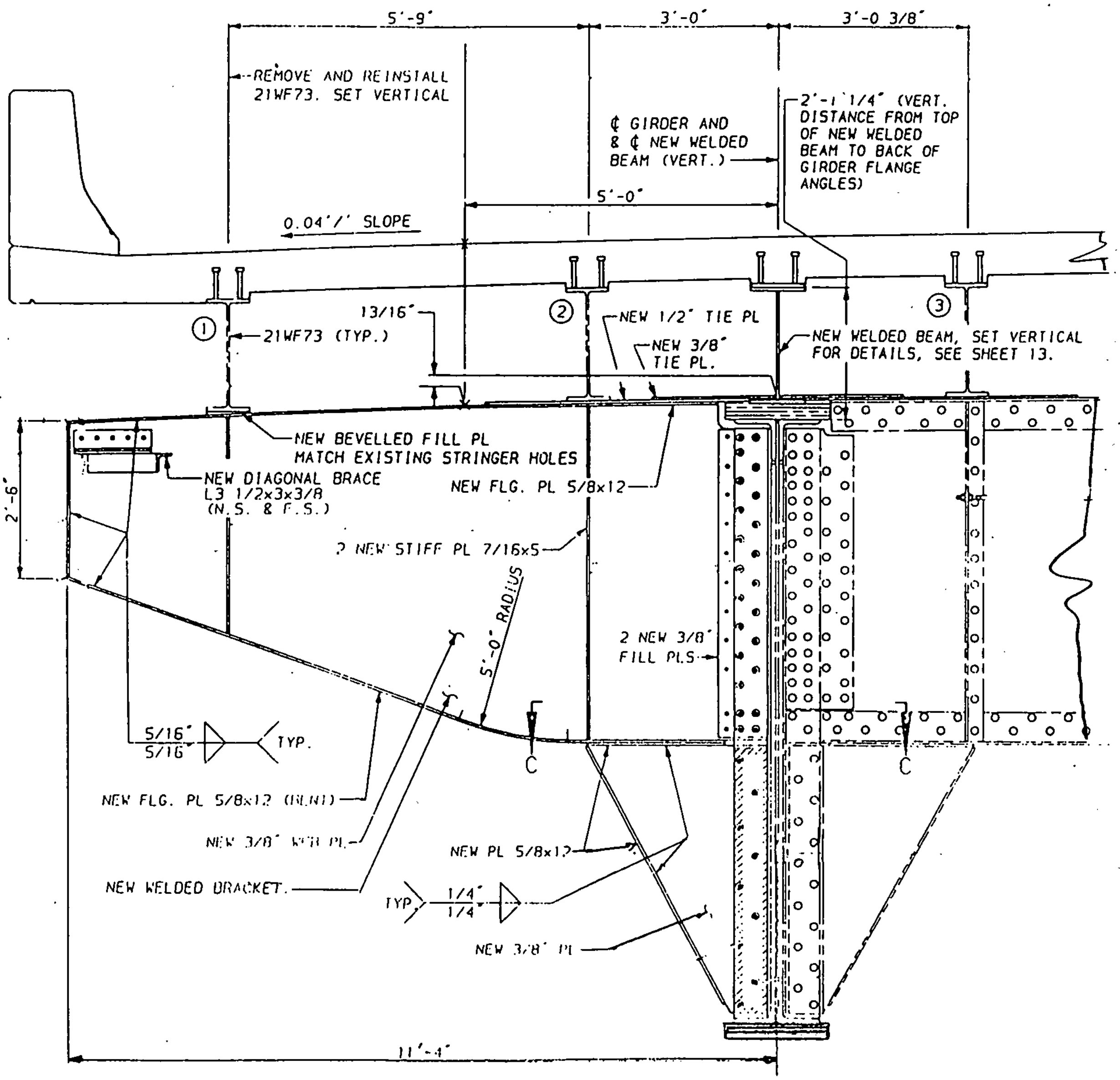


FIGURE 18 Worthington Bridge—US 422 (Modjeski and Masters).

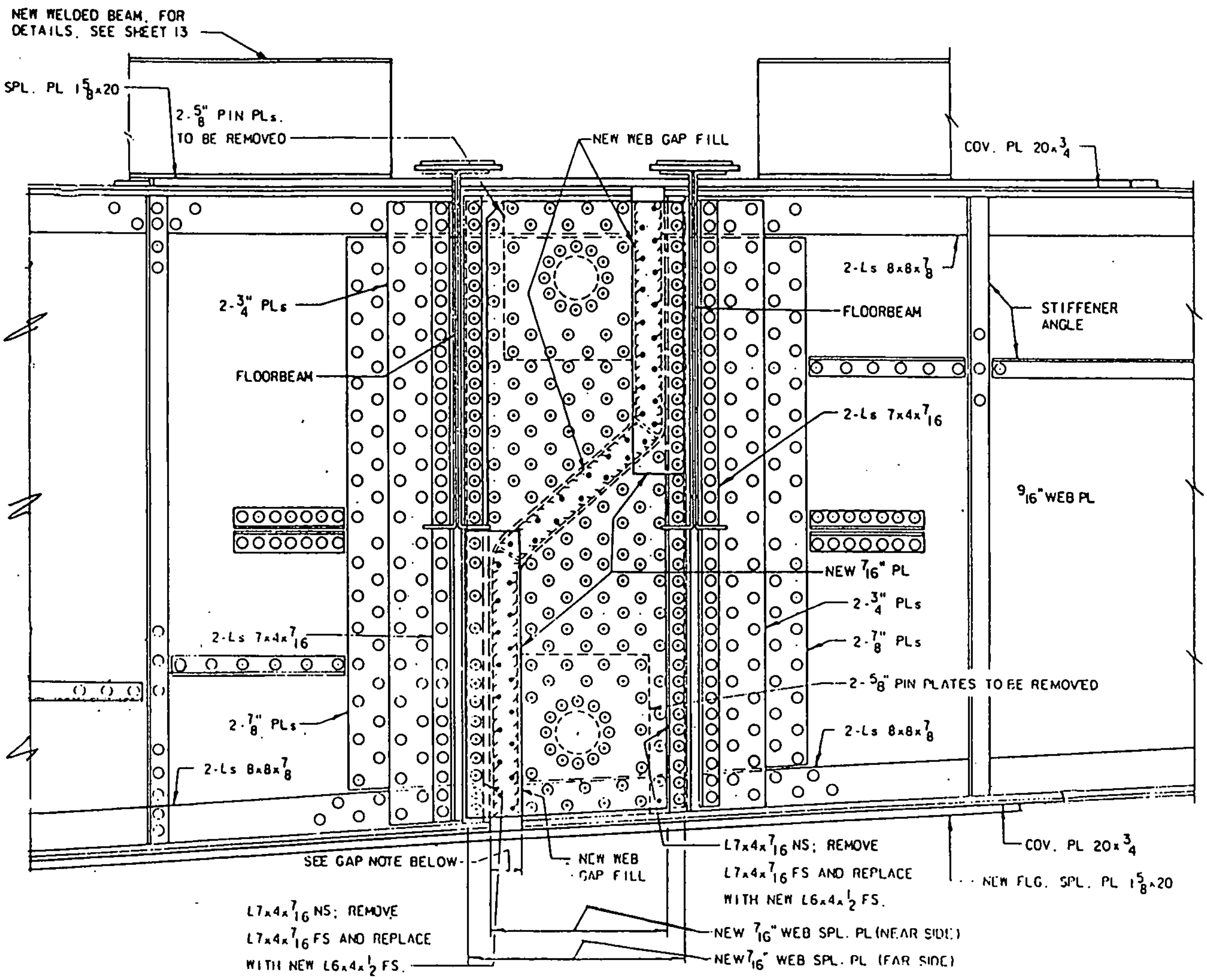


FIGURE 19 Worthington Bridge, hinge detail (Modjeski and Masters).

m (106 ft) to 40.5 m (133 ft), and truss spans from 54.9 m (180 ft) to 108.2 m (355 ft). The total redecking and upgrading cost will be more than $1 million. On the approach spans, the stringers are being made composite with the new deck, and tack welds are being removed to increase the fatigue life of the built-up longitudinal girders. On the cantilever deck-truss span, a back-up system of hanger rods is being added to provide redundancy at the pin and hanger locations (Figure 20).

The White Hill Overpass (I-581) in Pennsylvania is a 15-span structure that was redecked in 1992 at a contract cost of $4,194,000, with two percent of the cost for traffic protection. The live load capacity was increased from HS20 to HS25. The beam spans vary between 14 m (46 ft.) and 31.7 m (104 ft.) with two plate-girder spans of 36.6 m (120 ft). In the redecking, all girders, stringers, and beams were made composite by adding shear connectors. The shear capacity of the riveted plate girders was increased by adding web reinforcement plates. The moment capacity of the rolled beam floorbeams was increased by adding angles to the webs. Existing riveted cover plates were replaced with larger bolted plates, and bolted cover plates were added to the bottom flanges of existing rolled beams.

Other Methods

The Hagwilget Bridge is a small single-lane suspension bridge in British Columbia, constructed in 1937 for a maximum vehicle weight of 13.5 tonnes (29.7 kips) (Figure 21). By 1987, the open-grating deck had deteriorated from corrosion and fatigue and required replacement. As the local logging industry had been crossing the bridge with 52-tonne (144.4-kip) trucks, an upgrade in load level was sought. The chords of the stiffening truss were overstressed. The top-chord condition was corrected by making the new open-grating deck act compositely by a shear connection between the deck and the top chord. The bottom-chord condition was corrected with no strengthening or added material (*107*); rather the truss system was combined upwards by shortening the hangers and inducing moments

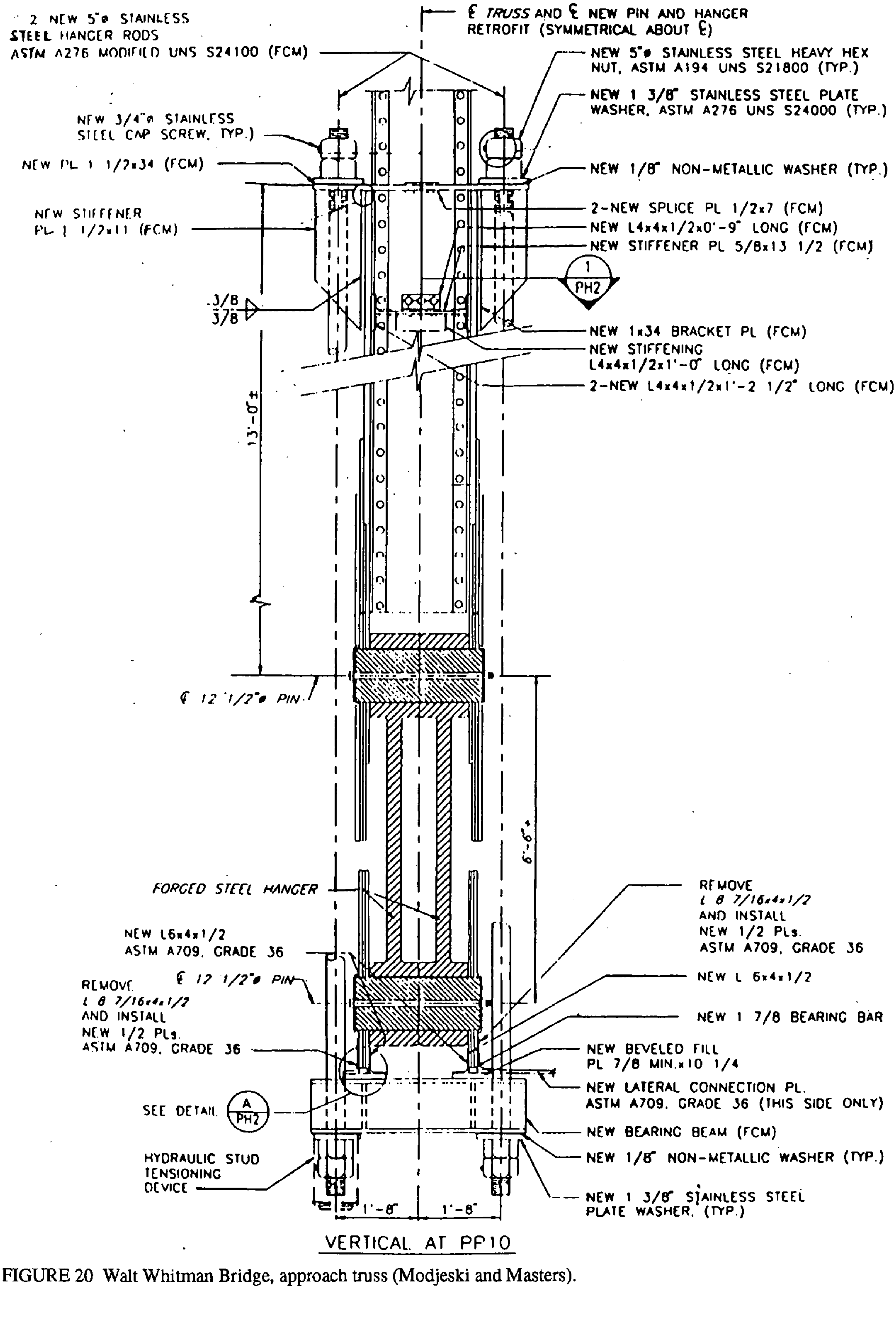


FIGURE 20 Walt Whitman Bridge, approach truss (Modjeski and Masters).

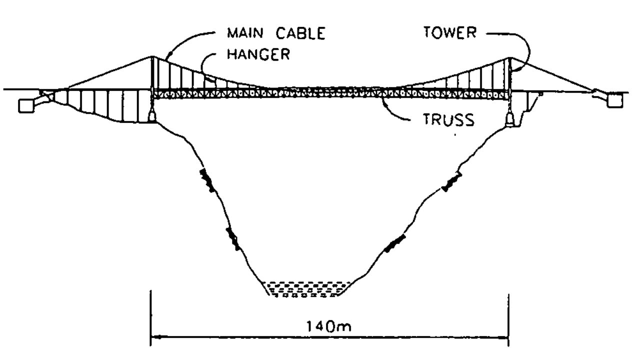


FIGURE 21 Hagwilget Bridge, general arrangement *(107)*.

opposite to those caused by the trucks. Adjustments to the hangers were made in seven passes to avoid local overloading and provide the required moment envelope. The resulting bottom-chord tension capacity then exceeded the demand (Figure 22). The renovation was completed in 1992 at a cost of $2.4 million Canadian.

The Avonmouth Bridge near Bristol in the United Kingdom is a major motorway river crossing, built in 1975, that requires strengthening to meet current loading standards. In 1999, 40-tonne (88-kip) trucks will be admitted into the United Kingdom and the bridge is undergoing assessment and strengthening design to meet this date (*108*). The present configuration is for three lanes in each direction, plus cycle paths and sidewalks on each side. The new layout will use the cycle/sidewalk space on one side to convert to four traffic lanes in each direction. The bridge superstructure consists of twin steel box girders with composite concrete deck on the approaches and steel orthotopic deck on the main river spans. The overall length is 1400 m (4,590 ft) with the 20 spans varying in length between 30 m (98 ft) and 174 m (570 ft). To strengthen for extra lanes and increased truck loads to current standards on such a large bridge would be expensive. Thus, to minimize the cost, designers have investigated specific site conditions to see what departures from the current standards could be justified.

Special restrictions will be imposed on the movement of heavy abnormal loads, which will be limited to a lane centered on a steel box girder. Study of the present usage of the cycle/sidewalk indicates the standard global loading is high and can be reduced at this location. Measurements on the existing wearing-course thickness show that the dead load factor for paving is higher than required, provided tight tolerances in construction can be controlled. The modified criteria will reduce this dead load factor for ultimate limit states to 1.20. The mill certificates from the original steelwork are available along with records of where each plate was used in the bridge. Designers are taking advantage of the fact that the yield points from the certificates exceed the guaranteed minimum of 355 MPa (51 ksi), and the actual values will be used in redesign. If the project had been treated as a new bridge to meet current standards, some 6000 tonnes (13,200 kips) of additional steel would be required. Gill et al. (*108*) note that:

> When strengthening an existing structure, minimizing the added strengthening steelwork has an additional benefit. The structure is designed to carry both its self weight and the applied loads; by minimizing the amount of extra steel added to strengthen the structure, the capacity for carrying traffic loading is increased.

By applying considerations outlined and modifying the current standards accordingly, significant savings in strengthening steelwork are expected.

REINFORCED CONCRETE

Strengthening reinforced concrete bridges will be considered under two of the standard categories, improve member strength and post-tension members. A third category, bonded steel plates, has been added, as there are many examples of this method in Europe, particularly in the United Kingdom, but few in North America. The application of ACM sheets for strengthening will not be considered in this section. As it is still in the developmental phase in North America it will be considered in chapter 7.

Improve Member Strength

Kansas DOT has a number of hollow-tube slab bridges that have been rehabilitated using a 127-mm (5-in.) thick bonded overlay. At the same time, openings to the tubes enable them

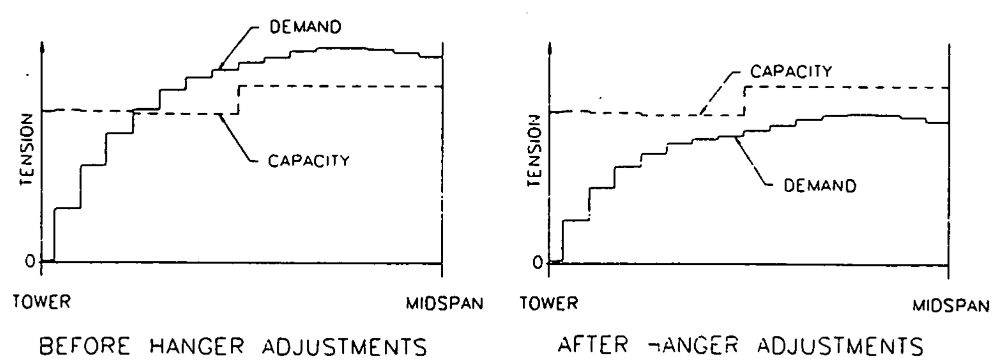


FIGURE 22 Hagwilget Bridge, tension in bottom chords of truss *(107)*.

to be concrete filled after rebar stirrups are placed (Figure 23). When load rated, the overlay is assumed to act compositely with the T-sections formed by the overlay and the concrete-filled cells.

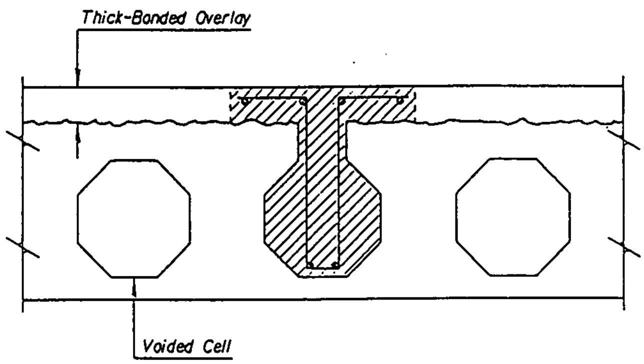


FIGURE 23 Hollow tube slab, with thick bonded overlay (Kansas DOT).

CALTRANS participated in a test program at the University of California, San Diego, for strengthening a T-girder bridge (*109*). A 25-year-old section of the obsolete Gepford Overhead was brought to the laboratory and a precast soffit slab panel was added to it (Figure 24). The 76-mm (3-in.) thick soffit panel was pretensioned with nine 13-mm (0.5-in.) Grade 270 strands prior to casting the high-strength concrete. The panel was supported by 10 set bolts, whereas the horizontal load transfer between the panel and the T-girder was designed to be achieved by the epoxy bonding. Tests showed that the panel not only increased the flexural capacity but also improved the transverse load distribution. The epoxy bonding was satisfactory for the overload tests, but a horizontal shear failure occurred in the concrete cover of the T-girder before reaching the ultimate limit state. For suitable performance at this level, additional mechanical anchoring of the soffit panel through dowels into the core of the T-girders is proposed.

Between 1949 and 1963, the province of Alberta built more than 30 cast-in-place concrete T-girder bridges with designs based on HS20 truck loading (72). Since then, the legal truck load has increased to nearly twice the HS20 truck weight, and the allowable shear stress in concrete has decreased. A shear assessment was carried out on seven of the bridges, followed by a refined assessment using more accurate shear capacity equations, in-situ concrete strength determinations, and a grid analysis for improved load distribution. As a result of these assessments, the number of bridges requiring shear strengthening was reduced to three, and strengthening was achieved by using high-strength rods in an external stirrup assembly (Figure 25). The Highwood River bridge was strengthened in 1987 by installing 804 such assemblies, for a contract cost of $112,000 Canadian, or $139 Canadian per external assembly.

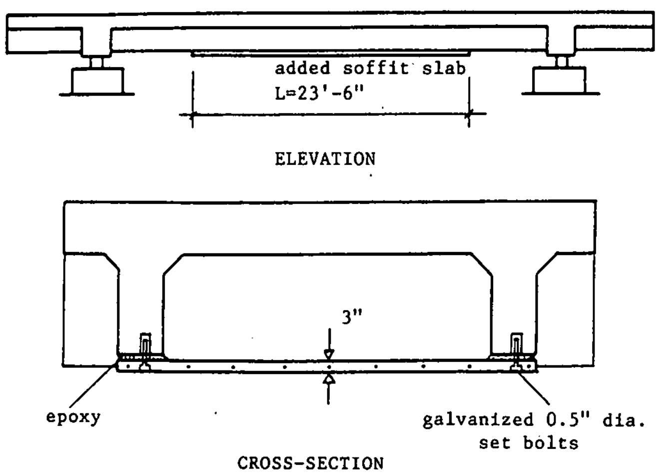


FIGURE 24 Bottom soffit slab panel addition (*109*).

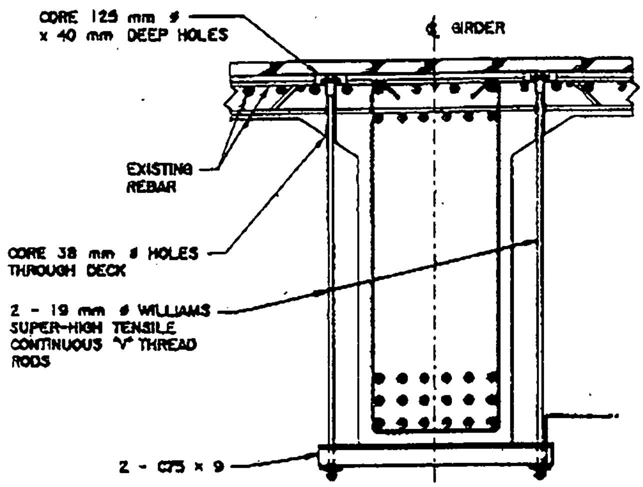


FIGURE 25 Shear strengthening system using high strength rods (*72*).

Steel plates can be bolted to reinforced concrete beams to increase flexural and shear capacity. A paper from Australia (*70*) covers both theoretical and practical aspects of soffit plating and side plating, and limitations of the methods.

Post-tension Members

The addition of external post-tensioning cables has become a widely used method to increase the capacity of prestressed concrete bridges. The technique has not been used as frequently on reinforced concrete bridges. One of the difficulties with stressing reinforced concrete members is that it creates a partially prestressed structure, a system not addressed by most bridge codes. This situation has been addressed by Naaman (*110*) in a review of some code provisions and in a proposed analytical solution to determine the resistance of partially prestressed beams using unbonded tendons.

With the addition of a streetcar track, a reinforced concrete T-beam bridge in Romania required strengthening for heavier loading (111). This bridge has a central span of 24.2 m (79 ft) and two cantilever spans of 9.5 m (31 ft), with transverse concrete diaphragms approximately every 3.5 m (11.5 ft). A tendon profile was selected to reduce dead load moments at the

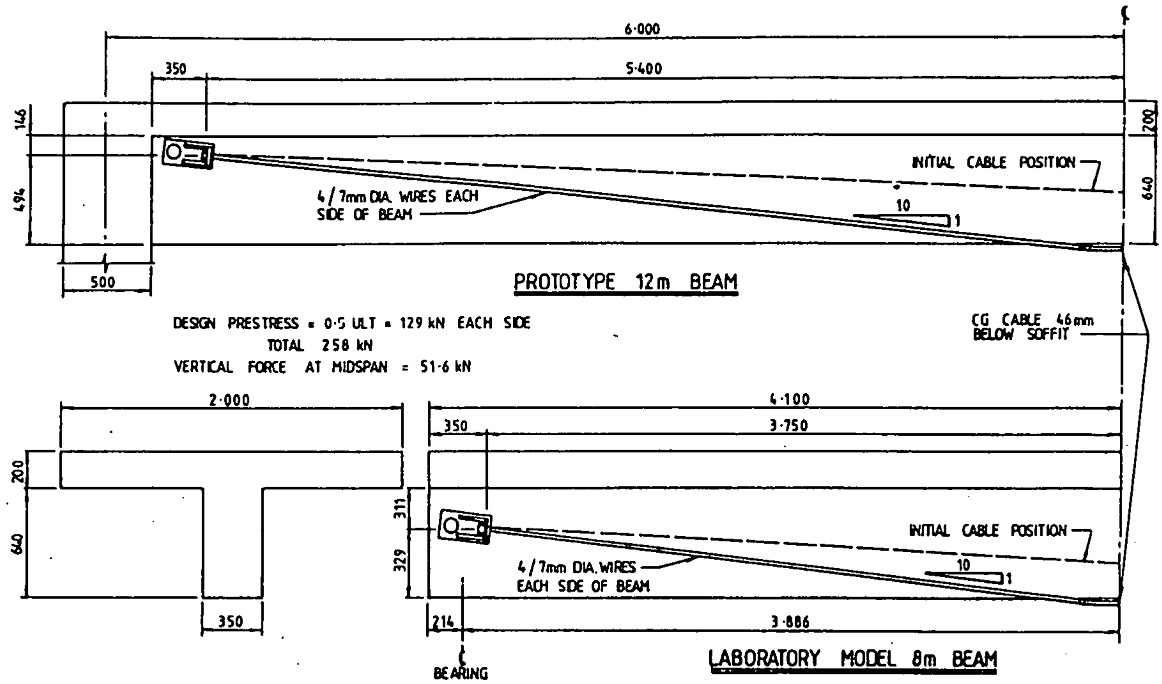


FIGURE 26 Bowstring post-tensioning system *(112)*.

supports and midspan; the tendons and steel ducts were passed through holes drilled in the diaphragms and were anchored at each end of the cantilever spans.

A different method of applying external post-tensioning has been developed in Australia (*112*). Called "bowstring" post-tensioning, the method was developed using laboratory models (Figure 26) and later applied in the field to the Kaiawha Stream bridge, a T-beam structure with a 15-m (49-ft) simple span. With this method, the cable is initially installed high then pulled down by jacking against the soffit to develop the required prestress load. A bracket with a large pin through the beam web provides end anchorage. Laboratory tests were conducted with one pull-down position, whereas the field installation required two positions because of the presence of a central diaphragm. The field installation was estimated to be between 10 and 20 percent less expensive than the standard post-tensioning method, probably because of the simplicity of the stressing system.

Bonded Steel Plates

Since 1974, more than 30 concrete highway bridges in the United Kingdom have been strengthened by using externally bonded steel plates (2). The method has been used in other European countries, such as Finland (*113*) and Portugal (*114*), but the United Kingdom has by far the most European applications and continues to use the method. The survey questionnaire did not identify any North American applications, but one responding consultant, T.Y. Lin International, has applied steel plate bonding to 202 concrete cap beams on an elevated transit structure in Taipei. With a contract cost of $6 million in 1995, this could be the method's largest application. The country with the most applications, however, is Japan. In 1994, the U.K. Department of Transport issued an advice note on the subject as part of their Design Manual (*115*), which covered all aspects including design, materials, quality control, and specifications for the epoxy-resin adhesive. The largest application in the United Kingdom is for the Bolney Flyover (*116*) where a total of 676 plates were applied in up to three layers. The plate bonding method is not without concerns, however, and questions have been raised about the durability of the adhesive (*117*), possible corrosion at the steel/adhesive interface, and acceptability of the method at ultimate limit states. Nevertheless, an examination of the first major bridges strengthened this way in the United Kingdom, the Quinton bridges, indicated satisfactory structural performance 20 years after strengthening (*6*).

From an installation viewpoint, however, this method has a number of disadvantages including transportation and handling of heavy plates, limited delivery lengths of plates, the need to prepare the steel surface for bonding, and the need for expensive falsework to hold the plates in position during curing of the adhesive. These items, plus the concerns about durability, have probably had a negative effect and account for the limited interest in North America for this strengthening method.

PRESTRESSED CONCRETE

The most obvious and common method of increasing the strength of prestressed concrete bridges is the addition of external prestressing. This has been covered in NCHRP Reports 293 (*1*) and 280 (*118*) for I-beam bridges, the most frequently used bridge type since the 1950s. An example of an I-beam bridge is shown in Figure 27 (*82*). Side-by-side precast box beams have been strengthened by adding epoxy-coated strand

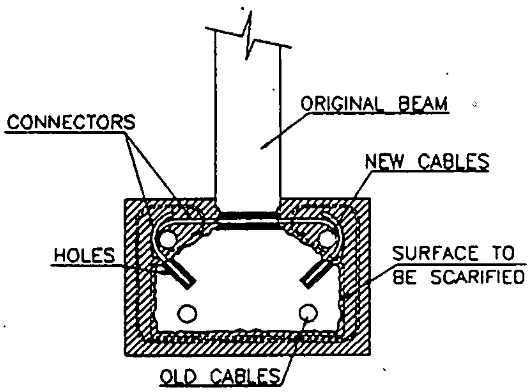


FIGURE 27 New cables incorporated in I-beam *(82)*.

and bonding with gunite; this method was the subject of a Pennsylvania DOT and FHWA research project (*119*). As these are now standard methods, this section will only address the addition of cables inside box girders generally built by segmented methods, either precast or cast-in-place. These boxes are large enough for access so that anchorages and deviator blocks can be added inside for the cable installation. This form of large single-cell box construction was used in Europe some 10 to 15 years before its introduction to North America in the mid 1960s. There is thus a greater history of strengthening these structures in Europe, and this is reflected in the literature search. In the late 1950s, a number of segmental bridges in France were built by balanced cantilever methods, with a hinge at the center of the span. Because of a lack of knowledge of concrete creep at the time, though, large mid-span deflections required strengthening by restraining rotations at the hinges to make the spans continuous (*79*). Bridges designed as continuous structures also had some deformation problems in Europe as a result of insufficient information on temperature gradient effects, as well as creep, elastic modulus, and friction values. Although the ultimate strength was usually adequate, the service load capacity could be impaired. These bridges were built with cables fully bonded inside the webs or flanges. They were generally strengthened to restore service live load capacity, when necessary, by the addition of unbonded tendons inside the boxes *(80, 81)*. In France, about 50 box-girder bridges have been strengthened since 1970 by additional post-tensioning, and the method is considered to be very effective (*79*). In Germany, of the prestressed concrete bridges built since 1950, five have had to be replaced, and more than 25 have been strengthened with external tendons (*81*).

The strengthening of box girders for increased live load capacity by the use of external tendons has a number of advantages.

- The work can normally be done under traffic, which is often a requirement (*83*).
- The method adds very little extra dead weight and does not alter the appearance of the bridge.
- The deviators can often be provided by drilling through existing diaphragms.
- The end anchorage can sometimes be provided in the end diaphragms, but may need additional anchor blocks.
- Straight tendons may be used when only flexural strength is deficient. Draped tendons may be more effective, however, they also increase the shear capacity.
- The external cables are visible for inspection.

The most widely used tendon comprises seven wire strands protected by a polyethylene outer tube, with a cement grout injected into the tube.

A recent example of this strengthening method is the Oleron Viaduct in France (*120*). Constructed in 1966 by the balanced cantilever method using precast single-box segments, the bridge's total length is 2,862 m (9,387 ft) consisting of 46 spans varying in length, but with the majority 79 m (259 ft). The viaduct is made up of nine continuous units, each about 320-m (1,050-ft) long. Figure 28 shows the external cable layout for the haunched girder spans, and details of the deviation saddles provided at the piers and at mid-span. Continuous tendons over the full 320-m (1,050-ft) length of each unit were provided, with new anchorage blocks at each end.

The first precast segmental bridge in North America was the Lièvre River Bridge in Quebec, built in 1967. The segments were not epoxy glued, as became the normal practice, and they started opening at the joints because of insufficient prestress. In 1987, the bridge was restored to its design live load capacity by external post-tensioning. The first cast-in-place segmental bridge in North America was also constructed in Quebec, crossing the Mulets River near Ste. Adèle, about 70 km (42 mi.) north of Montreal. The crossing consists of twin single-box structures with continuous haunched girder spans of 40.38 m (132.5 ft), 80.77 m (265 ft), and 40.38 m (132.5 ft). Built by the balanced cantilever method with traveling forms, the bridge opened in 1964. The design criteria were much the same as used in Europe at that time, and, predictably, the bridge exhibited similar behavior through the years of excessive mid-span deflection and some cracking, and it was decided to strengthen the bridge (*121*). The original design did not allow for moment redistribution from concrete creep or for thermal gradient effects. The use of lower values of friction and wobble coefficients than currently considered realistic resulted in actual prestress forces below those anticipated at the design stage. The structure was reanalyzed and indicated tensile stresses at mid-span of 4.3 MPa (615 psi). The strengthening design allowed for temperature gradient effects and an increase in live load to reflect current traffic loads. Straight longitudinal tendons were attached in the end spans and at mid-span, as shown in Figure 29. The tendon anchorages were provided by concrete blocks attached by prestressed bars on each side of the web. While this technique exposed tendons, which are protected by polyvinyl chloride (PVC) ducts, outside the boxes, it eliminated transverse flexural stresses that would result from blocks on the inside only (Figure 29). The box girder cracks were grouted before the tendons were stressed. The strengthening contract was completed in 1988, with the bridges being kept open to traffic at all times, but with some lane closures.

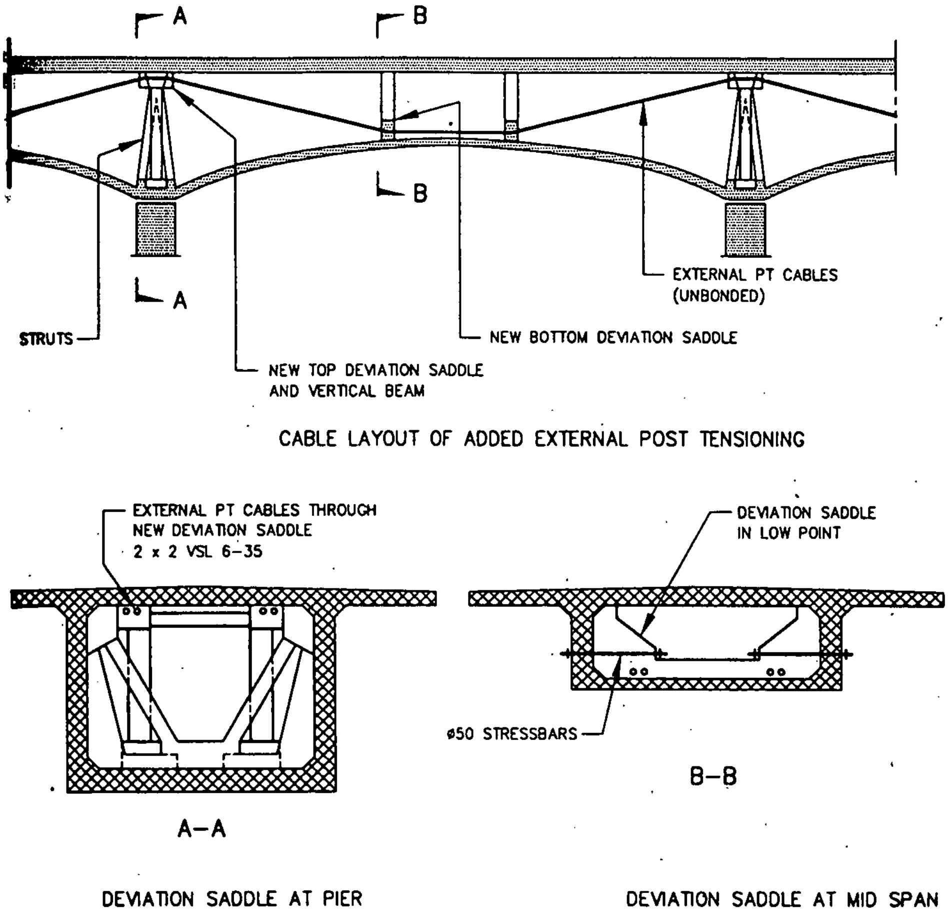


FIGURE 28 Oleron Bridge, external post-tensioning details *(120)*.

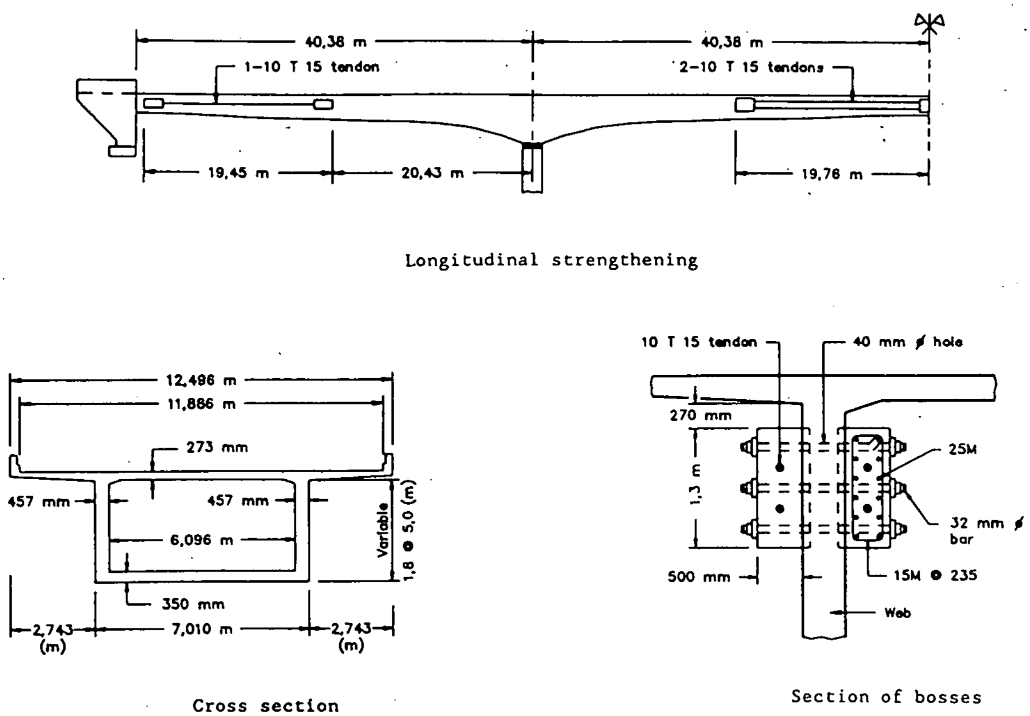


FIGURE 29 Mulets River Bridge, strengthening by post-tensioning *(121)*.

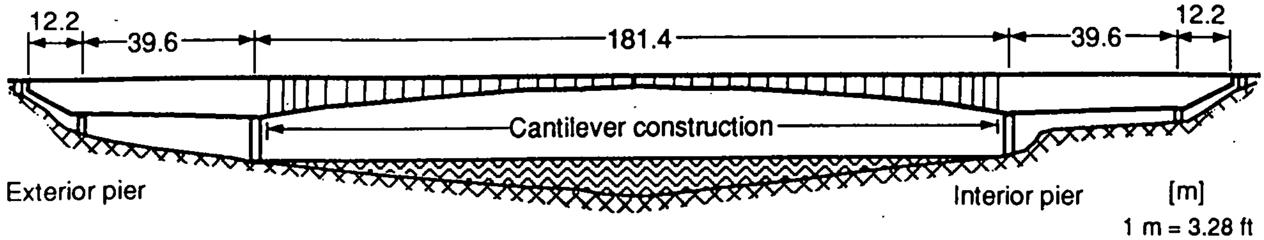
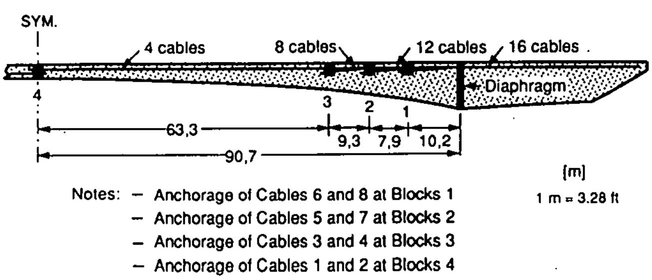
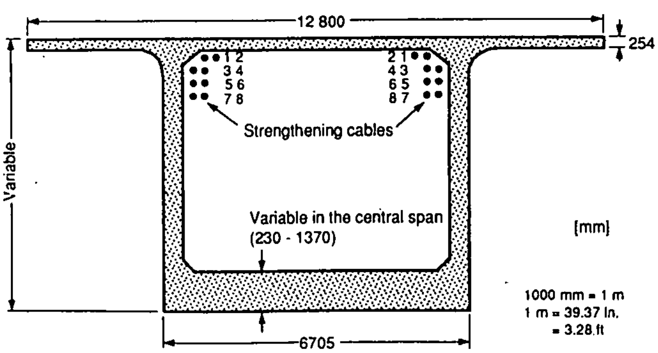


FIGURE 30 Elevation of Grand-Mère Bridge *(122)*.

Most North American segmental bridges were built after the mid 1970s when their behavior and the required design and material data were better understood. While no agencies reported on strengthening of such bridges, the procedure has taken place and is reported in two papers on external post-tensioning of a major cast-in-place segmental bridge at Grand-Mère, Quebec (*122, 123*). Built in 1977, the single-cell structure has a main span of 181.4 m (595 ft) with short counter-weighted end spans (Figure 30) (*122*). With a mid-span depth of only 2.9 m (9.5 ft.), the bridge has an unusually high span-to-depth ratio of 62.6. The first paper mentioned above (*122*) includes discussion of various problems during and after construction that resulted in local cracking and increasing main span deflection, which reached a value of 39.4 mm (15.5 in.). The distress was caused by insufficient prestress and some incorrect design assumptions, particularly regarding thermal effects. The bridge was strengthened in 1991 by additional prestress of 30 percent, shown in Figure 31 (*122*). Each cable consisted of several individually lubricated sheathed strands



Strengthening cables and anchorage blocks.



Location of strengthening cables inside cross section.

FIGURE 31 Added post-tensioning cables, Grand-Mère Bridge *(122)*.

grouted in a PVC duct, and the prestress was successively applied to each strand. Throughout the seven-month contract, which cost $1.3 million Canadian, the bridge was kept open to traffic, but with some lane and speed restrictions. The second paper listed above *(123)* reports on the research, testing, and monitoring program that was done in conjunction with the strengthening. The two papers provide useful data that may be applied to any future segmental bridge strengthening projects.

WOOD

Various types of wood bridges have been strengthened by post-tensioning, usually using bar systems. Oregon DOT reported on upgrading a timber stringer bridge crossing Bear Creek in 1993. The bridge was built in 1933, to H-15 live loading, which was inadequate for legal and permit loads. Eight stringers spanning 8.84 m (29 ft) were strengthened with a king-post system, using two rods per stringer. The live load capacity was increased by 25 percent at a cost of only $9,000.

In British Columbia, the Ministry of Transportation and Highways have post-tensioned the bottom chords of two timber Howe truss bridges, each spanning 55 m (180 ft), to carry live loads in excess of the original H-15 design load. The Willow River Bridge was strengthened in 1992, and the Bulkley River Bridge in 1994. On the latter bridge, the deck was replaced with a stress-laminated timber deck.

The stressing of laminated timber decks is the most significant new strengthening method developed for wood bridges in recent years. Longitudinal nail-laminated decks tend to loosen with time. By applying transverse post-tensioning, the planks are compressed and the system becomes a more rigid slab with improved load distribution capability. The method was developed in Ontario, where 10 laminated deck bridges were upgraded using this method prior to 1987 (*67*). Figure 32 shows a typical installation, with prestressing bars over and under the deck. The upper bars are covered with an asphalt wearing surface. The required prestress level is low, but a restressing cycle is normally needed because of the creep of the wood laminates. The concept has since been applied to new laminated decks and is covered by recent codes in Canada (*30*) and the United States (*35*).

An interesting example of the use of a new stress-laminated deck is the Sioux Narrows Bridge in northern Ontario (Figure 33). This wooden Howe truss bridge was built in 1936, and with a span of 64 m (210 ft) is thought to be the

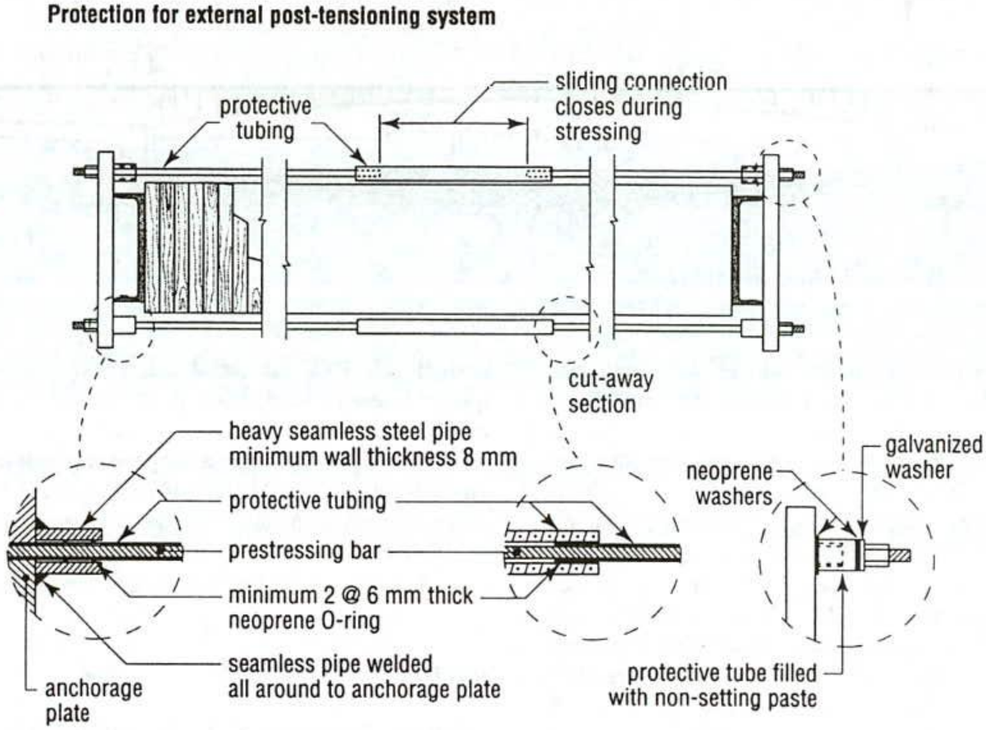


FIGURE 32 Post-tensioning of wood-laminated deck *(30)*.

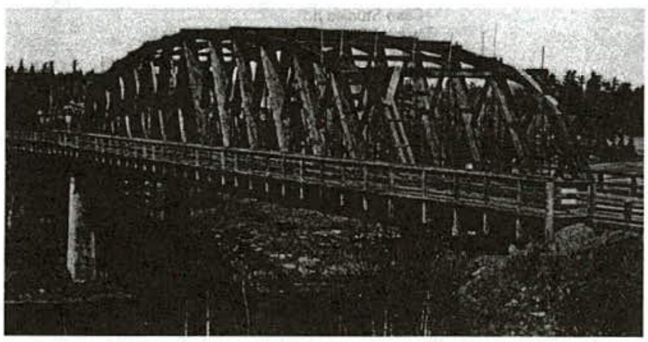


FIGURE 33 The Sioux Narrows Bridge.

longest single-span wood highway bridge in North America. The bridge is listed under the Ontario Heritage Bridge Program and is a fine example of wood bridge construction, using British Columbia Douglas fir in sizes no longer available. In an effort to keep the bridge in service beyond the lifetime of typical wooden bridges, a detailed condition survey and analysis were undertaken (*124*). The load capacity of the main trusses was found to be adequate; the capacity analysis even applied current loads in excess of the design loads. However, there was a significant overstress in the transverse floor-beam king-post and tie bars (Figure 34). The original floor system consisted of a transverse nail-laminated wood deck on longitudinal

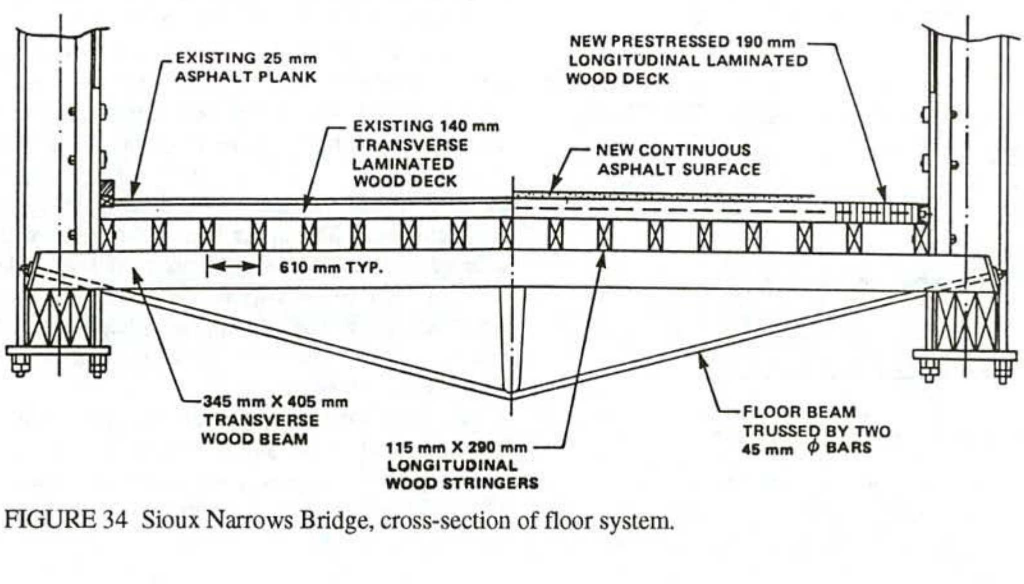


FIGURE 34 Sioux Narrows Bridge, cross-section of floor system.

wood stringers, and the deck had deteriorated and needed replacing. By using a new longitudinal stress-laminated deck, rather than transverse, it was calculated that longitudinal load distribution would be improved enough to reduce the stresses in the floorbeam to acceptable levels. The new deck was constructed in two halves to keep one lane open to traffic. The new deck was stressed with 25-mm (1-in.) bars in the center of the deck in predrilled holes at 1.52-m (5-ft) centers, tensioned by hydraulic jacks. The improved load distribution characteristics were confirmed by full-scale load tests carried out before and after the deck replacement (Figure 35). The tie bars of eight floorbeams were strain gauged, and the measured bar strains were reduced by 35 percent. No further strengthening of the floor system was needed.

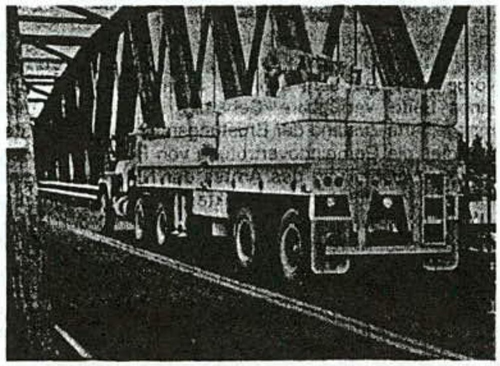


FIGURE 35 Load testing the rehabilitated Sioux Narrows Bridge.

CHAPTER FIVE

SUBSTRUCTURE PRACTICES

As the emphasis of this synthesis is on superstructures, strengthening of substructures for increased live load is a minor consideration. Two questions were put to the agencies in the questionnaire, (Appendix A) however, relating to substructures. The first asked for information on any substructure modification used specifically to increase live load capacity, and the second related to the effect on seismic behavior of any live load modifications to the structure. This chapter covers the responses received, but as there were only six replies to the second question, the discussion on seismic behavior is augmented by information from the literature search and current research.

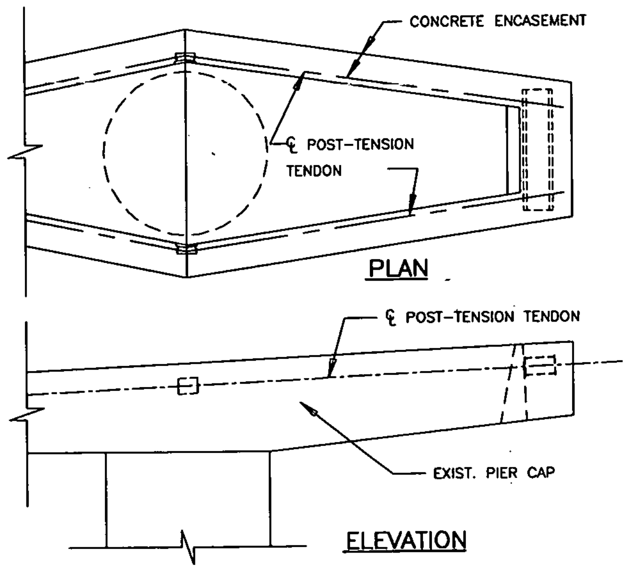


FIGURE 36 Pier cap strengthening by post-tensioning (Connecticut DOT).

STRENGTHENING METHODS

Sixteen agencies reported on substructure strengthening, but it is uncertain that a need to increase live load capacity was the prime reason in every case. The strengthening methods included:

- Adding piles to a pile bent,
- Encasing and bracing piles,
- Adding steel jacket and concrete to slender pier columns,
- Thickening concrete wall-type piers,
- Deepening pier cap on column bent,
- Installing additional pile bents, and
- Post-tensioning hammerhead pier cap.

Additional steel or timber frame bents have been added to about 15 timber stringer bridges in Oregon. Originally designed for H-15 loading, these bridges were found to be inadequate for legal or annual permit loads. Span lengths varied between 5.8 m (19 ft) and 12.2 m (40 ft), and the additional bents could solve both substructure and superstructure deficiencies. At an approximate contract cost of $10,000 per bent in 1994-95, this was a cost-effective solution for a projected service life in excess of 15 years.

Post-tensioning of hammerhead pier caps was reported by four agencies, including Connecticut. An example of how this agency employed the method is shown in Figure 36. When a design deficiency was discovered, the agency used post-tensioning on the pier cap to bring it up to design live load capacity. The procedure followed was:

- Concrete a wedge at the ends of the pier cap, doweled, epoxy-bonded, and reinforced to provide a vertical face;
- Install bearing plate and weldment with two rocker bearings and two anchorages for 12–15-mm (0.6-in.) tendons;
- Stress multistrand tendon on each side to 1,734 kN (390 kips) after losses and grout ducts; and
- Concrete a reinforced and doweled encasement at the sides and ends of the cap.

In Rhode Island, a pier cap was strengthened in the tension zones by external post-tensioning using anchoring cones inserted into steel tubes attached to both ends of the cap. A 20 percent increase in live load capacity was achieved for the under-designed pier cap at a contract cost of $100,000 in 1993.

A common seismic retrofit method for columns in California (*125,126*) is to encase them in a steel jacket. More recently, wrapping columns with ACM sheets has become an attractive alternative, particularly for round columns (*127*). These methods, although developed for seismic strengthening, can be of use to strengthen columns deficient in shear, bending, or axial load capacity. Tests on columns confined with ACM wrapping have shown strength increases of up to 70 percent for round columns, with ductilities seven times those of plain concrete columns (*128*). Square columns showed similar ductility increase but negligible increases in strength as a result of stress concentrations and ACM failures at the corners. The method has potential for restoring the strength of deteriorated concrete columns, and Ontario is investigating this aspect (*129*). A demonstration project is being carried out by MTO using glass and carbon fiber-reinforced plastic wrapping with a sprayed-on protection (Figure 37).

SEISMIC CAPACITY

Some of the live load strengthening methods used could affect the seismic capacity or behavior of a bridge. Agencies

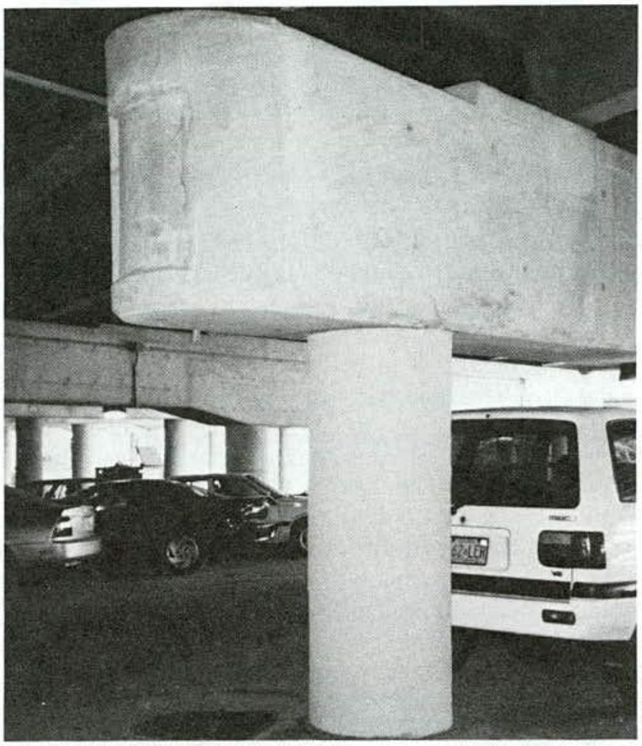


FIGURE 37 FRP wrapped column.

were asked if this effect was assessed when modifications were being designed. Only six agencies replied in the affirmative.

In one case the structural stiffening for live load capacity increased the dead load. As the bridge was at a site with an acceleration coefficient between 10 and 15 percent, the effect of the extra dead load on seismic behavior was checked. In another case, where a wall-type pier was thickened for live load capacity, the rebar was detailed to enable plastic hinge formation to take place, and the footing and piles were checked for earthquake effects. One agency noted that its superstructure strengthening had no adverse effects on seismic behavior, and could improve it when reducing dead loads or providing girder continuity. Providing girder continuity, however, does not necessarily improve seismic performance. The articulation at the piers will change, and depending on where fixity is established, the seismic performance of individual substructure elements may or may not be improved. Load sharing bearings or shock transmission units (*130*) may be required to maintain the same longitudinal load distribution to piers as assumed in the design of the original simple-span layout. It would appear, perhaps, that more consideration should be given to the effect on seismic performance of live load upgrading than apparently has been the norm in the past.

The reverse effect, namely the increase in live load capacity as a result of seismic retrofitting, could also be of interest. The current methods of increasing seismic resistance and ductility of concrete columns by jacketing with steel plates or wrapping with ACM sheets will increase live load braking capacity. Depending on the extent of the jacketing or wrapping of the column, the vertical live load capacity could also be increased.

CHAPTER SIX

DECISION MAKING

This chapter reviews the extent to which the responding agencies use bridge management systems in the decision-making process to strengthen existing bridges for increased live load. This chapter also presents a decision-making matrix that may assist in strengthening method selection.

BRIDGE MANAGEMENT SYSTEMS

Specific Bridge Management Systems

The following bridge management systems have been developed in recent years at the national level for use by various agencies: BRIDGIT (*131*) and PONTIS (*132*).

State Bridge Management Systems

The following states have developed or are developing their own bridge management systems: Alabama (*133*), Connecticut (*134*), Indiana (*135*), Iowa (*136*), New York (*137*), North Carolina (*138*), Pennsylvania (*139*), South Carolina (*140*), Texas (*141*), Virginia (*142*), and Washington (*143*).

Provincial Bridge Management Systems

The following Canadian provinces have developed or are developing their own bridge management systems: Alberta (*144*) and Ontario (*145*).

International Bridge Management Systems

The following countries are developing or have developed their own bridge management systems: Australia (*146*), Denmark (*147*), Finland (*148*), Poland (*149*), Thailand (*150*), and the United Kingdom (*151*).

PRIORITY SETTING MODULES IN PONTIS AND BRIDGIT

PONTIS and BRIDGIT do not specifically prioritize bridge strengthening. However, for existing bridges or their components at the network level, decision-making modules are available for prioritizing maintenance, repairs, and replacements (MR&R), and for improvements related to functional deficiencies.

PONTIS initially separates MR&R decisions from improvement decisions. In determining MR&R decisions, PONTIS uses a top-down approach. Optimal actions are first determined for each element of a bridge and are then brought together to form the optimal MR&R actions for each bridge. The total costs and benefits of the actions and the total benefit/cost ratio of the MR&R actions are calculated for each bridge. If a separate budget is specified for MR&R actions, projects are selected in order of decreasing benefit/cost ratio until the budget is exhausted. If the budget cannot accommodate all the projects, the remaining projects form the backlog and are carried to the next period. In the next period, projects on backlog go first, followed by new projects sorted by total benefit/cost ratio.

In determining functional improvement actions, PONTIS compares each bridge with standards for level of service. Bridges that do not meet the standards are identified for improvement actions. The total costs of the improvement actions, associated benefits, and the total benefit/cost ratio are calculated for each bridge.

If a separate budget is used for improvements, projects are selected in decreasing order of benefit/cost ratios until the budget is exhausted. If projects remain, the projects are carried to the next period and form the backlog.

If a total budget is specified for MR&R and improvements, the costs and benefits for all MR&R and improvement actions for the bridge are summed and the total benefit/cost ratio is calculated. Similar to separate budgets, projects are selected for the total budget in decreasing order of benefit/cost ratio until the budget is exhausted. Projects that remain are carried over to the next period, as described above (*152*).

BRIDGIT follows a bottom-up approach in determining network needs and priorities. Optimal MR&R and improvements are determined for each bridge in the network, and the results of this bridge level optimization are used to determine network needs and priorities. BRIDGIT uses a cost-effectiveness index (CEI) to determine which alternatives to select for a structure. The CEI is the rate of internal return between two alternatives. The CEI for an alternative is determined by comparing the present value cost of agency and user life-cycle costs with the present value of the do-nothing alternative. For each bridge, BRIDGIT determines the CEIs of all feasible alternatives in each period of the optimization analysis horizon. The alternative with the highest CEI over the analysis horizon is the optimal choice for that bridge. If the budget is unlimited, the alternatives with the highest CEIs are selected and allocated to the period of the analysis horizon in which they should be optimally implemented. If insufficient funds are available to match the needs, other lower cost alternatives are evaluated using an incremental benefit/cost approach. Alternatives with the highest CEI are iteratively selected until the budget constraints are satisfied. For any period, the selected actions are listed in order of CEI (*153*).

SURVEY OF CURRENT PRACTICES

All 50 states, nine Canadian provinces, FHWA, Public Works Canada (PWC), and a selected number of consultants in the United States and Canada were surveyed for their current usage practices of bridge management systems in the decision-making process to strengthen existing bridges. The following questions were asked:

Question 11—Was the need to increase capacity identified by your bridge management system for any of the projects you have listed on the bridge data sheets?

The answers are tabulated in Table 8.

Question 12—Does your bridge management system have decision matrices for increasing capacity?

The answers are tabulated in Table 9.

Analysis of the Answers TO Q-11 AND Q-12

Only four states indicated that PONTIS has a decision matrix for identifying bridge strengthening needs. One state indicated that BRIDGIT had a decision matrix for identifying bridge strengthening needs. However, 32 states answered in the negative to this question, which reflects the fact that most of the agencies have only recently acquired bridge management systems and that they are currently adding data to the systems but are not yet familiar with all the decision matrices available.

DECISION MATRIX

In the absence of necessary data in existing bridge management systems to make decisions on the most suitable methods to increase live load capacity on a particular bridge, an example matrix in Table 10 has been developed for informational purposes. Each structure type is assessed against the common methods of increasing capacity, in the following categories:

- Suitability of the method—Yes (Y) or No (N)
- First cost—High (H) Medium (M) or Low (L)
- Traffic disruption—High (H) Medium (M) or Low (L).

The entries are subjective and may not apply directly to any state, but should enable relative evaluations to be made to assist in decision making.

TABLE 8

USE OF BRIDGE MANAGEMENT SYSTEMS IN DECISION MAKING TO STRENGTHEN EXISTING BRIDGES

Jurisdiction	Yes	No
U.S. States	1	21
FHWA	1	–
Canadian Provinces	2	2
PWC	1	–
U.S. Consultants	0	7
Canadian Consultants	0	3

TABLE 9

USE OF DECISION-MAKING MATRICES IN BRIDGE MANAGEMENT SYSTEMS FOR INCREASING CAPACITY

Jurisdiction	Yes	No
U.S. States	5	32
FHWA	–	1
Canadian Provinces	1	3
PWC	–	1
U.S. Consultants	0	8
Canadian Consultants	1	2

TABLE 10

COMPARISON OF STRENGTHENING METHODS BY STRUCTURE TYPE (EXAMPLE)

Structure: Material	Structure: Type	Method: Lightweight Deck	Composite Action	Transverse Stiffness	Strengthen Member	Post Tension	Develop Continuity	FRP Sheet Bonding	Add Members
Steel	Multi Girders								
	Method	Y	Y	Y	Y	Y	Y	N	Y
	Cost	H	L	H	M	M	H	-	H
	Traffic	H	H	L	L	L	M	-	H
	Deck Trusses								
	Method	Y	Y	Y	Y	Y	Y	N	Y
	Cost	H	L	H	M	M	H	-	M
	Traffic	H	H	L	L	L	M	-	L
	Through Trusses								
	Method	Y	N	N	Y	Y	Y	N	Y
	Cost	H	-	-	M	M	H	-	M
	Traffic	H	-	-	M	L	M	-	M
Reinforced Concrete	Slabs								
	Method	N	N	N	N	Y	N	Y	N
	Cost	-	-	-	-	M	-	H	-
	Traffic	-	-	-	-	L	-	L	-
	I and T Beams								
	Method	Y	Y	Y	Y	Y	Y	Y	Y
	Cost	H	H	H	M	M	H	H	H
	Traffic	H	H	M	M	L	H	L	H
Prestressed Concrete	Precast I Beams					Y	Y	Y	Y
	Method	Y	Y	Y	Y	M	H	H	H
	Cost	H	H	H	M	L	H	L	H
	Traffic	H	H	M	M				
	Precast Adjacent Boxes								
	Method	Y	Y	Y	Y	Y	Y	Y	N
	Cost	H	H	H	H	H	H	H	-
	Traffic	H	H	H	M	M	H	L	-
	CIP Slabs								
	Method	N	N	N	N	Y	N	Y	N
	Cost	-	-	-	-	M	-	H	-
	Traffic	-	-	-	-	L	-	L	-
	CIP Multi-Cell Boxes								
	Method	N	N	Y	Y	Y	N	Y	N
	Cost	-	-	H	H	M	-	H	-
	Traffic	-	-	M	L	L	-	L	-

TABLE 10 (Continued)

Structure		Method							
Material	Type	Lightweight Deck	Composite Action	Transverse Stiffness	Strengthen Member	Post Tension	Develop Continuity	FRP Sheet Bonding	Add Members
Prestressed Concrete	Segmental Box								
	Method	N	N	Y	Y	Y	N	Y	N
	Cost	-	-	H	H	M	-	H	-
	Traffic	-	-	H	M	L	-	L	-
Wood	Beam and Stringer								
	Method	Y	Y	Y	Y	Y	Y	Y	Y
	Cost	H	H	M	M	M	M	H	H
	Traffic	H	H	M	M	L	M	L	H
	Laminated Decks								
	Method	N	Y	Y	N	Y	N	N	N
	Cost	-	H	M	-	M	-	-	-
	Traffic	-	H	M	-	H	-	-	-
	Through Trusses								
	Method	Y	Y	N	Y	Y	Y	Y	Y
	Cost	H	H	-	M	M	H	H	M
	Traffic	H	H	-	M	L	M	M	M

Note: Suitability of the method: Yes (Y) or No (N); First cost: High (H), Medium (M) or Low (L); Traffic disruption: High (H), Medium (M) or Low (L).

CHAPTER SEVEN

EMERGING TECHNOLOGIES

BACKGROUND

In the survey questionnaire, agencies were asked to identify any innovative techniques they had recently developed for increasing the live load capacity of bridges. Seven agencies and three consultants listed new techniques, some of which have been discussed in earlier chapters. Two agencies, Florida and Alberta, indicated they were working with universities on the use of carbon fiber-reinforced plastics (CFRP) for concrete beam strengthening. Agencies were also asked if they were aware of new techniques or materials being researched or developed by others. Several agencies mentioned use of fiber-reinforced plastics (FRP); they also noted work in this area at the University of Arizona, the University of Alberta, Case Western University, and the Georgia Institute of Technology. Development of a lightweight aluminum orthotropic deck to be used as a replacement deck to reduce dead load was also mentioned.

The literature search revealed that improving member strength has received a great deal of coverage. Bonding of steel plates to achieve this was addressed in 27 papers, 20 of which were from the United Kingdom, but only two from North America. Bonding of FRP sheets was the technique of most interest, with over 60 papers covering the topic applied to bridge superstructure strengthening. Most papers dealt with reinforced concrete or prestressed concrete beams with carbon as the most common fiber and epoxy resin as the most common adhesive. This application has the most potential for future use in North America. Several papers addressed lightweight decks in different materials.

The two emerging technologies of most interest, from both the survey responses and the literature search, are the use of bonded FRP laminates and new lightweight decks; these methods are the focus of this chapter.

LIGHTWEIGHT DECKS

Aluminum Decks

The Reynolds Metal Company and their consultants have recently developed three lightweight aluminum deck systems. The first application is for redecking of the Corbin Bridge, a 91-m (300-ft) single-lane suspension bridge in Pennsylvania (*86*). By reducing the dead weight of the structure, the orthotropic aluminum deck will enable the live load capacity to be raised from 62 kN (7 tons) to 196 kN (22 tons). The deck replacement was completed in 1996.

The second project, let for bid in late 1996, is for deck replacement on the Little Buffalo Creek Bridge in Virginia. The reduced weight will allow the bridge to be widened from 6.1 m (20 ft) to 8.5 m (28 ft) without strengthening the substructure. The deck system, which has near isotropic properties, is shown in Figure 38. The deck is made to act compositely with the steel girders by the use of welded shear studs grouted inside the local deck voids (Figure 9, Detail A). This method of connection isolates the aluminum deck from the steel girders. The thin wearing surface consists of a 9.5-mm (0.375-in.) layer of polymer concrete.

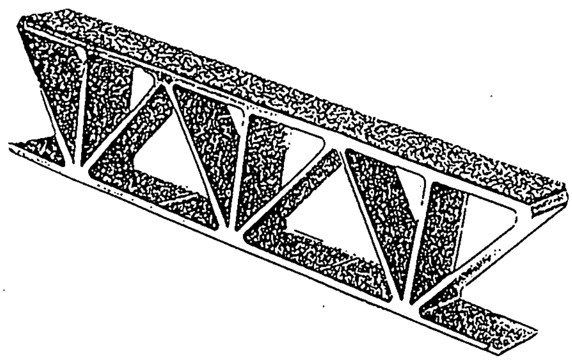


FIGURE 38 Aluminum deck system *(86)*.

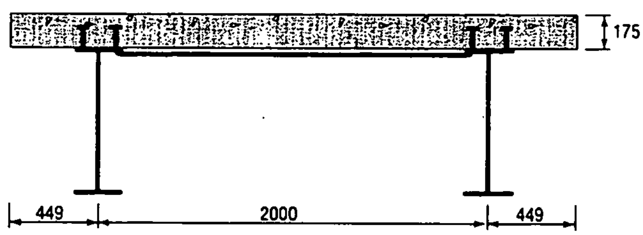


FIGURE 39 FRC deck slab model *(156)*.

Fiber-Reinforced Concrete Decks

Canada has developed a fiber-reinforced concrete (FRC) deck slab system using inexpensive polypropylene fibers with no reinforcing steel in the slab (*154, 155*). A full-scale model was tested in the laboratory. Figure 39 shows the cross-section of the model, with steel straps attached to the girder flanges to take the tie force generated by the arching action of the slab under a wheel load. The model was tested for fatigue as well as ultimate strength (*156*). Although the deck itself is not lightweight, the absence of reinforcing steel, with its cover requirements, enables a slab thickness of 175 mm (6.9 in.) to be used instead of the usual 225 mm (8.9 in.) in Ontario. In addition, the waterproofing and wearing surface normally used in Canada to protect against salt attack is not required because of corrosion-resistance qualities of the FRC slab.

The system was first used as a demonstration project on the Salmon River Bridge in Nova Scotia (*157*), which was opened in 1995. The bridge consists of two simple spans of 31.2 m (102 ft) each, one span with the FRC slab and the other with a conventional slab, so the performances could be compared. Figure 40 shows the placing of the FRC deck slab, which is supported on steel girders spaced at 2.7 m (8.9 ft). The FRC deck is performing satisfactorily, and the system is now being tried on projects in Ontario and Alberta. Design of FRC deck slabs is covered in the draft of the Canadian Highway Bridge Design Code presently being written, with an expected publication date in 1998. Based on deck prices from the Salmon River Bridge, \$63,000 (Can) for the conventional deck and \$66,850 (Can) for the FRC deck, the steel-free deck could be very competitive based on life-cycle costing. Although developed for new construction, the technology is equally applicable for deck replacement to increase live load capacity on selected projects. A variation on the FRC deck slab system is presently being researched, whereby the steel straps are replaced by CFRP prestressing rods to create a bridge deck devoid of steel (*158*).

The Ontario project is for a deck replacement on an existing bridge in Chatham (*159*). Reinforcement for negative moments was required at the deck cantilevers, and CFRP grids were used to maintain a steel-free deck. The bridge was re-opened to traffic in November 1996, and will be monitored on a long-term basis by MTO.

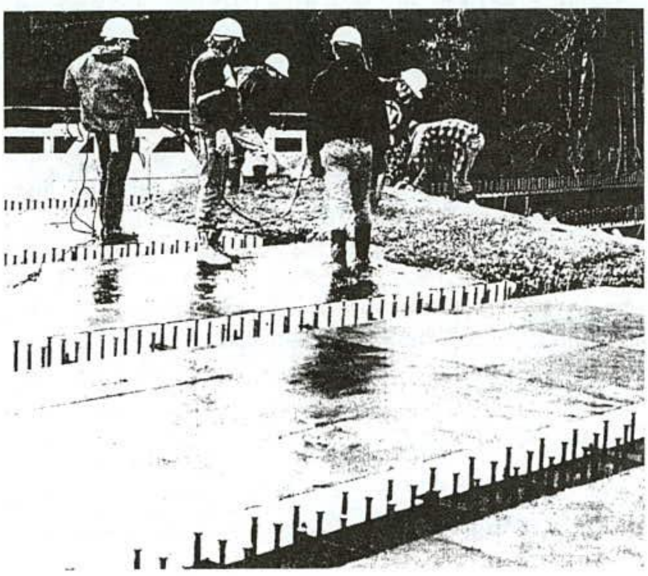


FIGURE 40 Placing FRC deck, Salmon River Bridge (*157*).

Fiber-Reinforced Composite Decks

Fiber-reinforced polymer composite materials were used for a complete deck system on a bridge in China and the Aberfeldy footbridge in Scotland. The University of California, San Diego is investigating the possibility of using such a deck type as a replacement bridge deck on existing steel or concrete girders (*160*). The topic has also been the subject of research sponsored by FHWA (*161*). The deck would be significantly lighter than other decks and, if cost-effective, could be attractive as a means of increasing live load capacity. With the high strength-to-weight ratios of these materials and their corrosion resistance, the decks could become economical based on life-cycle costing. To date, several hollow deck configurations have been investigated for ease of manufacture and have been load tested in the laboratory.

MEMBER STRENGTHENING

Strengthening of two segmental concrete box girder bridges in North America using straight external cables was described in chapter 4. The use of draped external cables may be more effective for future strengthening needs. Draped external cables have been used in a major new expressway construction in Houston. An extensive research and testing program was carried out at the University of Texas at Austin on a three-span one-quarter scale model of a post-tensioned segmental box girder bridge. Bonding of external cables at the deviators was tested (*162*), followed by tests for ultimate strength. The model was repaired and supplementary internal cables were added. The effects of improved bonding of the external cables and grouting of the supplementary cables on the ultimate strength and ductility were examined (*78,163*). The work will be of interest for strengthening existing segmental box girder bridges.

A new method of strengthening to accommodate increased live loading for simple-span steel girder bridges has been investigated at Iowa State University (*164*), using partial end restraint. A one-third scale model was tested with varying degrees of end restraint. The most effective technique was to restrain both the bottom flange and the web. Within the restriction of practical-sized restraint mechanisms, percentage reductions in mid-span strains and deflections ranged from 12 to 30, and the system appears feasible.

Ten years ago, the strengthening of concrete bridges by epoxy bonding steel plates appeared promising. Although popular in the United Kingdom, continental Europe and Japan, the method has not been used in North America. For reasons outlined in chapter 1, it now appears unlikely that it will be used in North America, and there is now greater interest in bonding FRP plates or sheets. The initial interest in North America in applying FRP to bridges was to increase shear strength and ductility of concrete columns in seismic areas. Research at the University of California, San Diego first tested glass fiber/epoxy jacketing (*165*), but has recently adopted carbon fibers (*127*) and found that such wrapping by automatic winding machine was faster than the usual steel jacketing system. For seismic retrofit applications, the carbon fibers would be oriented circumferentially. The technology could be applied to column strengthening for increased live load capacity in flexure by orienting the fibers both vertically and circumferentially. For smaller projects, FRP sheets and epoxy can be hand applied, rather than by wrapping machine. Figure 41 shows a hand application of glass FRP on bridge columns primarily for improved durability. The technology for column strengthening using FRP wrapping can be considered sufficiently advanced for selected application. As this synthesis is primarily concerned with superstructure strengthening, substructure applications will not be considered further.

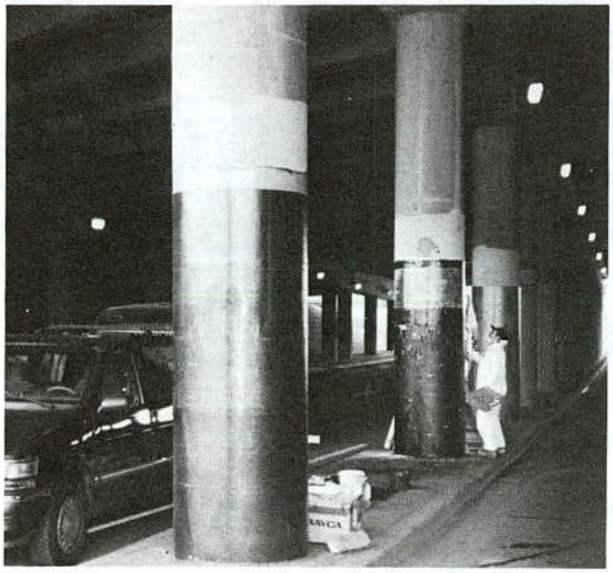


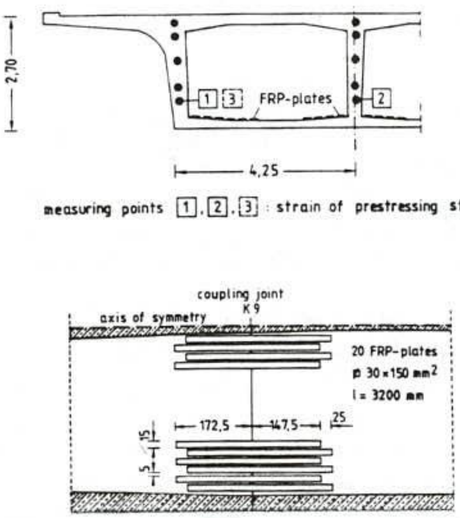
FIGURE 41 Glass RFP sheets applied to columns.

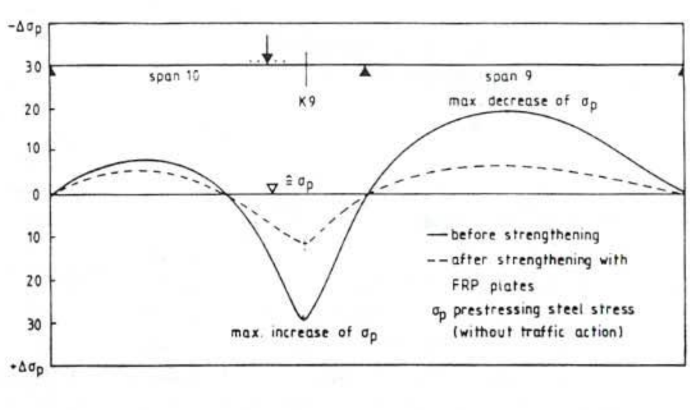
The first use of bonded FRP plates for strengthening bridge superstructures took place in Germany in 1987 on the Kattenbusch Bridge (*166*), an 11-span continuous post-tensioned box girder bridge. Wide cracks occurred at construction joints as a result of temperature gradient effects and the coupling of cables at these points. The increase in dynamic steel stresses at these cracks put the fatigue strength in jeopardy, and the structure required reinforcing at these points. The joints were strengthened, eight with steel plates and two with glass fiber-reinforced plastic (GFRP) plates in an epoxy-resin matrix, bonded to the bottom slab inside the box (Figure 42). The bridge was load tested before and after strengthening, and the substantial reduction in measured stress change in the lowest prestress tendon is shown in Figure 42. The strengthening effect of the steel plates and the GFRP plates was identical and the bridge is now functioning satisfactorily.

The first use of carbon laminates (CFRP) was to repair the Ibach Bridge in Switzerland (*167,168*) in 1991. This concrete box girder bridge had a prestressing tendon accidentally damaged, and heavy permit loads were banned until the original strength was restored. This was carried out by bonding three CFRP sheets, each 150-mm (5.9-in.) wide and 5-m (16.4-ft) long, with one 2-mm (0.079-in.) thick and the other two 1.75-mm (0.069-in.) thick. The bridge was load tested with an 840-kN (92-ton) truck and responded satisfactorily. The wood crossbeams of a 185-year-old bridge near Sins in Switzerland were subsequently strengthened with CFRP sheets without detracting from the historic design and appearance (*167*).

The application of bonded FRP laminates to bridge superstructure strengthening is just starting on a trial basis in North America. The first full-scale use of CFRP sheets for a bridge rehabilitation project in the United States was on the Foulk Road Bridge #26 in Wilmington, Delaware (*169*). The prestressed adjacent box beams developed longitudinal cracking on the bottom soffit because of insufficient transverse reinforcements on the bottom slab. In a joint project between the University of Delaware and the Delaware Department of Transportation, a field demonstration was undertaken using CFRP sheets, supplied by the Tonen Corporation, bonded to six of the soffits. The Forca tow sheets had the fibers oriented transversely to the beam length. The sheets had tensile strengths between 2942 MPa (427 ksi) and 3480 MPa (505 ksi). The work was completed in October 1994, and monitoring will continue for several years to evaluate effectiveness and durability.

In 1995, the Florida DOT repaired damaged concrete beams to restore them to their original capacity with CFRP sheets. In Ontario, a pretensioned concrete beam was hit by a high vehicle, which broke away concrete, severed three strands and damaged four more out of a total of twelve. The severed tendons were coupled, the beam grouted, and a CFRP sheet applied to the soffit to restore the original strength and durability of the beam. This experimental project was completed in 1995 and is now being monitored.





Measured reduction of stress change $\Delta\sigma_p$ in the lowest tendon by external strengthening

FIGURE 42 Strengthening Kattenbusch Bridge with CFRP plates *(165)*.

In another field demonstration, CFRP plates were applied to a concrete bridge in Butler County, Ohio, using a vacuum bag system (*170*). The purpose was to assess the durability of the material under severe environmental conditions. Finally, as part of an ongoing investigation of FRP applications at the Georgia Institute of Technology, two bridge decks located in Gilmer County, Georgia, will be repaired and strengthened to test the effectiveness of the techniques used (*171*).

Although trial installations are few, the interest in FRP strengthening methods is high in North America. Of the more than 60 relevant papers identified on this topic, the split was roughly one-third each between Europe, Canada, and the United States. The outcome of much of the current research will go a long way in determining how soon and how widely the technique will be used on a production basis. For this reason, the following section is devoted to recent research and development work in this area.

RESEARCH ON BONDED FRP LAMINATES

The wide range of materials available for bonded FRP laminates to strengthen bridge superstructures includes the many types of fibers and polymer resins, as well as adhesives. The field has narrowed in recent years, and most research is investigating glass, carbon, or aramid fibers, with carbon being the most common. Typical mechanical properties of these three laminates for continuous fibers laid in the direction of stress are shown in Table 11 (*172*). Even within each material combination, the range of properties is wide and can vary further for fibers laid in different directions and arranged in mat form. When selecting the most suitable material for the laminates, other criteria must be considered. Table 12 (*173*) shows a qualitative comparison of several criteria for glass, aramid, and carbon composite sheets, including relative price.

Europe

Since 1984, the Swiss Federal Laboratories for Material Testing and Research (EMPA) have been leaders in advanced composite materials research, and particularly in FRP laminates for concrete superstructure strengthening (*167*). Early concerns were the possible sudden tensile failure of the CFRP sheets and possible peeling at the end of the sheets from shear cracking. EMPA formulated a design rule that CFRP sheets should fail during yielding of the steel rebar, before failure of concrete in the compression zone. EMPA is now investigating the use of pretensioned CFRP sheets, stressed to 50 percent of the strength of the sheet, for improved performance compared to unstressed sheets. There are application difficulties, however, as the sheets must be mechanically stretched and held during bonding. The question of satisfactory anchorage of the ends of pretensioned sheets has to be solved, and it may be a year or two before pretensioned sheets are used in practice (*173*). Recommendations that are applicable to both unstressed and prestressed sheets have been made for preparation of the bonding surfaces to achieve optimum composite action, and for application of the adhesive and bonding of the sheets (*173*). In their concern for safety, EMPA has recommended that post-strengthening of a structure should not be more than 50 percent. Then, should an accidental failure of the strengthening system occur, a suitable residual factor of safety against collapse would already be in place (*174*). Sika AG, a Swiss manufacturer, has also provided guidelines on substrate preparation, field application of CFRP laminates, and characteristics of the epoxy-resin adhesive (*175*).

TABLE 11

TYPICAL MECHANICAL PROPERTIES FOR GRFP, AFRP, AND CFRP *(171)*

Uni-Directional Composite Material	Fiber Content (% by weight)	Longitudinal Tensile Modulus GPa (ksi)	Tensile Strength MPa (ksi)
Glass Fiber/Polyester	50–80	20–55	400–1800
GFRP laminate		(2900–7975)	(58–261)
Aramid/Epoxy	60–70	40–125	1000–1800
AFRP laminate		(5800–18125)	(145–261)
Carbon/Epxoy	65–75	120–250	1200–2250
CFRP laminate		(17400–36250)	(174–326)

TABLE 12

QUALITATIVE COMPARISON BETWEEN E-GLASS, ARAMID, AND CARBON FIBERS *(173)*

	Fiber Composite Sheets Made Of		
Criterion	Carbon Fibers	Aramid Fibers	E-glass Fibers
Tensile Strength	Very good	Very good	Very good
Compressive Strength	Very good	Inadequate	Good
Young's Modulus	Very good	Good	Adequate
Long-term Behavior	Very good	Good	Adequate
Fatigue Behavior	Excellent	Good	Adequate
Bulk Density	Good	Excellent	Adequate
Alkaline Resistance	Very good	Good	Inadequate
Relative Price	Very high	High	Moderate

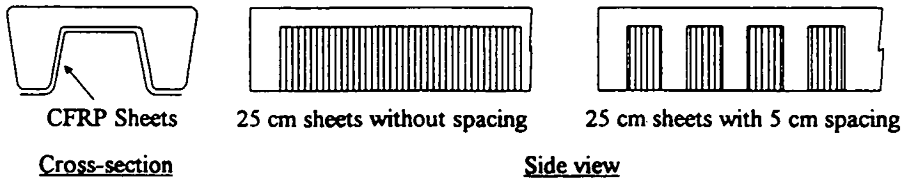


FIGURE 43 Upgrading shear capacity with CFRP sheets *(190)*.

In Greece, a reliability study of reinforced concrete beams strengthened with epoxy-bonded CFRP laminates has been performed, and tentative recommendations have been made on strength reduction factor values for ultimate strength design and evaluation purposes *(176)*. In France, CFRP plates bonded to the tension flange of reinforced concrete beams have been tested and analyzed for ultimate capacity. An iterative analytical model capable of simulating bond slip and material non-linearity has been developed *(177,178)*.

In May 1994, a 3-year project called ROBUST was launched in the United Kingdom to investigate the viability of using CFRP and GFRP materials as alternatives to the existing steel plate bonding used for strengthening reinforced or prestressed concrete bridges. ROBUST is a consortium of two universities and seven industrial partners, managed by L.G. Mouchel & Partners Ltd. *(179)*. It is investigating the technical, commercial, and economic viability and aims to develop guidelines for design and installation. Reinforced concrete beams with glass-fiber plates are being tested at the University of Surrey *(180)* using three different adhesives. At Oxford Brookes University, more than 30 beams are being tested using both glass and carbon fiber laminates *(181)*. The laminates are unstressed and are bonded using epoxy adhesives. Bonding of steel plates has been widely used in the United Kingdom. Much of the research for this method was conducted at the University of Sheffield, where testing is now being carried out on GFRP strengthening *(182)*. The university's review of the test data available from all sources on FRP strengthening recognizes the advantages of this method but cautions that there are many material and structural implications that are still unclear *(183)*.

Canada

The Advanced Composite Materials in Bridges and Structures (ACMBS) Network of Canada consists of a number of universities and agencies investigating and coordinating the use of advanced composite materials in bridges and structures. Research work has been conducted into the use of FRP laminates for bridge strengthening at eight universities. The main center for this technology is in Kingston, Ontario at the Royal Military College (RMC). Reinforced concrete beams strengthened with CFRP sheets have been tested, and an equivalent capacity concept has been proposed based on the load at which the tensile steel yields *(184,185)*. Tests on reinforced concrete slabs strengthened with CRFP and GRFP showed an increase in both stiffness and strength *(186)*. RMC and Queen's University have jointly investigated the use of prestressed CFRP sheets and found that prestressed sheets are slightly more effective at strengthening than unstressed sheets, and are most effective at reducing crack widths and delaying the onset of cracking *(187,188)*. The prestressing of sheets has been carried out in the laboratory, but effective field methods need further development.

Work at the University of Alberta has concentrated on upgrading the shear capacity of beams. Three precast reinforced concrete beams from a disused bridge were strengthened with CFRP sheets bonded to the web and tested for shear *(189)*. The results showed an increase in shear capacity of up to 73 percent, but work remains to be done on low-temperature applications and fatigue behavior. The second phase was to apply CFRP sheets to an actual rehabilitation project to assess the real-life performance *(190)*. Figure 43 shows the layout of the sheets with fibers perpendicular to the girder length. Six of the girders were reinforced continuously, whereas four girders had spaces left between sheets to allow visual monitoring of any crack propagation. To test if bonding is affected by bridge traffic, five girders were bonded with full traffic and five others with one lane closed. The project is now being monitored. Research at other universities includes:

- Carleton—Tested beams with CFRP sheets in combination with a high-density polypropylene grid to improve ductility *(191)*.
- Sherbrooke—Tested structural concrete strengthened with an aramid woven fiber/epoxy resin composite in a laboratory setting *(192)*.
- Laval—Tested concrete beams strengthened with GFRP plates, which showed ultimate capacity below theoretical value as a result of plate slippage *(193)*.
- Toronto—Compiled and analyzed a database including test results from 10 separate studies of FRP-strengthened beam performance *(194)*.
- Concordia—Conducted environmental testing of beams repaired with CFRP sheets by water immersion and hot and cold cycling *(195)*.
- British Columbia—Tested the use of a sprayed-on FRP with 8 percent chopped glass fibers as a thin coating to strengthen concrete beams *(196)*.

United States

Early tests at Lehigh University in 1991 were carried out on a series of concrete beams strengthened with glass, carbon, and aramid FRP materials (*197*). An analytical method, based on strain compatibility, was developed to predict strength and stiffness of such plated beams.

Research has been ongoing for several years at the University of Arizona, starting with an investigation of failure modes in reinforced concrete beams strengthened with FRP plates (*198*). The need for concrete surface preparation and an appropriate adhesive was emphasized. Tests were later carried out on one rectangular concrete beam and one T-beam, each strengthened by epoxy bonding a 6-mm (0.25-in.) thick glass FRP plate to the tension flange (*199*). The load carrying capacity was significantly increased, with most gain predicted for beams with low flexural reinforcement ratios. The flexural strength of concrete girders externally prestressed with epoxy-bonded FRP plates was also studied at Arizona (*200*). The girders have to be jacked upward and held during bonding, which makes it an unlikely method for strengthening in the field. Most recently, the high interfacial shear and normal stress concentrations at the ends of bonded FRP plates has been studied, and an analytical method has been presented to predict the distribution of stresses at these locations (*201*).

At the University of South Florida, the interest has been in strengthening composite steel beams by bonding CFRP plates to the bottom flange (*202,203*). The beams were first loaded past yield of the tension flange, then strengthened by bonding 2-mm (0.079-in.) or 5-mm (0.20-in.) laminates and testing to failure. Increases in ultimate strength ranged from 11 to 50 percent, but long-term durability studies need to be undertaken before the method is used in field applications.

Several studies have been carried out at the University of Delaware related to bridge superstructure strengthening. A series of reinforced concrete beams strengthened by CFRP tow sheets were tested with variations in the number of layers and fiber orientation. The increase in ultimate beam capacity ranged from 158 to 292 percent, with failure by tensile failure of the composite or shear failure of the concrete (*204*). Simple formulae were developed to predict the ultimate capacity of the strengthened beams. The nature of the bond between the composite plate and the concrete has been studied, and the influence of the surface preparation of the concrete, the concrete strength, and the adhesive type have been evaluated through laboratory tests (*205*). The use of CFRP plates to strengthen wood beams has also been investigated (*206*). Although the plates can be bonded effectively to the tensile face of wood beams to improve their overall flexural behavior, a number of issues remain to be addressed before design criteria can be developed. The University of Delaware is also conducting research on the rehabilitation of steel bridge girders through the application of ACMs.

Experimental studies on the feasibility of using CFRP materials in the repair of concrete bridges have been carried out at Florida Atlantic University (*207*). The contribution of the CFRP plate retrofit to the flexural resistance was evaluated for both solid and voided-slab bridge models. The flexural behavior of rectangular concrete beams strengthened with CFRP laminates has also been investigated (*208*). A significant reduction in deflections and crack width was observed, along with a substantial increase in ultimate flexural capacity.

In addition to the identified university research, considerable developmental work is being carried out by the various ACM industry companies. As much of this work could be proprietary, the results do not always appear in the usual technical press. The names and addresses of three American societies related to ACM are given as possible contacts for industry developments.

ASC (American Society for Composites)
P.O. Box 951597
University of California at Los Angeles
Los Angeles, CA 90095–1597
Tel: (310) 206–1840
Fax: (310) 206–4830

SAMPE (Society for Advanced Materials and Process Engineering)
P.O. Box 2459
Covina, CA 91722–8459
Tel: (818) 331–0616
Fax: (818) 332–8929

SPE (Society of Plastic Engineers)
14 Fairfield Drive
Brookfield, CT 06804
Tel: (203) 775–0471
Fax: (203) 775–8490

Seven societies and organizations contribute to ***FRP International***, a newsletter reporting on new developments. It is published through the University of Manitoba.

Department of Civil Engineering
Room 353A, Engineering Building
University of Manitoba
Winnipeg, Manitoba
Canada R3T 5V6
Tel: (204) 474–8506
Fax: (204) 261–5465

CHAPTER EIGHT

CONCLUSIONS

The agency responses to the survey questionnaire for this synthesis indicate that the methods used to increase live load capacity in the past 10 years are much the same as recorded in NCHRP Report 293 in 1987. Very few new techniques were reported, but there have been some applications of lightweight decks, external post-tensioning, and an interest in the use of advanced composite materials. It should be recognized that although the response to the survey questionnaire was good, a number of questions were often left unanswered. The numbers given for each strengthening method may not accurately reflect the numbers where increasing live load capacity was the main reason for strengthening. In many cases, an increase in capacity might result from carrying out rehabilitation because of deterioration rather than being the prime reason for doing the work. This is particularly the case for deck replacements.

Ideally, selection of the strengthening method should be based on life-cycle cost data. While first cost is the more usual criterion, very little cost data were provided in the responses. Although most agencies have operating bridge management systems, they were rarely used in the decision-making process to strengthen existing bridges, and very few systems have a decision matrix for this purpose. The reason for the present lack of use is the insufficiency of useful data in the systems.

Selection methods will only become efficient and cost-effective when bridge management systems contain the required data and become widely used for this purpose. Cost data, including lane-closure costs and particularly life-cycle costs, need to be collected so that they can be the basis for future method selection.

Although simple analytical methods are used for first calculation of live load capacity, roughly half the reporting agencies move to more sophisticated methods if the evaluation result is not satisfactory. This move generally results in a higher calculated capacity. The codes used for evaluation purposes are nearly all based on working stress design (WSD) or load factor design (LFD) methods. The use of load and resistance factor (LRFD) methods will generally result in a higher capacity. Such a method has been available since 1989 in the AASHTO *Guide Specification for Strength Evaluation of Existing Steel and Concrete Bridges*, but it appears to be little used. In spite of the advantages of the LRFD approach, it is unlikely that the approach will be widely used for evaluation purposes until it is part of a mandated code or specification. The NCHRP Project 12-46, Manual for Condition Evaluation and Load Rating of Highway Bridges Using Load and Resistance Factor Philosophy, represents an important step in this direction.

The emerging technology with the most potential for use in bridge strengthening is structural use of advanced composite materials. In North America there is now much research on the use of fiber-reinforced plastic (FRP) sheets bonded to beams to increase load capacity. The array of FRP sheet materials and adhesives is wide, but most interest seems to be on carbon FRP sheets and epoxy resins. FRP sheets have been applied to concrete, steel, and wood structures, but the most widespread use will be for reinforced and prestressed concrete structures. Several demonstration projects have been undertaken in North America, and the number is likely to increase as further investigation is required in such areas as long-term durability, ductility, and low-temperature behavior before FRP strengthening is widely adopted. In addition, the cost of FRP materials is high compared to traditional strengthening materials, and life-cycle costing may be required to justify their use. Data need to be collected on performance and costs.

FRP materials represent a design challenge because of the large number of variables involved. It is important that the industry settle on standard products and systems so that design methods can be developed similar to those available for traditional materials. These design methods may be most useful if developed in an LRFD format.

LRFD methods for evaluation and rating are widely used in Canada as they have been part of the design codes that have been used for many years. The benefits of the LRFD approach are well-established, and if bridge strengthening is to be performed most efficiently, this evaluation approach would have to be incorporated into all the accepted codes and standards in North America. The LRFD Bridge Design Specification has been available since 1994 as an AASHTO standard. Benefit may be gained if it becomes more widely adopted as the design standard and if a compatible evaluation standard is developed in the near future. The next logical step would be to include FRP materials in these standards.

REFERENCES

1. Klaiber, F.W., K.F. Dunker, T.J. Wipf, and W.W. Sanders, Jr., *NCHRP Report 293: Methods of Strengthening Existing Highway Bridges*, Transportation Research Board, National Research Council, Washington, D.C. (1987) 114 pp.
2. Henwood, A.M. and K.J. O'Connell, "Plate Bonding: A User's Guide Note," in *Bridge Modifications*, B. Pritchard, ed., Thomas Telford Services Ltd., London (1995) pp. 204–216.
3. Haynes, M. "Plate Bonding," *Construction Repair*, Palladian Publications Ltd., London (Jan/Feb 1994) pp. 43–44.
4. Jones, R., R.N. Swamy, and A. Charif, "Plate Separation and Anchorage of Reinforced Concrete Beams Strengthened by Epoxy-Bonded Steel Plates,"*The Structural Engineer,* London (March 1988) pp. 85–94.
5. Raoof, M. and S. Zhang, "Peeling Failure of Reinforced Concrete Beams with Externally Bonded Steel Plates,"in *Bridge Management 3*, J.E. Harding, ed., E.& F.N. Spon, London (1996) pp. 420–428.
6. Hutchinson, A.R., "Strengthening of the Quinton Bridges with Externally Bonded Steel Plate Reinforcement,"in *Bridge Management 3*, J.E. Harding, ed., E. & F.N. Spon, London (1996) pp. 743–750.
7. Garden, H.N., L.C. Holloway, A.M. Thorne, and G.A.R. Parke, "A Parameter Study of the Strengthening of Reinforced Concrete Beams with Bonded Composites," in *Bridge Management 3*, J.E. Harding, ed., E. & F.N. Spon, London (1996) pp. 400–408.
8. Hawk, H., "Development of Life-Cycle Activity Profiles in BRIDGIT Bridge Management System,"*Proceedings of the Fourth International Bridge Engineering Conference*, Transportation Research Board, Washington, D.C., Vol. 1 (1995) pp. 209–219.
9. Bakht, B. and L.G. Jaegar, "Bridge Testing—A Surprise Every Time," *Journal of Structural Engineering*, ASCE, Vol. 116, No. 5 (1990) pp. 1370–1383.
10. Burdette, F.G., and D.W. Goodpasture, *NCHRP Report 306: Correlation of Bridge Load Capacity Estimates with Test Data*, Transportation Research Board, National Research Council, Washington, D.C. (1988) 75 pp.
11. Lichtenstein, A.G., *Bridge Rating Through Nondestructive Load Testing,* NCHRP Project 12-28(13)A, Completed—Publication decision pending, Transportation Research Board, National Research Council, Washington, D.C. (1996).
12. *Extending the Lifespan of Structures*, Report of the IABSE Symposium, San Francisco, IABSE, Zurich, Switzerland, Vol. 73 (1995) 1,612 pp.
13. "Non-Metallic (FRP) Reinforcement for Concrete Structures," *Proceedings of the Second International RILEM Symposium*, L. Taerwe, ed., E. & F.N. Spon, London (1995) 714 pp.
14. *Advanced Composite Materials in Bridges and Structures*, Proceedings of the 2nd International Conference, Montreal, M.M. El-Badry, ed., CSCE, Montreal, Canada (1996) 1027 pp.
15. "Fourth International Bridge Engineering Conference," San Francisco, California, *Conference Proceedings 7*, Transportation Research Board, Washington, D.C. (1995).
16. American Association of State Highway and Transportation Officials, *Manual for Condition Inspection of Bridges*, AASHTO, Washington, D.C. (1988).
17. Government, *National Bridge Inspection Standards*, Code of Federal Regulations, Title 23, Part 650, Subpart C (Oct. 1988).
18. Federal Highway Administration, U.S. Department of Transportation, *Technical Advisory—Revisions to the National Bridge Inspection Standards (NBIS),* T5140.21, Washington, D.C. (Oct. 1991).
19. American Association of State Highway and Transportation Officials, *Manual for Bridge Maintenance*, AASHTO, Washington, D.C. (1988).
20. Federal Highway Administration, U.S. Department of Transportation, *Bridge Inspector's Manual for Movable Bridges*, Washington, D.C. (1977).
21. Federal Highway Administration, U.S. Department of Transportation, *Inspection of Fracture Critical Bridge Members*, Washington, D.C. (1986).
22. Federal Highway Administration, U.S. Department of Transportation, *Culvert Inspection Manual,* Washington, D.C. (1986).
23. Federal Highway Administration, U.S. Department of Transportation, *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*, Washington, D.C. (Dec. 1988).
24. Federal Highway Administration, U.S. Department of Transportation, *Bridge Inspectors Training Manual 90,* Washington, D.C. (1990).
25. Federal Highway Administration, U.S. Department of Transportation, *PONTIS Users' Manual,* Washington, D.C. (Dec. 1991).
26. Ministry of Transportation, *Ontario Structure Inspection Manual,* Toronto, Ontario (1989).
27. Federal Highway Administration, U.S. Department of Transportation, *Non-destructive Testing Methods for Steel Bridges*, Washington, D.C. (1986).
28. Federal Highway Administration, U.S. Department of Transportation, *Technical Advisory—Evaluating Scour at Bridges*, Washington, D.C. (1991).
29. Federal Highway Administration, U.S. Department of Transportation, *Underwater Inspection of Bridges*, Washington, D.C. (1989).
30. Ministry of Transportation, *Ontario Highway Bridge Design Code,* Third Edition, Toronto, Ontario (1991).
31. American Association of State Highway and Transportation Officials, *Standard Specification for Movable Highway Bridges,* AASHTO, Washington, D.C. (1988).
32. American Association of State Highway and Transportation Officials, *Guide Specification for Strength*

Evaluation of Existing Steel and Concrete Bridges, AASHTO, Washington, D.C. (1989).
33. American Association of State Highway and Transportation Officials, *Guide Specification for Fatigue Evaluation of Existing Steel Bridges*, AASHTO, Washington, D.C. (1990).
34. American Association of State Highway and Transportation Officials, *Manual for Condition Evaluation of Bridges,* AASHTO, Washington, D.C. (1994).
35. American Association of State Highway and Transportation Officials, *LRFD Bridge Design Specifications, S.I. First Edition,* AASHTO, Washington, D.C. (1994), with annual updated interim specifications.
36. American Association of State Highway and Transportation Officials, *Standard Specification for Highway Bridges,* AASHTO, Washington, D.C. (1996), with annual updated interim specifications.
37. Imbsen, R.,W. Liu, R. Schamber and R. Nutt, *NCHRP Report 292: Strength Evaluation of Existing Concrete Bridges*, Transportation Research Board, National Research Council, Washington, D.C. (June 1987).
38. Moses, F. and D. Verma, *NCHRP Report 301: Load Capacity Evaluation of Existing Bridges,* Transportation Research Board, National Research Council, Washington, D.C. (December 1987).
39. Daniels, J.H., W. Kim and J. Wilson, *NCHRP Report 319: Recommended Guidelines for Redundancy Design and Rating of Two-Girder Steel Bridges,* Transportation Research Board, National Research Council, Washington, D.C. (October 1989).
40. Galambos, T., R. Leon, C. French, M. Barker and B. Dishongh, *NCHRP Report 352: Inelastic Rating Procedures for Steel Beam and Girder Bridges,* Transportation Research Board, National Research Council, Washington, D.C. (1993).
41. *Existing Bridge Evaluation,* CAN-CSA-S6, Supplement No. 1, Rexdale, Ontario (1990).
42. Department of Transport, *Technical Requirements for the Assessment and Strengthening Programme of Highway Structures, Stage 2—Modern Short Span Bridges,* BD 46/92, London (1992).
43. Department of Transport, *Technical Requirements for the Assessment and Strengthening Programme of Highway Structures, Stage 3—Long Span Bridges,* BD 50/92, London (1992).
44. Nowak, A.S. and H.N. Grouni, *Calibration of the Ontario Highway Bridge Design Code,* 1991 edition; *Canadian Journal of Civil Engineering,* Vol. 21, No. 1, pp. 25–31 (1994).
45. Nowak, A.S., *Research Report UMCE 89-14, Calibration of the Ontario Highway Bridge Design Code—Phase 2,* Department of Civil Engineering, University of Michigan, Ann Arbor (October 1989).
46. Nowak, A.S., *Research Report UMCE 1990-1, Load and Resistance Factors for the Ontario Highway Bridge Design Code,* Department of Civil Engineering, University of Michigan, Ann Arbor (1990).
47. Nowak, A.S., *Calibration of the LRFD Code,* draft report from NCHRP Project 12-33, Transportation Research Board, National Research Council, Washington, D.C.
48. Agarwal, A. and V. Jovanovic, *Impact on the Highway Infrastructure of the Existing and Alternate Vehicle Configurations and Weight Limits, Section 4, Impact on Bridges*, Ministry of Transportation, Ontario, Downsview (1997).
49. Cowherd, D.C. and G.H. Degler, *Evaluation of Long-span Corrugated Metal Structures,* Bowser-Morner Associates, Inc., Ohio Department of Transportation and Federal Highway Administration, U.S. Department of Transportation, Washington, D.C. (1986).
50. Abdel-Sayed, G., B. Bakht, and L. Jaeger, "Distress, Monitoring, and Repairs," Chapter 10 in *Soil-Steel Bridges,* McGraw-Hill, New York (1994), pp. 291–335.
51. Fisher, J.W., J. Jin, D. Wagner and B. Yen, *NCHRP Report 336: Distortion Induced Cracking of Steel Bridges*, Transportation Research Board, National Research Council, Washington, D.C. (December 1990).
52. Ministry of Transportation, *Structural Financial Analysis Manual,* Toronto, Ontario (March 1990).
53. Thompson, P.D. and M.J. Markow, *NCHRP Synthesis of Highway Practice 227: Collecting and Managing Cost Data for Bridge Management Systems,* Transportation Research Board, National Research Council, Washington, D.C. (1996).
54. Ministry of Culture and Communications, *Ontario Heritage Bridge Program,* Toronto, Ontario (1991).
55. Plowden, D., *Bridges, The Spans of North America,* W.W. Norton & Company, New York (1984).
56. Issa, M.A., A.A. Yousif, M.A. Issa, I.I. Kaspar and S.Y. Khayyat, "Field Performance of Full Depth Precast Concrete Panels in Bridge Deck Reconstruction," *PCI Journal,* Precast/Prestressed Concrete Institute, Vol. 40, No. 3 (1995) pp. 82–107.
57. Issa, M.A., A.T. Idriss, I.I. Kaspar and S.Y. Khayyat, "Full Depth Precast and Precast, Prestressed Concrete Bridge Deck Panels," *PCI Journal,* Precast/Prestressed Concrete Institute, Vol. 40, No. 1 (1995) pp. 59–81.
58. Panak, J.J., "Bridge Strengthening, Reconstruction, and Rehabilitation," *Proceedings of the 7th Annual International Bridge Conference,* Engineers' Society of Western Pennsylvania, Pittsburgh (1990) pp. 44–50.
59. *Structural Lightweight Aggregate Concrete Performance,* Report No. ACI SP-136, T.A. Holm, ed., American Concrete Institute, Detroit (1992) 430 pp.
60. Mays, G.C. and R.A. Barnes, "The Performance of Lightweight Aggregate Concrete Structures in Service," *The Structural Engineer,* U.K. (October 1991) pp. 351–360.
61. Bettigole, R.A., "Exodermic Deck Boosts Bridge Load Rating," *Roads and Bridges,* Vol. 31, No. 8 (1993) pp. 44–45.
62. Wolchuk, R., "Orthotropic Redecking of Bridges on the North American Continent," *Structural Engineering International,* IABSE, Zurich, Switzerland, Vol. 2, No. 2 (1992) pp. 125–130.
63. Shimada, S., K. Yamada, H. Aoki, and I. Fukushima, "Structural Modifications of the Two-Hinged Arch Semimaru Bridge," in *Bridge Management 3,* J.E.

Harding, ed., E. & F.N. Spon, London (1996) pp. 279–286.

64. Kuroda, M. and M. Shitara, "Reinforcement Works on the Showa Bridge, a Stiffened Steel Arch Bridge," in *Developments in Short and Medium Span Bridge Engineering '94*, A.A. Mufti, ed., CSCE, Montreal, Canada (1994) pp. 803–813.
65. Azizinamini, A., S. Kathol and M. Beaucham, "Effect of Cross Frames on Behavior of Steel Girder Bridges," *Proceedings of the Fourth International Bridge Engineering Conference,* Transportation Research Board, National Research Council, Washington, D.C., Vol. 1 (1995) pp. 117–124.
66. Leon, R.T., D.O. Beltaos, and R.T. Seavy, *Retrofit of Wood Bridges, Final Report*, Report No. MN/RC-94/16, Minnesota Department of Transportation, St. Paul (1993) 82 pp.
67. Taylor, R.J. and F.J. Keenan, *Wood Highway Bridges*, Canadian Wood Council, Ottawa, Canada (1992) 193 pp.
68. Ulyatt, N.B., "Case Study—Rehabilitation of the St. James Bridge (Northbound) in the City of Winnipeg," *Proceedings of the 1989 Annual Conference of RTAC*, Transportation Association of Canada, Ottawa (1989) pp. A19-A38.
69. Dunker, K.F., F.W. Klaiber, and W.W. Sanders Jr., "Strengthening of Steel Stringer Bridges by Selective Stiffening," in *Transportation Research Record 1118*, Transportation Research Board, National Research Council, Washington, D.C. (1987) pp. 43–48.
70. Oehlers, D.J. and A. Ahmed, "Upgrading Reinforced Concrete Beams by Bolting Steel Side Plates," in *Bridge Management 3*, J.E. Harding, ed., E. & F.N. Spon, London (1996) pp. 412–419.
71. Mosher, T.A., "The Evaluation and Rehabilitation of Mill Village Bridge and Shelburne Bridge," in *Developments in Short and Medium Span Bridge Engineering '94*, A.A. Mufti, ed., CSCE, Montreal, Canada (1994) pp. 755–766.
72. Channon, J. and R.J. Ramsay, "Assessment and Strengthening of Bridges in Alberta," *Proceedings of the 1988 Annual Conference of RTAC*, Transportation Association of Canada, Ottawa (1988) pp. B3-B22.
73. Ramsay, R.J., "Evaluation and Strengthening of Concrete T-Girder Bridges for Shear," in *Developments in Short and Medium Span Bridge Engineering '90*, B. Bakht, ed., CSCE, Montreal, Canada (1990) pp. 381–391.
74. Chai, Y.H., M.J.N. Priestley, and F. Seible, "Seismic Retrofit of Bridge Columns by Steel Jacketing," in *Transportation Research Record 1290*, Transportation Research Board, National Research Council, Washington, D.C., Vol. 2 (1991) pp. 95–103.
75. Nilsson, I., "Crack-Free Surround Concrete Repairs of the Oland Bridge Piers," in *Extending the Lifespan of Structures*, IABSE Symposium Report, IABSE, Zurich, Switzerland, Vol. 73 (1995) pp. 89–94.
76. Troitsky, M.S., *Prestressed Steel Bridges—Theory and Design*, Van Nostrand Reinhold Company, New York (1990).
77. *External Prestressing of Bridges*, A. Naaman, ed., ACI-SP-120, American Concrete Institute, Detroit (1990).
78. Hindi, A.N.A., M.E. Kreger, and J.E. Breen, *Enhancing the Strength and Ductility of Post-Tensioned Segmental Box-Girder Bridges*, Research Report 1209-2F, Center for Transportation Research, University of Texas at Austin (1991) 352 pp.
79. Godart, B., "Strengthening Prestressed Concrete Box Girder Bridges: The French Experience," *Structural Engineering International*, IASBSE, Zurich, Switzerland, Vol. 5, No. 2 (1995) pp. 81–84.
80. Straninger, W. and M. Wicke, "Bridge Strengthening with Additional Prestressing," *Structural Engineering International*, IABSE, Zurich, Switzerland, Vol. 5, No. 2 (1995) pp. 78–80.
81. Falkner, H., Z. Huang, and M. Teutsch, "Strengthening Bridges with Unbonded Prestressing," *Structural Engineering International*, IABSE, Zurich, Switzerland, Vol. 5, No. 2 (1995) pp. 76–77.
82. Petrangeli, M.P., "External Prestressing for Bridge Rehabilitation in Italy," *Proceedings of the Fourth International Bridge Engineering Conference*, Transportation Research Board, National Research Council, Washington, D.C., Vol. 2 (1995) pp. 213–218.
83. Zahn, F.A. and J.M. Voumard, "The Use of External Post-Tensioning in Bridge Rehabilitation," *Proceedings of the 3rd International Workshop on Bridge Rehabilitation*, Ernst & Sohn, Berlin (1992) pp. 471–480.
84. *NCHRP Report 322: Design of Precast Prestressed Bridge Girders Made Continuous*, Transportation Research Board, National Research Council, Washington, D.C. (1989) 97 pp.
85. Wall, M.A., G.D. Gardner, and C.D. Posner, "Strengthening of Langleybury Lane Bridge, Hertfordshire," in *Bridge Management 3*, J.E. Harding, ed., E. & F.N. Spon, London (1996) pp. 93–100.
86. Ahlskog, J.J., "Aluminum Bridge Decks—A Viable Economic Alternative," *Proceedings of the 13th Annual International Bridge Conference*, Engineers' Society of Western Pennsylvania, Pittsburgh (1996) 9 pp.
87. Murray, M.J., "Friarton Bridge Strengthening," in *Bridge Management 3*, J.E. Harding, ed., E. & F.N. Spon, London (1996) pp. 79–85.
88. Farago, B., "Rehabilitation of Steel Truss Bridges in Ontario," in *Bridge Evaluation, Repair and Rehabilitation*, A.S. Nowak, ed., Kluwer Academic Publishers, Norwell, MA (1990) pp. 555–566.
89. Flint, A.R., "Strengthening and Refurbishment of the Severn Crossing," *Proceedings of the Institution of Civil Engineers, Civil Engineering*, Vol. 92, No. 2, London (May 1992) pp. 57–65.
90. *Inquiry into the basis of design and method of erection of steel box girder bridges*, Final Report, HMSO, London (1975).
91. *Design analysis doubts strength of recent bridges*, ENR, McGraw-Hill, New York (May 28, 1987) pp. 22–25.
92. Chatterjee, S., "Strengthening and Refurbishment of Severn Crossing Part 1: Introduction," *Proceedings of the*

Institution of Civil Engineers: Structures and Buildings, Vol. 94, London (February1992) pp. 1–5.

93. Flint, A.R., "Strengthening and Refurbishment of Severn Crossing Part 2: Design," *Proceedings of the Institution of Civil Engineers: Structures and Buildings*, Vol. 94, London (February 1992) pp. 7–22.
94. Evans, J.E., "Strengthening and Refurbishment of Severn Crossing Part 3: Construction," *Proceedings of the Institution of Civil Engineers: Structures and Buildings*, Vol. 94, London (February 1992) pp. 23–36.
95. Cuninghame, J.R. and C. Beales, "Strengthening and Refurbishment of Severn Crossing Part 4: TRRL research," *Proceedings of the Institution of Civil Engineers: Structures and Buildings*, Vol. 94, London (February 1992) pp. 37–49.
96. Klaiber, F.W., K.F. Dunker, and S.M. Planck, "Strengthening of a Three-Span Continuous Bridge by Post-tensioning," in *Bridge Evaluation, Repair and Rehabilitation*, A.S. Nowak, ed., Kluwer Academic Publishers, Norwell, MA (1990) pp. 525–538.
97. Klaiber, F.W., T.J. Wipf, F.S. Fanous, and H. El-Arabaty, "Strengthening of Continuous Span Steel Stringer Bridges," in *Developments in Short and Medium Span Bridge Engineering '94*, A.A. Mufti, ed., CSCE, Montreal, Canada (1994) pp. 1247–1258.
98. Wipf, T.J., F.W. Klaiber, F.S. Fanous, and H. El-Arabaty, "Strengthening of Continuous-Span Composite Steel-Stringer Bridges," *Proceedings of the Fourth International Bridge Engineering Conference*, Transportation Research Board, National Research Council, Washington, D.C., Vol. 2 (1995) pp. 33–44.
99. Klaiber, F.W., F.S. Fanous, T.J. Wipf, and H. El-Arabaty, *Design Manual for Strengthening of Continuous-Span, Composite Bridges*, Report 94404, Engineering Research Institute, Iowa State University, Ames (1993).
100. Pullaro, J.J. and B. Sivakumar, "Capacity of Truss Tension Members via High Strength Strands," *IABSE Reports Vol. 60*, IABSE, Zurich, Switzerland (1990) pp. 695–696.
101. Sadler, C., "Post-tensioning a Steel Bridge: Strengthening of the Burlington Bay Skyway," in *Developments in Short and Medium Span Bridge Engineering '90*, B. Bakht, ed., CSCE, Montreal, Canada (1990) pp. 537–546.
102. Mancarti, G.D., "Strengthening Short Span Bridges for Increased Live Loads," in *Developments in Short and Medium Span Bridge Engineering '90*, B. Bakht, ed., CSCE, Montreal, Canada (1990) pp. 169–180.
103. Goldsby, I.F., "The Strengthening and Repair of Rakewood Bridge, Part 2, the Contractor's Experience," *Highways and Transportation*, London (April 1990) pp. 39–43.
104. Miyamoto, A., A. Maki, F. Yagishita, and Y. Kobukata, "Application of Prestressing Technique for Bridge Strengthening," in *Developments in Short and Medium Span Bridge Engineering '94*, A.A. Mufti, ed., CSCE, Montreal, Canada (1994) pp. 1231–1245.
105. Bradford, M.A., "Stability of a Steel Bridge Girder Strengthened by Prestress," *Extending the Lifespan of Structures*, IABSE Report No. 73., Vol. 2, Zurich, Switzerland (1995) pp. 1497–1502.
106. Englehardt, V.L., "Strengthening of a Bridge with Epoxy Beam and Prestressed Steel," *Durability of Structures*, IABSE Report No. 57, Vol. 2, Zurich, Switzerland (1989) pp. 805–810.
107. Buckland, P.G., G.C. Medilek, and D.D. Matson, "Hagwilget Suspension Bridge: Increasing Capacity Without Strengthening," in *Bridge Management 3*, J.E. Harding, ed., E. & F.N. Spon, London (1996) pp. 867–874.
108. Gill, J., J. Jayasundara, and C. Cocksedge, "Avonmouth bridge: assessment and strengthening," in *Bridge Modifications*, B. Pritchard, ed., Thomas Telford Services Ltd., London (1995) pp. 162–173.
109. Seible, F., M.J. Priestley, and K. Krishnan, "Bridge Superstructure Rehabilitation and Strengthening," in *Transportation Research Record 1290*, Transportation Research Board, National Research Council, Washington, D.C., Vol. 1 (1991) pp. 59–67.
110. Naaman, A., "Partial Prestressing in the Rehabilitation of Concrete Bridges," *Proceedings of the 1st US-European Workshop on Bridge Evaluation, Repair and Rehabilitation*, The University of Michigan, Ann Arbor (1987) pp. 391–406.
111. Tertea I., E. Ionescu, and I. Szalma, "Repair and Strengthening of Reinforced Concrete Bridge by Prestressing," *Proceedings of the 3rd International Workshop on Bridge Rehabilitation*, Ernst & Sohn, Berlin (1992) pp. 507–513.
112. Dowrick, D.J. and P.R. Stanford, "Strengthening of Concrete Bridges by Bowstring Post-Tensioning," *New Zealand Roading Symposium*, National Roads Board, Wellington, Vol. 4 (1987) pp. 827–836.
113. Luttinen V., K. Hurskainen, and K. Saksio, "Road Bridges Strengthened by Epoxy Bonded Steel Plates," in *Bridge Management 3*, J.E. Harding, ed., E. & F.N. Spon, London (1996) pp. 409–411.
114. Appleton, J. and V. Silva, "Strengthening of Reinforced Concrete Beams by External Reinforcement," in *Extending the Lifespan of Structures*, IABSE Symposium Report, IABSE, Zurich, Switzerland, Vol. 73 (1995) pp. 1179–1184.
115. *Strengthening of Concrete Highway Structures Using Externally Bonded Plates*, Design Manual for Roads and Bridges, Vol. 3, Sect. 3, Part 1, Department of Transport, HMSO, London (1994) 38 pp.
116. Henwood, A.M. and K.J. O'Connell, "The use of externally bonded steel plates to strengthen Bolney Flyover," in *Bridge Management 2*, Thomas Telford Services Ltd., London (1993) pp. 660–671.
117. Ramsay, B., "Steel plate bonding for concrete bridge strengthening," *Construction Repair*, Palladian Publications Ltd., U.K. (Jan/Feb 1993) pp. 14–16.
118. Shanafelt, G.O. and W.B. Horn, *NCHRP Report 280: Guidelines for Evaluation and Repair of Prestressed*

Concrete Bridge Members, Transportation Research Board, National Research Council, Washington, D.C. (1985) 84 pp.

119. Preston, H.K., A.E.N. Osborn, and C.E. Roach, *Restoration of Strength in Adjacent Prestressed Concrete Box Beams*, Report No. FHWA-PA-86-044, Federal Highway Administration, Washington, D.C. (1987) 59 pp.
120. Staehli, E., J.C. Peslier, and P. Funck, "Repair and Strengthening of Concrete Structures by Post-Tensioning," in *Extending the Lifespan of Structures*, IABSE Symposium Report, IABSE, Zurich, Switzerland, Vol. 73 (1995) pp. 961–966.
121. Ouellet, C. and Y. Gaumond, "Strengthening of Two Prestressed Segmental Box Girder Bridges," in *Developments in Short and Medium Span Bridge Engineering '90*, B. Bakht, ed., CSCE, Montreal, Canada (1990) pp. 259–269.
122. Massicotte, B., A. Picard, Y. Gaumond, and C. Ouellet, "Strengthening of a Long Span Prestressed Segmental Box Girder Bridge," *PCI Journal*, Precast/Prestressed Concrete Institute, Chicago (May/June 1994) pp. 52–65.
123. Massicotte, B. and A. Picard, "Monitoring of a Prestressed Segmental Box Girder Bridge During Strengthening," *PCI Journal*, Precast/Prestressed Concrete Institute, Chicago (May/June 1994) pp. 66–80.
124. Dorton, R.A., "The Sioux Narrows Bridge, Ontario," in *IABSE Structures C-46*, IABSE, Zurich, Switzerland (1988) pp. 42–43.
125. Priestley, M.J.N., F. Seible, Y. Xiao, and R. Verma, "Steel Jacket Retrofitting of Reinforced Concrete Bridge Columns for Enhanced Shear Strength," *ACI Structural Journal*, American Concrete Institute, Vol. 91, No. 5 (1994) pp. 537–551.
126. Chai, Y.H., M.J.N. Priestley, and F. Seible, "Analytical Model for Steel-Jacketed RC Circular Bridge Columns," *Journal of Structural Engineering*, ASCE, Vol. 120, No. 8 (1994) pp. 2358–2376.
127. Seible, F., G.A. Hegemier, and V. Karbhari, "Advanced Composites for Bridge Infrastructure Renewal," *Proceedings of the Fourth International Bridge Engineering Conference*, Transportation Research Board, National Research Council, Washington, D.C., Vol. 1 (1995) pp. 348–357.
128. Demers, M. and K.W. Neale, "Strengthening of Concrete Columns with Unidirectional Composite Sheets," in *Developments in Short and Medium Span Bridge Engineering '94*, A.A. Mufti, ed., CSCE, Montreal, Canada (1994) pp. 895–905.
129. Erki, M.A. and A. Agarwal, "Strengthening of Reinforced Concrete Axial Members Using Fibre Composite Materials— A Survey," *CSCE Annual Conference Proceedings*, Ottawa, CSCE, Montreal, Canada (1995).
130. Pritchard, B., "Bridge Strengthening with Minimum Traffic Disruption," in *Bridge Modifications*, B. Pritchard, ed., Thomas Telford Services Ltd., London (1995) pp. 185–203.
131. National Engineering Technology Corporation, *BRIGDIT, Bridge Management System, User's Manual, Version 1.00*, NCHRP Project 12-28(2) (December 1994).
132. American Association of State Highway and Transportation Officials, *National Ware Proprietary Computer Software., PONTIS, Release 3.1, Users Guide, Beta Version (1)*, AASHTO, Washington, D.C., prepared by Cambridge Systematics, Inc. (March 4, 1996).
133. Green, S.G. and J.A. Richardson, "Development of a Bridge Management System in Alabama," in *Transportation Research Circular 423*, Transportation Research Board, National Research Council, Washington, D.C. (1994) pp. 72–80.
134. Lauzon, R.G. and J.M. Sime, "Connecticut's Bridge Management Information System,"in *Transportation Research Circular 423*, Transportation Research Board, National Research Council, Washington, D.C. (1994) pp. 81–86.
135. Gion, L. C., K.C. Sinha, and R.E. Woods, *User's Manual for the Implementation of the Indiana Bridge Management System*, Joint Highway Research Project, Purdue University,Lafayette, IN (1993).
136. Iowa Department of Transportation, *Iowa's Transportation Management Systems Work Plan*, Iowa Department of Transportation, Des Moines (1995) 111 pp.
137. Grivas, D.A., B. Schultz, D. Elwell, and A. Dalto, "Span based Network Characterization for Bridge Management," in *Transportation Research Record 1442*, Transportation Research Board, National Research Council, Washington, D.C. (1994) pp. 123–127.
138. Johnston, D.W. and J.D. Lee, "Analysis of Bridge Management Data in North Carolina," in *Transportation Research Circular 423*, Transportation Research Board, National Research Council, Washington, D.C. (1994) pp. 97–109.
139. Arner, R. C., J. Kruegler, H. Lathia, R. McClure and J. Mesaric, *The Pennsylvania Bridge Management System: Final Report*, Pennsylvania Department of Transportation, Bureau of Bridge and Roadway Technology, Harrisburg (1987).
140. Ruinen, F.A. and L. Bell, *Bridge Management System— Phase 1*, Clemson University, Department of Civil Engineering, Clemson, SC (1993).
141. Stukhart, G., R. James, A. Garcia-Diaz, R. Bligh and J. Sobanjo, *Study for a Comprehensive Bridge Management System for Texas (Revised)*, Texas Transportation Institute, Texas A&M University, College Station (1991).
142. McGhee, K.K., G.Allen and W. McKeel, Jr., *Development of Performance and Deterioration Curves as a Rational Basis for a Management System for Structures*, Virginia Transportation Research Council, Charlottesville, Virginia (1993).
143. Babaei, K., *Development of a Bridge Deck Management System for Washington State Department of Transportation*, Washington State Transportation Centre (TRAC), Seattle, Washington (1988).

144. Dasmohapatra, D.K. and A. Herbert, *Bridge Inspection and Management System, User Guide, Version 2.1,* Transportation and Utilities Information Systems, Alberta (September 1991).
145. Reel, R.S. and D.F. Conte, "Project Bridge Management System In Ontario," in *Transportation Research Circular 423*, Transportation Research Board, National Research Council, Washington, D.C. (1994) pp. 62–71.
146. Alambai, S. and H. Pawenang, "Strategy for Bridge Rehabilitation and Replacement," in *Bridges—Essential to our Economy, Proceedings of the Austroads Bridges Conference*, 49.1-13, Vol. 2, Austroads, New South Wales, Australia (1994).
147. Andersen, N. H., and J. Lauridsen, "Unique Characteristics of Denmark's Bridge Management System," in *Transportation Research Circular 423*, Transportation Research Board, National Research Council, Washington, D.C. (1994) pp. 55–61.
148. Mannisto, V. and R. Tapio, "Infrastructure management System: Case Study of the Finnish National Road Administration," in *Transportation Research Record 1455*, Transportation Research Board, National Research Council, Washington, D.C. (1994) pp. 132–138.
149. Legosz, A., A. Wysokowski and A. Hutnik, "Planning Bridge Maintenance in Poland Using a BMS based on Taxonomic Investigation," in *Bridge Management 3: Inspection, Maintenance, Assessment and Repair*, ed. Harding, J.E., et. al., E. & F.N. Spon, London (1996) pp.587–594.
150. Bach, P.E. and A.B. Sörensen, "Bridge Management System Implemented in Thailand," in *Proceedings, Pacific Rim Transtech Conference*, Seattle, Washington, ASCE (July 25–28, 1993).
151. Blakelock, R., "Experience in the development of a computerized Bridge Management System and its use in a number of Authorities in the UK and Overseas," in *Bridge Management 2: Inspection, Maintenance, Assessment and Repair*, ed. Harding, J.E., et. al., Thomas Telford Services Ltd., London (1993), pp.880–888.
152. Optima, Inc., Cambridge Systematics, Inc., *PONTIS Technical* Manual, prepared for the Federal Highway Administration (Jan. 31, 1992), pp. 2—8, 4–9 to 4–16.
153. Lipkus, S., "BRIDGIT, Bridge Management System Software," *in Transportation Research Circular 423*, Transportation Research Board, National Research Council, Washington, D.C. (April 1994) pp. 43–54.
154. Mufti, A.A., L.G. Jaeger, B. Bakht and L.D. Wegner, "Experimental investigation of fibre-reinforced concrete deck slabs without internal steel reinforcement," *Canadian Journal of Civil Engineering*, National Research Council of Canada, Vol. 20 (1993) pp. 398–406.
155. Bakht, B. and A.A. Mufti, "FRC Deck Slabs Without Tensile Reinforcement," *Concrete International*, American Concrete Institute, Detroit (Feb 1996) pp. 5055.
156. Bakht, B. and A.P.S. Selvadurai, "Performance of Steel-Free Deck Slabs under Simulated Rolling Wheel Loads," in *Advanced Composite Materials in Bridges and Structures*, M. El-Badry, ed., CSCE, Montreal, Canada (1996) pp. 767–776.
157. Newhook, J.P. and A.A. Mufti, "A Reinforcing Steel-Free Concrete Deck Slab for the Salmon River Bridge," *Concrete International*, American Concrete Institute, Detroit (June 1996) pp. 30–34.
158. Braimah, A., M.F. Green, and K.A. Soudki, "Eliminating Steel from Bridge Deck Slabs by Combining CFRP Tendons with Polypropylene FRC," in *Advanced Composite Materials in Bridges and Structures*, M. El-Badry, ed., CSCE, Montreal, Canada (1996) pp. 735–742.
159. "The First Steel-Free Deck Slab in Ontario," report in *Canadian Civil Engineer*, CSCE, Montreal (February 1997).
160. Karbhari, V.M., "Fiber Reinforced Composite Decks for Infrastructure Renewal," in *Advanced Composite Materials in Bridges and Structures*, M. El-Badry, ed., CSCE, Montreal, Canada (1996) pp. 759–766.
161. Zureick, A., B. Shih and E. Munley, "Fiber-Reinforced Polymeric Bridge Decks," Structural Engineering Review, Elsevier Science Ltd., Vol. 7, No. 3 (1995) pp. 257–266.
162. Radloff, B.J., M.E. Kreger, and J.E. Breen, *Bonding of External Tendons at Deviators*, Research Report 1209-1, Center for Transportation Research, University of Texas at Austin (1991) 133 pp.
163. Hindi, A., R. MacGregor, M.E. Kreger, and J.E. Breen, "Enhancing Strength and Ductility of Post-Tensioned Segmental Box Girder Bridges," *ACI Structural Journal*, American Concrete Institute, Detroit, Vol. 92, No. 1 (1995) pp. 33–44.
164. Klaiber, F.W., T.J. Wipf, and R.B. Abu-Kishk, "Utilization of partial end restraint in bridge strengthening," *Canadian Journal of Civil Engineering*, National Research Council of Canada, Vol. 20, No. 5 (1994) pp. 836–846.
165. Priestley, M.J.N., F. Seible, and E. Fyfe, "Column Seismic Retrofit Using Fibreglass/Epoxy Jackets," in *Advanced Composite Materials in Bridges and Structures*, K.W. Neale, ed., CSCE, Montreal, Canada (1992) pp. 287–298.
166. Rostasy, F.S., C. Hankers, and E.H. Ranisch, "Strengthening of R/C and P/C Structures with Bonded FRP Plates," in *Advanced Composite Materials in Bridges and Structures*, K.W. Neale, ed., CSCE, Montreal, Canada (1992) pp. 253–263.
167. Meier, U., M. Deuring, H. Meier, and G. Schwegler, "Strengthening of Structures with CFRP Laminates: Research and Applications in Switzerland," in *Advanced Composite Materials in Bridges and Structures*, K.W. Neale, ed., CSCE, Montreal, Canada (1992) pp. 243–251.
168. Meier, U., M. Deuring, H. Meier, and G. Schwegler, "Strengthening of Structures with Advanced Composites," in *Alternative Materials for the Reinforcement and Prestressing of Concrete*, J.E. Clarke, ed., Chapman and Hall, London (1993) pp. 153–171.

169. *Rehabilitation of Foulk Road Bridge #26 (Wilmington, Delaware) Using Advanced Composite Materials*, FRP International, S.H. Rizkalla, ed., University of Manitoba, Winnipeg, Canada, Vol. III, Issue 2 (1995) pp. 3–4.
170. *Carbon Fiber Plates for Ohio Bridge*, FRP International, S.H. Rizkalla, ed., University of Manitoba, Winnipeg, Canada, Vol. IV, Issue 3 (1996) p. 3.
171. *Research at Georgia Institute of Technology*, FRP International, S.H. Rizkalla, ed., University of Manitoba, Winnipeg, Canada, Vol. IV, Issue 1 (1996) p. 5.
172. Head, P.R., "Advanced Composites in Civil Engineering—A Critical Overview at this High Interest, Low Use Stage of Development," in *Advanced Composite Materials in Bridges and Structures*, M. El-Badry, ed., CSCE, Montreal, Canada (1996) pp. 3–15.
173. Meier, U., and A. Winistörfer, "Retrofitting of Structures Through External Bonding of CFRP Sheets," in *Non-Metallic (FRP) Reinforcement for Concrete Structures*, L. Taerwe, ed., E. & F.N. Spon, London (1995) pp. 465–472.
174. Meier, U., "Strengthening of Structures using Carbon Fibre/Epoxy Composites," *Construction and Building Materials,* Elsevier Science Ltd., Vol. 9, No. 6 (1995) pp. 341–351.
175. Steiner, W., "Strengthening of Structures with CFRP Strips," in *Advanced Composite Materials in Bridges and Structures*, M. El-Badry, ed., CSCE, Montreal, Canada (1996) pp. 407–417.
176. Triantafillou, T.C. and N. Plevris, "Reliability Analysis of Reinforced Concrete Beams Strengthened with CFRP Laminates," in *Non-Metallic (FRP) Reinforcement for Concrete Structures*, L. Taerwe, ed., E. & F.N. Spon, London (1995) pp. 576–583.
177. Varastehpour, H., and P. Hamelin, "Analysis and Study of Failure Mechanism of RC Beam Strengthened with FRP Plate," in *Advanced Composite Materials in Bridges and Structures*, M. El-Badry, ed., CSCE, Montreal, Canada (1996) pp. 527–536.
178. Varastehpour, H. and P. Hamelin, "Experimental Study of RC Beams Strengthened with CFRP Plate," in *Advanced Composite Materials in Bridges and Structures*, M. El-Badry, ed., CSCE, Montreal, Canada (1996) pp. 555–563.
179. Peshkam, V. and M.B. Leeming, "The Use of Advanced Composite Materials in Strengthening and Maintaining Bridges," in *Bridge Management 3*, J.E. Harding, ed., E. & F.N. Spon, London (1996) pp. 732–742.
180. Quantrill, R.J., L.C. Holloway, A.M. Thorne, and G.A.R. Parke, "Preliminary Research on Strengthening of Reinforced Concrete Beams Using GFRP," in *Non-Metallic (FRP) Reinforcement for Concrete Structures*, L. Taerwe, ed., E. & F.N. Spon, London (1995) pp. 541–550.
181. Hutchinson, A.R. and H. Rahimi, "Flexural Strengthening of Concrete Beams with Externally Bonded FRP Reinforcement," in *Advanced Composite Materials in Bridges and Structures*, M. El-Badry, ed., CSCE, Montreal, Canada (1996) pp. 519–526.
182. Swamy, R.N., C.J. Lynsdale, and P. Mukhopadhyaya, "Effective Strengthening with Ductility: Use of Externally Bonded Plates of Non-Metallic Composite Materials," in *Advanced Composite Materials in Bridges and Structures*, M. El-Badry, ed., CSCE, Montreal, Canada (1996) pp. 481–488.
183. Swuramy, R.N. and P.Mukhopadhyaya, "Role and Effectiveness of Non-Metallic Plates in Strengthening and Upgrading Concrete Structures," in *Non-Metallic (FRP) Reinforcement for Concrete Structures*, L. Taerwe, ed., E. & F.N. Spon, London (1995) pp. 473–482.
184. Hefferman, P.J. and M.A. Erki, "Equivalent Capacity—A Design Concept for FRP Strengthened Reinforced Concrete Beams," in *Non-Metallic (FRP) Reinforcement for Concrete Structures*, L. Taerwe, ed., E. & F.N. Spon, London (1995) pp. 517–524.
185. Hefferman, P.J. and M.A. Erki, "Equivalent Capacity and Efficiency of Reinforced Concrete Beams Strengthened with Carbon Fibre Reinforced Plastic Sheets," *Canadian Journal of Civil Engineering*, National Research Council of Canada, Vol. 23 (1996) pp. 21–29.
186. Erki, M.A. and P.J. Hefferman, "Reinforced Concrete Slabs Externally Strengthened with Fibre-Reinforced Plastic Materials," in *Non-Metallic (FRP) Reinforcement for Concrete Structures*, L. Taerwe, ed., E. & F.N. Spon, London (1995) pp. 509–516.
187. Wight, R.G., M.F. Green, and M.A. Erki, "Post-Strengthening Concrete Beams with Prestressed FRP Sheets," in *Non-Metallic (FRP) Reinforcement for Concrete Structures*, L. Taerwe, ed., E. & F.N. Spon, London (1995) pp. 568–575.
188. Wight, R.G., M.F. Green, and M.A. Erki, "Post-Strengthening Concrete Beams with Prestressed FRP Sheets," in *Advanced Composite Materials in Bridges and Structures*, M. El-Badry, ed., CSCE, Montreal, Canada (1996) pp. 449–456.
189. Drimoussis, E. and J.J.R. Cheng, "Shear Strengthening of Concrete Bridge Girders Using Carbon Fiber-Reinforced Plastic Sheets," *Proceedings of the Fourth International Bridge Engineering Conference*, Transportation Research Board, National Research Council, Washington, D.C. (1995) pp. 337–347.
190. Alexander, J.G.S. and J.J.R. Cheng, "Field Application and Studies of Using CFRP Sheets to Strengthen Concrete Bridge Girders," in *Advanced Composite Materials in Bridges and Structures*, M. El-Badry, ed., CSCE, Montreal, Canada (1996) pp. 465–472.
191. Razaqpur, A.G. and M.M. Ali, "Ductility and Strength of Concrete Beams Externally Reinforced with CFRP Sheets," in *Advanced Composite Materials in Bridges and Structures*, M. El-Badry, ed., CSCE, Montreal, Canada (1996) pp. 505–512.
192. Demers, M., D. Hébert, M. Gauthier, P. Labossière, and K.W. Neale, "The Strengthening of Structural Concrete with an Aramid Woven Fibre/Epoxy Resin Composite," in *Advanced Composite Materials in Bridges and Structures*, M. El-Badry, ed., CSCE, Montreal, Canada (1996) pp. 435–442.

193. Blais, C. and A. Picard, "Flexural Strengthening of Reinforced Concrete Beams with Composite Plates," in *Developments in Short and Medium Span Bridge Engineering '94*, A.A. Mufti, ed., CSCE, Montreal (1994) pp. 919–929.
194. Bonacci, J.F., "Strength, Failure Mode and Deformability of Concrete Beams Strengthened Externally with Advanced Composites," in *Advanced Composite Materials in Bridges and Structures*, M. El-Badry, ed., CSCE, Montreal, Canada (1996) pp. 419–426.
195. Hoa, S.V., M. Xie, and X.R. Xiao, "Repair of Steel Reinforced Concrete with Carbon/Epoxy Composites," in *Advanced Composite Materials in Bridges and Structures*, M. El-Badry, ed., CSCE, Montreal, Canada (1996) pp. 573–580.
196. Banthia, N., C. Yan, and N. Nandakumar, "Sprayed Fiber Reinforced Plastics (FRP's) for Repair of Concrete Structures," in *Advanced Composite Materials in Bridges and Structures*, M. El-Badry, ed., CSCE, Montreal, Canada (1996) pp. 537–545.
197. Ritchie, P.A., D.A. Thomas, L.W. Lu, and G.M. Connelly, "External Reinforcement of Concrete Beams using Fiber Reinforced Plastics," *ACI Structural Journal*, American Concrete Institute, Vol. 88, No. 4 (1991) pp. 490–500.
198. Saadatmanesh, H. and M.R. Ehsani, "RC Beams Strengthened with FRP Plates I: Experimental Study," *Journal of Structural Engineering*, ASCE, Vol. 117, No. 11 (1991) pp. 3417–3433.
199. Ehsani, M.R. and H. Saadatmanesh, "Fiber Composites for Bridge Rehabilitation," in *Bridge Evaluation, Repair and Rehabilitation*, A.S. Nowak, ed., Kluwer Academic Publishers, Norwell, MA (1990) pp. 579–588.
200. Char, M.S., H. Saadatmanesh, and M.R. Ehsani, "Concrete Girders Externally Prestressed with Composite Plates," *PCI Journal*, Precast/Prestressed Concrete Institute (May-June 1994) pp. 40–51.
201. Malek, A.M., H. Saadatmanesh, and M.R. Ehsani, "Shear and Normal Stress Concentrations in RC Beams Strengthened with FRP Plates," in *Advanced Composite Materials in Bridges and Structures*, M. El-Badry, ed., CSCE, Montreal, Canada (1996) pp. 629–637.
202. Sen, R. and L. Liby, *Repair of Steel Composite Bridge Sections using CFRP Laminates*, College of Engineering, University of South Florida (1994) 94 pp.
203. Sen, R., L. Liby, K. Spillet, and G. Mullins, "Strengthening Steel Composite Bridge Members Using CFRP Laminates," in *Non-Metallic (FRP) Reinforcement for Concrete Structures*, L. Taerwe, ed., E. & F.N. Spon, London (1995) pp. 551–558.
204. Chajes, M.J., T.A. Thomson, and B. Tarantino, "Reinforcement of Concrete Structures Using Externally Bonded Composite Materials," in *Non-Metallic (FRP) Reinforcement for Concrete Structures*, L. Taerwe, ed., E. & F.N. Spon, London (1995) pp. 501–508.
205. Chajes, M.J., W.W. Finch, T.F. Januszka, and T.A. Thomson, "Bond and Force Transfer of Composite Material Plates Bonded to Concrete," *ACI Structural Journal*, American Concrete Institute, Vol. 93, No. 2 (1996) pp. 208–217.
206. Chajes, M.J., V.N. Kaliakin, S.D. Holsinger, and A.J. Meyer "Experimental Testing of Composite Wood Beams for Use in Timber Bridges," *Proceedings of the Fourth International Bridge Engineering Conference*, Transportation Research Board, National Research Council, Washington, D.C. (1995) pp. 371–380.
207. Arockiasamy, M., *Experimental Studies on the Feasibility of Use of Carbon Fiber Reinforced Plastics in Repair of Concrete Bridges*, Center for Infrastructure and Constructed Facilities, Florida Atlantic University (1995) 230 pp.
208. Arockiasamy, M., R. Sowrirajan, and M. Zhuang, "Behaviour of Beams Prestressed or Strengthened with Fiber Reinforced Plastic Composites," in *Extending the Lifespan of Structures*, IABSE Symposium Report, IABSE, Zurich, Switzerland, Vol. 73 (1995) pp. 997–1002.

GLOSSARY OF ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
ACM	Advanced Composite Materials
AFRP	Aramid Fiber-Reinforced Plastic
ASD	Allowable Stress Design
CEI	Cost-Effectiveness Index
CFRP	Carbon Fiber-Reinforced Plastic
CSA	Canadian Standards Association
DOT	Department of Transportation
EMPA	Swiss Federal Laboratories for Material Testing and Research
FEM	Finite Element Method
FHWA	Federal Highway Administration
FLS	Fatigue Limit State
FRC	Fiber-Reinforced Concrete
FRP	Fiber-Reinforced Plastic
GFRP	Glass Fiber-Reinforced Plastic
LFD	Load Factor Design
LRFD	Load and Resistance Factor Design
MTO	Ministry of Transportation, Ontario
MR&R	Maintenance, Repair, and Replacement
NCHRP	National Cooperative Highway Research Program
OHBDC	Ontario Highway Bridge Design Code
PVC	Polyvinyl Chloride
PWC	Public Works Canada
RF	Rating Factor
RMC	Royal Military College
SD	Strength Design
SLD	Service Load Design
SLS	Serviceability Limit State
ULS	Ultimate Limit State
WSD	Working Stress Design

APPENDIX A

Survey Questionnaire

This questionnaire was sent to transportation agencies and consultants in the United States and Canada

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Project 20-5, Topic 27-04

Methods for Increasing Live Load Capacity of Existing Highway Bridges

QUESTIONNAIRE

Name of Respondent: ______
Agency or Company: ______
Title: ______
Phone No: ______ Fax No.: ______

Please enter your agency or company name at the top of each subsequent sheet. **If more space is required to answer or comment on any of the questions, please use the margins or a separate sheet of paper**. If you wish additional persons in your agency to reply, please make copies for their use.

Please circle your answers to Yes/No questions.

Q-1. What analytical methods do you normally use to evaluate the live load capacity of bridges?

______

Q-2. Have you used more sophisticated analytical methods in an attempt to raise the predicted live load capacity?
Yes No

If the answer to the above is Yes, please describe the methods used and the results.______

NCHRP Topic 27-04 Agency Name:

Q-3. Have you used full scale load testing, directly or by contract, to upgrade the live load rating of a bridge?
Yes No

If the answer to the above is Yes, please describe the test and the results.______

Q-4. Have you retrofitted any fatigue prone details on steel bridges in order to increase live load capacity?
Yes No

If the answer to the above is Yes, please describe the method used and the results.______

Q-6. Have you modified a bridge superstructure since 1986, specifically to increase its live load capacity?
Yes No

If the answer to the above is Yes, please indicate what methods were used, and what considerations were taken into account, in making the decision to strengthen rather than replace.

______

If the answer to Q-6 above is Yes, please complete the table on page 3, indicating the number of bridges strengthened for live load since 1986, by superstructure material and method.

Any comments on your table entries.______

NCHRP Topic 27-04 Agency Name:

Table Showing Number of Bridges Strengthened for Live Load Since 1986

	Superstructure					
Methods for Increasing Live Load Capacity	Steel	Reinforced Concrete	Pretensioned Concrete	Post-Tensioned Concrete	Wood	Masonry
Reduce Dead Load						
Composite Action						
Increase Transverse Stiffness						
Improve Member Strength						
Post-tension						
Develop Continuity						
Other Methods						

Note: For Other Methods please give details in Bridge Data Sheets

NCHRP Topic 27-04 Agency Name:

Q-7. Have you modified a bridge substructure element (pier, abutment, or foundation) specifically to increase its live load capacity since 1986?
Yes No

If the answer to the above is Yes, please describe how the work was done, and how the method was selected.________________

Q-8. If your answer to Q-6 or Q-7 is Yes, have you assessed the effect of the modification on seismic behavior?
Yes No

If the answer to the above is Yes, please describe the results.

Q-9. Have you recently developed any innovative techniques for increasing the live load capacity of bridges?
Yes No

If the answer to the above is Yes, please describe the techniques and the status of their development.________________

Q-10. If you answered No to both Q-6 and Q-7, please proceed to Q-12. If you answered Yes to Q-6 or Q-7, please continue.

We would like to obtain as much detailed information as is available on those methods of increasing live load capacity that you consider to be of special interest or significance. We have a particular interest in any methods you might have listed under Other Methods in the table on page 3, or any that you consider as experimental.

For each bridge, please use a separate Bridge Data Sheet, photocopying page 5 as necessary, and assigning a Bridge No. to each.

NCHRP Topic 27-04 Agency Name:

BRIDGE DATA SHEET

Bridge #________

Bridge type and superstructure material:________________

Method used for increasing capacity:________________

Was the above method: Standard? Yes No or Experimental? Yes No

Why was an increase in capacity needed?________________

What was the original design vehicle?________________

What % increase in live load capacity was achieved?________________

Was the decision to increase capacity justified on the basis of:
First cost only? Yes No or Life cycle cost analysis? Yes No

Construction contract year:________ Contract cost:________________

What % of contract cost was for traffic protection?________ Were lane closures required? Yes No

Overall deck width:________ Length of each span modified:________________

Bridge Code used: ________

Additional comments: (Any information available on such items as difficulties encountered, suggested improvements, factors affecting costs, construction time, quality control testing, projected service life, field performance, and maintenance costs, would be of great value)

If plans, reports of photographs are available, kindly send copies with your response, if possible, or indicate below who we might contact for copies.
Name:________________ Title:________________
Address:________________ Phone #:________________

Q-11. Was the need to increase capacity identified by your Bridge Management System, for any of the projects you have listed on the Bridge Data Sheets?

Yes No

Q-12. Does your Bridge Management System have decision matrixes for increasing capacity?

Yes No

If the answer to the above is Yes, please identify the system:

____

Q-13. From your recent experience, do you think the need to strengthen bridges for live load capacity is increasing?

Yes No

Q-14. We are interested in finding out what new techniques or materials are being developed for increasing live load capacity. If you know of any research organizations, product manufacturers or consultants active in this area we would appreciate your identifying them, and the nature of their work.

Name:______________________ Title:______________________

Address:______________________ Phone No:______________________

______________________ Fax No:______________________

____

Comments:______________________________

____

____

****

Thank you for your valuable assistance!

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

Attn: Stephen F. Maher
NCHRP Research Syntheses

If you have any questions, please call Stephen Maher at (202) 334-3245.
If you wish, you may fax your response to (202) 334-2527.

We would appreciate your response by April 5, 1997.

APPENDIX B

Summary of Survey Responses

The responses to the survey questionnaire (Appendix A) are summarized in Table B1 on the following pages. Numerical totals are given separately for public agencies and consultants. The questions asked are given below in an abbreviated form as Table B1 shows only the question number. The responses to Q1 are shown in Table B2 and more detailed responses to Q2 and Q3 are shown in Tables B3 and B4, respectively.

Question 1—What analytical methods do you normally use to evaluate live load capacity?

Question 2—Have you used more sophisticated analytical methods to raise the capacity?

Question 3—Have you used full-scale load testing to upgrade the live load rating?

Question 4—Have you retrofitted any fatigue-prone steel details to increase capacity?

Question 6—Have you modified a bridge superstructure since 1986 to increase capacity?
(The numbers are given under the heading of *Table 5*.)

Question 7—Have you modified a bridge substructure since 1986 to increase capacity?

Question 8—Have you assessed the effect of any Q7 modification on seismic behaviour?

Question 9—Have you recently developed any innovative techniques to raise capacity?

Question 10—Would you please fill out a data sheet for methods of particular interest?
(The number of completed data sheets is shown in the Q10 column.)

Question 11—Was the need to increase capacity identified by your bridge management system?

Question 12—Does your bridge management system have decision matrices for raising capacity?

Question 13—Do you think the need to strengthen bridges for live load capacity is increasing?

Question 14—Would you please identify new techniques for increasing capacity, by others?

TABLE B-1

SUMMARY OF SURVEY RESPONSES

Agency	Q1	Q2	Q3	Q4	Table 5	Q6	Q7	Q8	Q9	Q10	Q11	Q12	Q13	Q14
Alabama		Yes	Yes	Yes	0	No	Yes	No	No	0	-	No	Yes	-
Alaska		No	No	Yes	8	Yes	Yes	Yes	No	5	-	-	-	-
Arizona		No	No	No	0	No	No	-	No	0	-	No	No	-
Arkansas		No	No	No	12	Yes	Yes	Yes	No	-	No	No	Yes	1
California		Yes	No	No	48	Yes	No	No	No	-	No	No	No	1
Colorado		Yes	Yes	No	32	Yes	-	-	-	1	-	Yes	-	-
Connecticut		No	Yes	No	11	Yes	Yes	No	No	2	No	No	No	-
Florida		Yes	Yes	No	3	Yes	No	No	Yes	1	No	No	No	1
Georgia		No	No	No	0	No	No	-	No	0	-	No	No	1
Iowa		Yes	No	No	4	Yes	No	No	No	-	No	Yes	Yes	-
Kansas		Yes	Yes	Yes	55	Yes	No	No	Yes	2	No	No	Yes	-
Kentucky		No	No	No	0	No	No	-	No	0	-	No	No	-
Louisiana		Yes	Yes	Yes	7	Yes	No	No	No	1	No	No	No	-
Maine		Yes	No	No	7	Yes	No	-	No	-	No	Yes	No	-
Massachusetts		No	No	Yes	3	Yes	Yes	Yes	No	1	No	Yes	Yes	-
Michigan		No	Yes	No	2	Yes	No	-	No	1	-	-	-	-
Minnesota		No	No	No	55	Yes	Yes	No	Yes	-	No	No	No	-
Missouri		No	Yes	No	90	Yes	No	Yes	No	-	No	No	Yes	-
Mississippi		No	No	No	4	Yes	No	No	No	0	No	No	Yes	-
Montana		Yes	No	No	0	No	No	-	No	0	-	No	No	-
Nebraska		No	No	No	0	No	No	-	No	0	-	No	No	1
Nevada		No	No	No	1	Yes	No	-	No	-	-	No	No	-
New Jersey		Yes	No	Yes	8	Yes	Yes	No	No	4	No	No	Yes	1
New Mexico		No	No	No	1	Yes	No	-	No	-	-	No	No	-
New York		Yes	Yes	No	5	Yes	No	Yes	No	-	No	No	Yes	-
North Carolina		No	No	No	40	Yes	Yes	No	No	-	No	No	Yes	-
North Dakota		No	No	No	0	No	No	-	No	0	-	No	Yes	-
Ohio		Yes	Yes	No	575	Yes	No	No	No	-	No	No	Yes	-
Oklahoma		Yes	Yes	Yes	0	No	No	-	No	0	-	-	Yes	2
Oregon		Yes	Yes	Yes	37	Yes	Yes	Yes	No	3	No	No	Yes	-
Pennsylvania		Yes	Yes	No	5	Yes	Yes	No	Yes	-	Yes	No	Yes	-
Rhode Island		Yes	No	Yes	2	Yes	Yes	No	No		No	No	Yes	-
Tennessee		No	No	No	0	No	No	-	No	0	-	Yes	Yes	-
Texas		Yes	No	Yes	3	Yes	Yes	No	Yes	1	No	No	Yes	-
Vermont		No	No	No	0	No	No	-	No	0	-	No	Yes	-
Virginia		No	No	No	-	Yes	Yes	No	No	-	No	No	No	-
Washington		No	No	No	0	No	No	-	-	-	-	No	Yes	-
West Virginia		No	No	No	2	Yes	No	No	No	0	No	No	Yes	-
Wisconsin		No	No	No	15	Yes	No	No	No	-	-	No	Yes	-
Wyoming		No	Yes	No	0	No	No	-	No	-	-	No	No	1
Washington, FHWA/EFLHD		No	No	No	3	Yes	No	-	-	0	Yes	No	No	1

TABLE B-1 (Continued)

Agency	Q1	Q2	Q3	Q4	Table 5	Q6	Q7	Q8	Q9	Q10	Q11	Q12	Q13	Q14
Canadian Provinces														
Federal, PWC		No	Yes	-	29	Yes	No	No	No	4	Yes	No	Yes	-
Alberta		Yes	No	No	89	Yes	Yes	No	Yes	-	Yes	Yes	Yes	1
British Columbia		Yes	No	Yes	10	Yes	No	No	No	6	No	No	Yes	1
Manitoba		No	No	No	3	Yes	No	-	No	1	No	-	Yes	-
New Brunswick		Yes	No	No	80	Yes	No	No	No	-	-	-	-	-
Ontario		Yes	Yes	No	20	Yes	Yes	No	No	-	-	No	No	-
Saskatchewan		No	No	No	9	Yes	Yes	No	Yes	7	Yes	No	Yes	-
Totals: US & Canada		Y - 21 N - 27	Y - 16 N - 32	Y - 11 N - 36	1278	Y - 36 N - 12	Y - 16 N - 31	Y - 6 N - 24	Y - 7 N - 38	40	Y -5 N - 23	Y - 6 N - 36	Y - 27 N - 17	12

Consultants	Q1	Q2	Q3	Q4	Table 5	Q6	Q7	Q8	Q9	Q10	Q11	Q12	Q13	Q14
Consultants (Canada)														
Hatch Associates		Yes	Yes	Yes	8	Yes	No	Yes	No	2	No	No	Yes	1
Proctor & Redfern		Yes	Yes	No	12	Yes	No	No	No	-	No	No	Yes	-
Delcan		Yes	Yes	No	0	No	No	-	No	0	-	Yes	-	1
Morrison Hershfield		Yes	No	No	11	Yes	No	No	No	-	No	-	Yes	-
Buckland & Taylor		Yes	No	Yes	7	Yes	No	Yes	Yes	7	-	-	No	-
Consultants (United States)														
PBQD		Yes	Yes	No	6	Yes	Yes	Yes	No	0	-	-	Yes	1
Wilbur Smith Associates		No	No	No	1	Yes	No	No	No	0	-	-	Yes	-
Burgess and Niple		No	No	Yes	Many	Yes	Yes	Yes	No	1	No	-	Yes	1
P.P. Xanthakos		-	Yes	-	2	Yes	Yes	-	-	-	-	-	-	-
Hardesty & Hanover		No	Yes	Yes	4	Yes	No	No	No	4	-	-	Yes	1
Roman Wolchuk		-	No	No	2	Yes	No	-	Yes	2	-	-	-	1
T.Y. Lin		No	No	No	1	Yes	No	No	Yes	1	No	No	Yes	1
HNTB (Denver)		Yes	No	No	0	No	No	-	No	0	-	No	No	-
HNTB (Boston)		No	No	No	5	Yes	No	Yes	No	3	-	-	Yes	-
HNTB (Dallas)		No	No	No	1	Yes	Yes	No	No	2	No	-	No	-
HNTB (Topeka)		Yes	Yes	No	2	Yes	No	No	No	2	-	-	Yes	1
HNTB (Cleveland)		Yes	Yes	Yes	2	Yes	No	No	No	-	-	-	-	-
HNTB (Kansas City)		Yes	No	Yes	13	Yes	Yes	No	No	-	No	No	-	-
HNTB (New York)		Yes	No	Yes	6	Yes	No	-	No	-	-	No	Yes	-
HNTB (Virginia)		Yes	No	Yes	3	Yes	No	-	No	1	No	No	No	-
NNTB (Raleigh)		Yes	No	No	1	Yes	No	No	No	1	No	No	-	-
HNTB (Houston)		No	No	No	1	Yes	No	-	No	-	-	-	Yes	-
HNTB (Atlanta)		Yes	No	No	1	Yes	No	No	No	1	No	No	Yes	-
HNTB (Minneapolis)														
HNTB (Hartford)														
Modjeski & Masters		Yes	Yes	Yes	5	Yes	Yes	Yes	Yes	7	-	-	Yes	-
Imbsen & Associates		Yes	No	No	1	Yes	No	No	No	0	-	No	Yes	-
Combined U.S. & Canada Consultants		Y - 16 N - 7	Y - 9 N - 16	Y - 9 N - 15	95+	Y - 23 N - 2	Y - 6 N - 19	Y - 6 N - 12	Y - 4 N - 20	34	Y - 0 N - 10	Y - 1 N - 10	Y - 15 N - 4	8

TABLE B-2

ANALYTICAL METHODS USED TO EVALUATE LIVE LOAD CAPACITY OF BRIDGES

Analytical Method	United States and FHWA	U.S. Consultants	Canadian Provinces and PWC	Canadian Consultants
Working Stress Design	13	3	-	-
Load Factor Design	16	5	-	-
- BARS	8	1	-	-
- BRASS	3	1	1	-
- BDS	-	1	-	-
- STRUDL	3	3	-	-
- SFRAME	2	-	-	-
- PFRAME	1	1	-	-
- 2D	2	1	-	-
- STADD	1	-	-	-
- MICAS	1	-	-	-
- Line Girder, AASHTO	4	3	-	-
- Texas DOT	-	2	-	-
- SAP	-	1	-	-
- DESCUS	-	1	-	-
- DSDI	-	1	-	-
- FEA	-	1	-	-
- Own computer program	2	2	-	-
- Hand calculations	1	1	-	-
Load and Resistance Factor Design	1	3	-	-
Ultimate Limit State	-	-	1	1
- OHBDC	-	-	1	1
- GRID	-	-	-	1
- OMBAS	-	-	-	1
- PSFRAME	-	-	-	1
- Grid analysis, CAMIL	-	-	-	1
- STADD III	-	-	-	1
- STRUDL	-	-	-	1
- SAP 90				

TABLE B-3

MORE SOPHISTICATED ANALYTICAL METHODS USED TO RAISE THE PREDICTED LIVE LOAD CAPACITY

Analytical Method	United States and FHWA	U.S. Consultants	Canadian Provinces and PWC	Canadian Consultants
Linear Methods				
- LFD	2	-	-	-
- Grid analysis	-	6	3	1
- Semi-continuum	-	-	2	-
- PSFRAME	-	-	1	-
- 2D	1	-	-	-
- 3D	4	4	-	1
- SAP 90	1	-	-	1
- FEM	7	6	1	3
- BRUFEM	2	-	-	-
- CURV Bridge	1	-	-	-
- As required to avoid posting	1	-	-	-
- Influence Surfaces	-	1	-	-
- Beam	-	1	-	-
- Plate	-	1	-	-
- LRFD	1	1	-	-
Nonlinear Methods				
- Plastic	-	-	-	2

TABLE B-4

FULL-SCALE LOAD TESTING USED TO UPGRADE THE LIVE LOAD RATING OF A BRIDGE

Reason/Bridge Type Tested	United States and FHWA	U.S. Consultants	Canadian Provinces and PWC	Canadian Consultants
To remove posting	1	-	1	-
To verify analysis	2	1	-	-
To proof load	1	-	1	-
Slab bridge	4	-	-	-
Timber bridge	2	-	-	-
Skew reinforced concrete	1	-	-	-
Prestressed Beams	1	-	-	-

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

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