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Proceedings of the Fourth National Workshop on Bridge Research in Progress

▼

June 17–19, 1996
Buffalo, New York

Edited by

Ian G. Buckle
State University of New York at Buffalo

Ian M. Friedland
National Center for Earthquake Engineering Research

Sponsored by the National Science Foundation

Organized by the State University of New York at Buffalo and the National Center for Earthquake Engineering Research

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PREFACE

This report is the proceedings of the Fourth National Workshop on Bridge Research in Progress, which was held in Buffalo, New York, from June 17 to 19, 1996. This workshop was in fact a forum for the bridge research and design community to exchange information about the nature and progress of research in North America. Participants included bridge researchers and designers, and representatives from agencies that own and maintain bridges, and those which support transportation research. Sponsored by the National Science Foundation, the workshop was organized by the State University of New York at Buffalo and the National Center for Earthquake Engineering Research. Previous workshops in this series were held at Iowa State University in 1988, the University of Nevada, Reno in 1990, and the University of California, San Diego in 1992.

More than 60 oral presentations and 15 poster presentations were made during the 2½ days of the workshop. These proceedings contain the abstracts and technical papers presented during the meeting. The papers are grouped into six subject areas: (1) bridge management and condition assessment; (2) seismic performance, design, and retrofitting; (3) concrete, masonry, and composite bridges; (4) steel bridges; (5) composite and other materials; and (6) analysis, loads, and bearings. These proceedings also contain a list of poster presentations (as of press time) and a report summarizing the results of research that was "in progress" at the time of the third workshop in San Diego in 1992.

Proceedings of the previous three workshops have been published as follows:

> "Bridge Research in Progress Proceedings: A symposium funded by the National Science Foundation and sponsored by Iowa State University, September 26-27, 1988," Iowa State University Bridge Engineering Center, Des Moines, Iowa, 1988, 323 pp.
>
> "Proceedings: Second Workshop on Bridge Engineering Research in Progress," sponsored by the National Science Foundation and Civil Engineering Department, University of Nevada, Reno, October 29-30, 1990, Reno, Nevada, 1990, 320 pp.
>
> "Proceedings: Third NSF Workshop on Bridge Engineering Research in Progress," November 16 & 17, 1992,La Jolla, California, University of California, San Diego, 1992 364 pp.

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15. Supplementary Notes

This workshop was sponsored by the National Science Foundation under a grant to the State University of New York at Buffalo.

16. Abstract (limit 200 Words)

The Fourth National Workshop on Bridge Research in Progress was held June 17 - 19, 1996 in Buffalo, New York. This volume contains 67 papers presented at the Workshop. These papers represent a cross-section of current bridge research being conducted throughout the United States and Canada. The Workshop's sessions were divided into the following topics: Bridge Management and Condition Assessment; Seismic Performance, Design, and Retrofitting: Concrete, Masonry, and Composite Construction: Steel Bridges; Composite and Other Materials; and Analysis, Loads, and Bearings. These proceedings serve as an excellent resource for bridge researchers, engineers, and owners seeking up-to-date information on topics, progress and sponsors of bridge research in the United States and Canada.

17. Document Analysis a. Descriptors

Earthquake Engineering. Bridge management. Bridge condition assessment. Seismic performance. Seismic design. Retrofitting. Masonry bridges. Composite bridges. Concrete bridges. Steel bridges. Analysis. Loads. Bearings.

b. Identifiers/Open-Ended Terms

c. COSATI Field/Group

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WORKSHOP STEERING COMMITTEE

Ian G. Buckle, Chair
State University of New York at Buffalo, Buffalo, New York

Ian M. Friedland, Co-chair
National Center for Earthquake Engineering Research, Buffalo, New York

James D. Cooper, Federal Highway Administration, McLean, Virginia

Bruce M. Douglas, University of Nevada, Reno

Catherine French, University of Minnesota, Minneapolis, Minnesota

F. Wayne Klaiber, Iowa State University, Des Moines, Iowa

Zolan Prucz, Modjeski and Masters, Inc., New Orleans, Louisiana

Mary Lou Ralls, Texas Department of Transportation, Austin, Texas

Scott A. Sabol, Transportation Research Board, Washington, DC

Frieder Seible, University of California, San Diego

Arun M. Shirole, National Steel Bridge Alliance, Robbinsdale, Minnesota

John B. Scalzi, Cognizant Program Director
National Science Foundation, Arlington, Virginia

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Bridge Management and Condition Assessment

SESSION I

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Design of Bridge Inspection Programs for Structural Reliability

Author(s) and Affiliation(s):

George Hearn, Dan M. Frangopol, and Steven Marshall
Department of Civil, Environmental and Architectural Engineering
University of Colorado, Boulder, CO

Principal Investigator: George Hearn

Sponsor(s): National Science Foundation

Research Start Date: September 1994
Expected Completion Date: August 1997

Research Objectives:

The quality of condition assessments of bridges depends on the accuracy and precision of inspection methods. Limitations in inspection methods produce uncertainties in the condition of bridges. The uncertainty of condition assessment is a basis for the evaluation of inspection methods. Condition assessments are estimated both as mean values and as lower bounds consistent with the variability in performance of an inspection method. These estimates are incorporated in evaluations of bridge strength and safety. The disparity between mean values of strength and safety and lower bound values is a measure of the quality of the inspection. This is an end-use basis for measurement of performance of inspection methods and leads naturally to requirements for high accuracy and precision in inspections for critical defects, and confirms the utility of simpler inspections where limited precision may be acceptable. The measurement of performance of inspection methods makes possible the design of inspection programs. Inspection methods can be selected and applied so that goals for assurance of structural condition are met.

Expected Products or Deliverables:

Products of the research include models of the performance of inspection methods, procedures for the evaluation of significance of errors in condition assessments, and a process for the selection of inspection methods to achieve a desired level of assurance in condition assessments. The process for the selection of inspection methods can also indicate when a desired level of accuracy or precision can not be obtained.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

George Hearn is Assistant Professor of Civil Engineering in the College of Engineering and Applied Science at the University of Colorado at Boulder. He specializes in bridge engineering with emphasis in bridge assessments, inspections, and the development of quantitative evaluations of strength and safety within bridge management systems. Professor Hearn is the principal investigator for projects that include the creation of procedures for reliability-based design of inspection programs for bridges, the automated generation of NBI condition ratings from bridge management system data, for the enhancement of element condition reports to allow load capacity evaluation of bridges within BMS, the use of transient response of bridges in support of condition assessment, and the study of the vulnerability of bridges to differential settlements at foundations. Professor Hearn has worked with the Colorado DOT in the design and evaluation of transportation structures, resulting in two US patents: cable-net rockfall barriers and flexible facing systems for mechanically stabilized earth walls. He is a member of TRB and ASCE, and is active in numerous committees in both organizations. Before joining the faculty at Boulder, he was a consulting engineer working on rehabilitation of suspension bridges. Professor Hearn is a licensed professional engineer in New York and Colorado.

Dan M. Frangopol is Professor of Civil Engineering at the University of Colorado at Boulder. He received his Ph.D. in civil engineering from the University of Liege, Belgium in 1976. His background covers academic-oriented research and development in the fields of structural reliability, safety evaluation of existing structures, bridge engineering and structural optimization, and professional practice including design of masonry, steel, reinforced and prestressed concrete structures. Professor Frangopol is currently teaching graduate courses in the fields of structural and geotechnical reliability, structural optimization and inelastic theory of structures. He is also a researcher and consultant to industry in the fields of structural reliability and bridge engineering. A Fellow of ASCE and a member of ACI and TRB, Professor Frangopol currently serves as chair or member of many committees. He has lectured at several world renowned universities and published more than 150 papers on the applications of probabilistic procedures to structural engineering. He is the editor of two books, co-author of a book on reliability of steel structures and editorial board member of three international journals.

Steven Marshall is a graduate research assistant at the University of Colorado.

Principal Investigator's Telephone Number:	(303) 492-6381
Facsimile Number:	(303) 492-7317
E-mail Address:	hearn@stripe.colorado.edu

DESIGN OF BRIDGE INSPECTION PROGRAMS FOR STRUCTURAL RELIABILITY

George Hearn, Dan M. Frangopol, and Steven Marshall
Dept. of Civil, Environmental and Architectural Engineering
University of Colorado
Boulder, Colorado 80309-0428

Period: Sept. 1994 to Aug. 1997
Sponsor: National Science Foundation

RESEARCH OBJECTIVES

The quality of condition assessments of bridges depends on the accuracy and precision of inspection methods. Limitations in inspection methods produce uncertainties in the condition of bridges. The uncertainty of condition assessment is a basis for the evaluation of inspection methods. Condition assessments are estimated both as mean values and as lower bounds consistent with the variability in performance of an inspection method. These estimates are incorporated in evaluations of bridge strength and safety. The disparity between mean values of strength and safety and lower bound values is a measure of the quality of the inspection. This is an end-use basis for measurement of performance of inspection methods. It leads naturally to requirements for high accuracy and precision in inspections for critical defects and it confirms the utility of simpler inspections where limited precision may be acceptable. For highway bridges, this is the difference between inspecting fracture critical details, and making basic serviceability assessments of decks. The measurement of performance of inspection methods makes possible the design of inspection programs. Inspection methods can be selected and applied so that goals for assurance of structural condition are met. This project studies the performance of inspection methods to develop guidelines for the selection and application of methods to achieve a reasonable level of assurance of condition. The project focuses on nondestructive test methods.

RESEARCH APPROACH

Models of the performance of nondestructive test methods are developed to study the mean values and dispersion of condition assessments. Models are developed from laboratory studies and field demonstrations of inspection methods. Models recognize limits on detection, noise in measurement, variations in performance for different levels of damage, and the influence of environmental effects such as humidity and temperature. Models of inspection methods yield the expected value and dispersion of condition assessments. These models are used together with models of deterioration in structures to determine (a) the uncertainties in structural strength and safety that can exist, and to determine (b) the impact that limitations in inspections can have on repair planning and optimization in bridge management systems.

PRODUCTS OF THE RESEARCH

Products of the research include models of the performance of inspection methods, procedures for the evaluation of significance of errors in condition assessments, and a process for the selection of inspection methods to achieve a desired level of assurance in condition assessments. The process for the selection of inspection methods can also indicate when a desired level of accuracy or precision can not be obtained.

PRELIMINARY RESULTS: PROBABILITY OF DETECTION AND PROBABILITY OF DAMAGE

For some inspection methods, such as ultrasonic inspection for flaws, models of performance focus mainly on detection. Flaw size, location, and orientation are important factors in the model of performance. For other methods, such as half-cell potential surveys, measurement of chloride ion content, and measurement of dielectric constants, models of performance must recognize the correlation of a physical quantity with damage. Such methods are limited by the response of a physical quantity to damage, and are based on a change in physical quantity that can indicate damage. The initial distribution of values of a physical quantity is important. If the initial distribution includes values in a range that might indicate damage, then the interpretation is uncertain. Moreover, the interpretation remains uncertain despite improvements in the precision of measurements.

Example - Performance of Half-Cell Potential Surveys. Half-cell potentials indicate the activity of corrosion of reinforcing steel in concrete members. Using a copper-copper sulfate half-cell, the occurrence of potentials more negative than -0.35V is taken as an indication of active corrosion. It is known that there is an abrupt change in electrical potential when corrosion of reinforcing steel begins. It is also known that initial (noncorroding) values of electrical potential exist at and beyond -0.35V. Potential distributions in corroding and noncorroding areas are different, but they overlap. Typical distributions of potentials are shown in Figure 1(a). There is a significant overlap in the probability density functions in areas of active corrosion and no corrosion. For electrical potentials in the range -0.15V to -0.35V, the interpretation is ambiguous. Corrosion might exist, or it might not. A probabilistic interpretation index for potential as an indicator of damage is proposed as

$$P(\text{Damage}(v)) = \frac{f_D(v)}{f_D(v) + f_S(v)}$$

where v is electrical potential, $f_D(v)$ is the pdf for potentials in damaged portions of the deck, and $f_S(v)$ is the pdf for potentials in sound (undamaged) deck.

The overlap in electrical potentials in damaged and in sound deck limits the ability of the half-cell method to identify areas of active corrosion. The separation of potentials can be expressed as a β factor

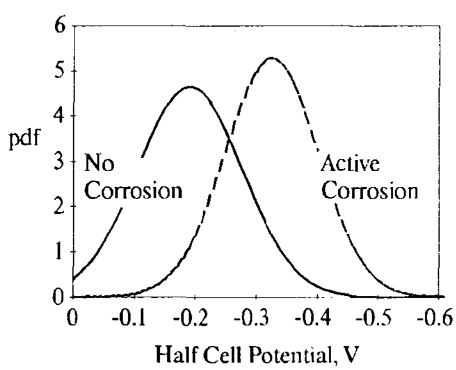
$$\beta = \frac{\mu_S - \mu_D}{\sqrt{\sigma_S^2 + \sigma_D^2}}$$

where μ_S, μ_D indicate mean values and σ_S, σ_D indicate standard deviations of potentials in sound and in undamaged regions, respectively. This β value is a useful index for the utility of a physical quantity as an indicator of damage. For comparison, interpretation curves are shown in Figure 1(b) for half-cell potentials and for fictitious pdfs of potentials with less overlap and therefor, higher β value. Notice that for higher β, the interpretation curve becomes steeper and has a smaller range of ambiguous indications of active corrosion.

Accuracy of assessment by half-cell survey is shown in Figure 2. Threshold assessment (2a) of total damage offers good agreement with real damage near 5% damage, but underestimates damage at higher levels of real damage. In addition, the threshold yields many false interpretations of individual potential readings. Many readings in areas of no corrosion are interpreted wrongly (False Damage) as are readings in areas with active corrosion (False Health). The probabilistic interpretation (2b) is calibrated to yield accurate assessments of level of damage, and can also be used to identify all readings where there is probability of damage above a selected level. The probability of damage that can be assigned using a continuous interpretation offers advantages in mapping areas of damage. In Figure 3, maps for a simulated set of potential readings are shown based on (a) threshold interpretation and (b) on continuous function interpretation.

The precision of assessment that might be obtained in half-cell surveys is a function of the number of locations where potentials are measured. Figure 4 shows how the range of assessments by half-cell method narrows as the number of locations N is increased. Notice that there is rapid narrowing of the range for increased testing at relatively low values of N, and diminishing incremental benefit for additional tests at higher values of N.

A probabilistic approach to interpretation of inspection data can offer more accurate assessments, can indicate the precision of assessments (this is also an indication the uncertainty in assessments), and can indicate the opportunity to improve the precision of assessments by additional testing.



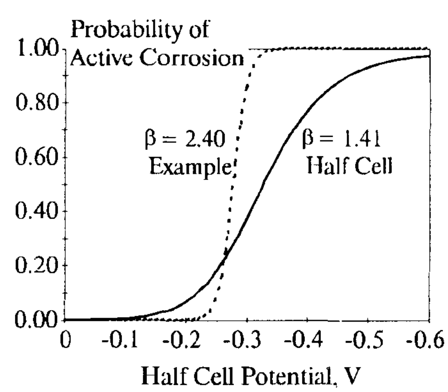
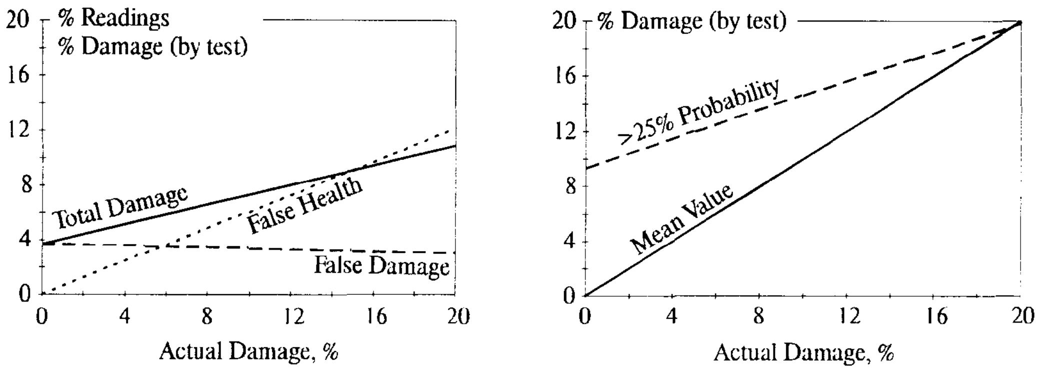
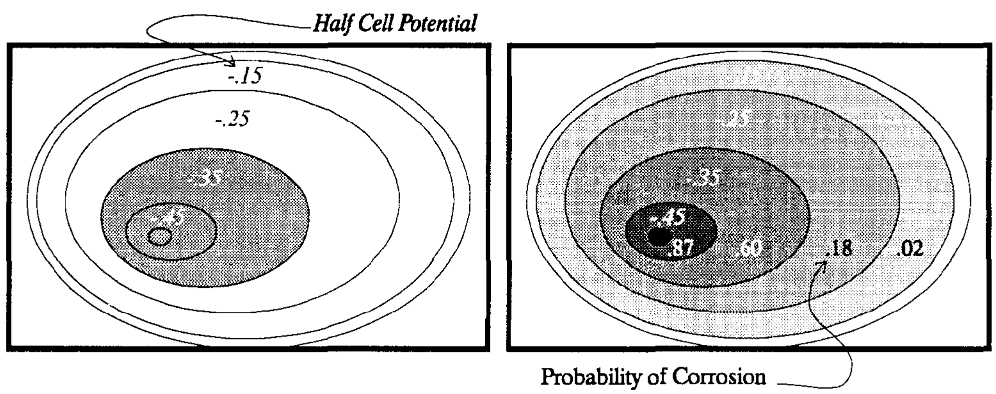


Figure 1 - Half-Cell Inspection: (a) pdfs of Electrical Potentials, (b) Probability of Damage



(a) Interpretation using -0.35 V Threshold (b) Interpretaion by Probabilty of Damage

Figure 2 - Performance of Half-Cell Inspection of Bridge Decks



(a) Threshold Interpretation (b) Continuous Interpretation

Figure 3 - Interpretation of Half-Cell Potentials: (a) Threshold (b) Continuous Function

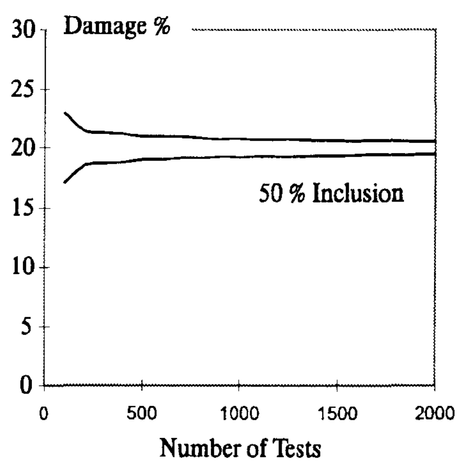


Figure 4 - Half-Cell Inspection: Range of Assessments versus Number of Tests

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Life Cycle Cost Analysis for Bridges

Author(s) and Affiliation(s):

Hugh Hawk
National Engineering Technology Corporation
North York, Ontario, Canada

Principal Investigator: Hugh Hawk

Sponsor(s): National Cooperative Highway Research Program

Research Start Date: April 15, 1996
Expected Completion Date: July 15, 1998

Research Objectives:

Transportation officials consider life-cycle cost analysis an important technique for assisting with investment decisions. The objective of this research is to develop a methodology for bridge life-cycle cost analysis (BLCCA) for use by transportation agencies. This methodology will be augmented by practical tools, including a guide manual for carrying out BLCCA and a software package (with appropriate documentation) that automates the methodology.

Expected Products or Deliverables:

Guide Manual for Bridge Life Cycle Cost Analysis (BLCCA), BLCCA software package, and BLCCA User's Manual

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Hugh Hawk received his BSCE in 1976 and his MSCE in structural dynamics in 1978 from the University of Calgary. He is currently a Senior Project Engineer with the National Engineering Technology Corporation in Toronto, Canada, responsible for the development of several bridge management systems and tools, including: Metro-North Commuter Railroad BMS; New Jersey Transit BMS; BRIDGIT, for the National Cooperative Highway Research Program; Municipal Bridge/Culvert Appraisal Data Entry System, Tri-Committee; and Railway Load Rating Analysis System, BC Rail. Mr. Hawk is also responsible for the preliminary and final design of several projects, including: Humber Bridges complex on the Gardiner Expressway, Metro Toronto Roads and Traffic; Dundas Street Bridge, City of Trenton; Brimley Road over 401, City of Scarborough; DVP Ramp under CP Rail - Jacked Tunnel Structure, Metro Toronto Roads and Traffic; York Boulevard High Level Bridge - Rehabilitation, Region of Hamilton-Wentworth; Thames River Crossing and Horton Street extension, City of London, Ontario; Widening of 49th Avenue and Gaetz Avenue Bridges, City of Red Deer, Alberta; 12 Street NE over Memorial Drive, City of Calgary; 16th Avenue over 14th Street, City of Calgary and Glenmore Trail/14th Street Interchange, City of Calgary. In addition, he has worked as an Independent Appraiser on a number of Canadian and International projects and as a Contractor's Designer for several alternative design projects.

Principal Investigator's Telephone Number: (416) 441-4111
Facsimile Number: (416) 441-4131
E-mail Address:

LIFE CYCLE COST ANALYSIS FOR BRIDGES
NCHRP PROJECT 12-43

Hugh Hawk, P.Eng

National Engineering Technology Corporation
133 Wynford Drive, North York, Ontario, Canada M3C-1K1

Research Objectives

Transportation officials consider life-cycle cost analysis an important technique for assisting with investment decisions. The objective of this research is to develop a methodology for bridge life-cycle cost analysis (BLCCA) for use by transportation agencies. This methodology shall be augmented by practical tools, including a guidance manual for carrying out BLCCA and a software package (with appropriate documentation) that automates the methodology.

Research Approach

Background: Several recent legislative and regulatory requirements recognize the potential benefits of life-cycle cost analysis and call for consideration of such analyses for infrastructure investments, including investments in highway bridge programs. However, a commonly accepted, comprehensive methodology for bridge life-cycle cost analysis (BLCCA) currently does not exist. Therefore, research is needed to provide bridge engineering professionals with a methodology for the determination of life-cycle costs for bridges to aid them in selecting the most appropriate bridge improvement alternatives.

Life Cycle Cost Analysis: The cost to an agency for a bridge is seldom a one-time cost. It is a long-term, multi-year investment. Throughout its useful life, a bridge requires periodic maintenance and possibly repair or rehabilitation actions. At the end of its useful life, the bridge is replaced.

The goal of life cycle cost analysis is to identify the best repair and functional improvement strategy for a given bridge. For this analysis, it is necessary to identify and compare feasible alternative strategies, referred to as life-cycle activity profiles (LCAPs). Each LCAP represents a series of repair and improvement actions expected to occur over the life of a structure.

The present value of all costs associated with each LCAP is calculated and used as a basis for economic comparison. The costs considered include agency costs for the various repair or improvement actions as well as user costs associated with accidents or detours due to load capacity deficiencies, vertical clearance deficiencies or bridge width deficiencies.

Current bridge life cycle costing methodologies do not consider obsolescence, risk and uncertainty. These will be addressed during the course of the research.

Obsolescence: The calculation of life cycle costs depends on the choice of a distinct period over which operations and maintenance costs are accrued, discounted, and compared with capital costs. This time period is often termed the planning time-horizon or design-service life. The specific time period selection, is generally decades-long. Over the course of as little as ten to thirty years, obsolescence may reduce substantially the value of a bridge.

Obsolescence is a condition of being antiquated, old-fashioned, or out-of-date. An obsolete item is not necessarily broken, worn out, or otherwise dysfunctional, although these conditions may underscore its obsolescence. Rather, the item simply does not measure up to current needs or expectations. Obsolescence results when there is a change in the requirements or expectations. Changes in structural design standards to reduce the risks of earthquake failure and mitigate the consequences of such a disaster render many older structures effectively obsolete since they no longer comply. In most cases things that are obsolete continue to function but at levels below contemporary standards.

A number of factors can cause obsolescence:

- Technological changes can influence the scope or levels of service a bridge is to provide.
- Regulatory changes impose new requirements on infrastructure.
- Economic or social changes can substantially alter the demands placed on infrastructure.
- Changes in values or behaviour can similarly alter demands but are more difficult to foresee. For example, a societal commitment to private auto travel spurred removal of street railways in most urban areas. As a result, some heavily congested bridges now carry fewer people than they did when they were first built.

Forecasting of obsolescence is difficult at best, and certainly beyond the realm of current life cycle cost analysis models. However, effective life cycle cost management should include explicit consideration of obsolescence.

Risk and Uncertainty: Engineering risks and uncertainty are generally related to the interaction of environmental factors and bridge characteristics causing partial or complete loss of functionality and/or collateral damage. The most easily recognized sources of engineering risk and uncertainty are:

a) Condition related reduction in load capacity and/or life.

 As a bridge deteriorates and undergoes repeated load cycles, the travelling public is exposed to increasing risk and/or potential inconvenience. A bridge in poor condition may require posting, which has a direct effect on user costs, or be at a higher risk of failure. A partial or total bridge failure can cause substantial hardship, and thus increasing the risk of failure must carry a cost and be considered in Life Cycle Cost Analysis. Direct and indirect costs could include replacement of the bridge, detouring of traffic and associated traffic congestion, loss of life and thus substantial liability, effects on area businesses serviced by the facility; etc.

b) Seismic vulnerability

Particular types of structures carry a higher risk of damage due to seismic events. A structure's seismic vulnerability and the uncertainty in seismic events needs to be addressed. The risk of damage from a seismic event needs to be evaluated against the cost of mitigating that damage. Seismic events also tend to be unusual in that a single event can cause damage to many structures and compromise the entire road network. The value of providing, at least some, post-disaster emergency routes must be recognized. The user costs associated with earthquake damage are difficult to assign to individual structures. It is an entire road network or even an economy based issue. Seismic vulnerability is a high profile issue, yet engineers are ill-equipped to economically evaluate their decisions.

c) Bridge Scour

A recent study in New York State has shown that scour is, by far, the primary cause of bridge structural failure in that state. Engineers need a systematic way of evaluating the susceptibility of a bridge to scour, the probability of events causing scour, and the consequences (damage or failure). This needs to be balanced against the cost of mitigating measures. In some cases, it may be warranted to replace a particularly scour-susceptible bridge, even if the structure is in good condition, if the consequences of failure are severe, the risk of failure is unacceptably high, and no other mitigating measures are available.

d) Overloads

Live loads for which bridges are designed are upper bound representations of typical traffic found on the nation's roads. Changing trucking regulations, unscrupulous operators, and steadily increasing loads are putting the nation's bridges at a greater risk of being overloaded. Overloads can cause direct damage, changes to structural behaviour (through plastic hinge formation causing permanent moment re-distribution, deck cracking, connection slippage, etc.) or cumulative damage (fatigue, etc.). Similarly to point a) above, this can lead to direct agency and user costs as well as an increased risk of future damage or failure.

Another area of risk concerns public safety and agency liability. Substandard bridge railing, guiderail, transitions, end treatments, etc. can expose the agency to litigation even if the design of such appurtenances was appropriate when the bridge was constructed. Even a policy which is as seemingly benign as "All new bridges shall have concrete barriers" may, by inference, condemn all steel railing as being deficient in the eyes of a court. Exposure to litigation is obviously an issue which must be addressed.

Uncertainties can be considered to contribute to the risk of making the wrong preservation decision for a particular bridge. Uncertainties can apply to many parameters such as the short-term and long-term discount rates, rates of traffic growth, changes to road usage, value of litigation awards, costs of repair, effectiveness of repairs, timing of repairs, construction of competing facilities, changes to load permits, definition of "substandard" geometry, etc.

Generally, these uncertainties are two-fold: the current value is uncertain and the future trends are uncertain. For example, it may be appropriate to express the current cost of a particular deck repair as being a standard normal function with a mean of $100 per square metre and a standard deviation of $20 per square metre and that the cost of a similar repair in the future will increase by, on average, 5% per year with a standard deviation of 1%. Stated another way, the compounding of uncertainty makes the value more and more uncertain with time.

<u>Software Package:</u> A software package will be developed for use by transportation agencies and will be architected so that it satisfies the following needs:

- It can be used by both small and medium sized state and local bridge authorities, with or without NBI data.

- The proposed micro-computer based software should be able to batch load existing information or connect to the corporate database.

- The software package must be capable of organizing potentially large amounts of data in a micro-computer environment.

- The software must be thoroughly tested and de-bugged if it is to be accepted for use by the bridge management community.

- Companion documentation such as a Guidance Manual and User Manual must be of benefit to the bridge community as stand-alone documents.

We are currently developing the scope and specifications of the Bridge Life-Cycle Cost Analysis Software and will submit a report to NCHRP for its review and approval in about six months.

Expected Products or Deliverables

The objective of this research is to develop a methodology for bridge life-cycle cost analysis (BLCCA) for use by transportation agencies. This methodology shall be augmented by practical tools, including a guidance manual for carrying out BLCCA and a software package (with appropriate documentation) that automates the methodology.

Preliminary Results

None

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS RESEARCH ABSTRACT

Title: Life-Cycle Engineering for Bridge Decks

Author(s) and Affiliation(s):

David Veshosky, Lafayette College, Easton, PA
Stephanie J. Wagaman, Carl R. Beidleman, and John L. Wilson
Lehigh University, Bethlehem, PA

Principal Investigator: David Veshosky

Sponsor(s): Advanced Technology for Large Structural Systems Engineering Research Center and Lehigh University

Research Start Date: 1994
Expected Completion Date: 1998

Research Objectives:

The objectives of this research are to determine the feasibility of an approach to bridge life-cycle analysis based on developing models to support specific decisions, and to develop a prototype life-cycle engineering model to support decisions concerning design of new and replacement decks.

Expected Products or Deliverables:

A model for life-cycle engineering of bridge decks, better understanding of bridge economies, a list of decisions for additional life-cycle engineering models, and specifications for data systems to support such models.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

David Veshosky is Assistant Professor of Civil and Environmental Engineering at Lafayette College and a Research Associate at Lehigh University's Engineering Research Center for Advanced Technology for Large Structural Systems (ATLSS). He has a BSCE from Catholic University and a Ph.D. in Economics and Business from Lehigh University.

Stephanie J. Wagaman is a Research Engineer at Lehigh's ATLSS Center. She has a B.S. in Computer Science from Cedar Crest College and an M.S. in Computer Science from Lehigh University.

Carl R. Beidleman is DuBois Professor of Finance at Lehigh University and a Research Associate at ATLSS. He has a BS in metallurgical engineering from Lafayette College and a Ph.D. from The University of Pennsylvania's Wharton School.

John L. Wilson is Professor of Civil Engineering at Lehigh University and Associate Director of Research for the ATLSS Center. He has a BSCE from Tuffs University, MS in engineering mechanics from Yale University, and a Ph.D. in civil engineering from the University of Pittsburgh.

Principal Investigator's Telephone Number: (610) 250-5424
Facsimile Number: (610) 250-5059
E-mail Address: veshoskd@lafayette.edu

Life-cycle Engineering for Bridge Decks

David Veshosky[1], Stephanie J. Wagaman[2], Carl R. Beidleman[3], and John L. Wilson[4]

[1]Department of Civil and Environmental Engineering
Lafayette College, Easton, PA 18042
[2,3,4]ATLSS Engineering Research Center
Lehigh University, Bethlehem, PA 18015

Research Objectives

Lack of reliable, usable models has hindered implementation of the ISTEA mandate that life-cycle costs be considered in award of bridge contracts on federally-funded projects, and resulted in lost opportunities to improve the allocation of increasingly constrained funds for infrastructure preservation and improvement. While the concepts of life-cycle analysis are relatively simplistic, operationalizing those concepts for highway bridges has proven problematic. The objectives of this research are to determine the feasibility of an approach to bridge life-cycle analysis based on developing life-cycle models to support specific bridge engineering decisions, and to develop a prototype model to support the decision concerning design of new and replacement bridge decks.

Research Approach

Background: Recognition of the need to allocate public sector resources more effectively, as reflected in the mandate in the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 that highway and bridge contracts on federally-funded projects consider life-cycle costs, has resulted in increased attention to life-cycle analysis of bridges. Some Requests for Proposals for bridge engineering services have included requirements that life-cycle analysis be conducted. However, there do not appear to be any specific, practical guidelines or reliable, useful models available to assist bridge engineers in performing life-cycle analysis.

Although the concepts of life-cycle analysis are relatively simplistic, and life-cycle costs have been used as the basis of decision-making in some industries for many years, operationalizing the concepts for highway bridges has proven problematic. While development of life-cycle cost models in industries such as electronics, for example, is relatively straightforward - data on costs of a large number of identical, short-lived products are used to develop a probabilistic model which can be applied in design, maintenance, repair, and replacement decisions - bridges are unique structures with long lives and extensive maintenance, renewal, and replacement alternatives; reliable data, particularly on bridge maintenance and renewal costs, are difficult to acquire on a systematic basis.

Numerous authors have presented conceptual discussions of bridge life-cycle analysis (Bettigole, 1993; Yanev, 1993; Nickerson and Veshosky, 1992; Novick, 1990). Some studies have involved development of highway, bridge and pavement life-cycle analysis models, but few, if any, have included actual data (Al-Subhi and Johnston, 1988; Hyman and Hughes, 1983). The two primary, and related, obstacles to applying life-cycle analysis to bridges have been difficulty in developing a feasible, reliable approach, compounded by lack of systematic, readily accessible data which could be used to evaluate the feasibility and reliability of alternative approaches.

Difficulties in identifying a feasible approach for bridge life-cycle analysis reflect the broad and unstructured nature of bridge engineering. A variety of approaches have been suggested, differing in terms of the level at which life-cycle analysis should be applied (e.g., a network of bridges, individual bridges, major bridge subsystems, bridge elements, etc.) and when, during the life of a bridge, it should be applied (e.g., initial construction, repairs, rehabilitation/replacement decisions, etc.). An additional problem relates to who should perform bridge life-cycle analysis (i.e., owners or consultants) and, if performed by consultants, how they should be compensated.

<u>Approach:</u> The approach being employed at Lehigh University's Engineering Research Center for Advanced Technology for Large Structural Systems (ATLSS) is based on life-cycle analysis as representing decision support, to provide useful information to bridge engineers faced with decisions in design, maintenance, renewal, and replacement. Such decisions are currently made regularly by bridge engineers, with life-cycle considerations either ignored or considered implicitly based on experience and judgment. Providing bridge engineers with a relatively simple and practical means of obtaining objective, quantitative information about the life-cycle implications of decisions they face should result in improved decision-making.

This approach, termed life-cycle engineering, is premised on identifying specific engineering decisions which occur throughout the life of a bridge and significantly influence cost and performance. To determine whether this approach is feasible, a prototype model is being developed. The protoype is intended to provide support to bridge engineers deciding among alternatives for a new or replacement deck. The alternatives considered in the model include a bare deck with uncoated reinforcing bars; a bare deck with epoxy-coated rebar; a deck with a protective overlay and uncoated rebar; and a deck with a protective overlay and coated rebar. Users can also analyze additional alternatives.

The model calculates costs both to owners and users, annualized over the life of the bridge. Owner costs include engineering, construction, maintenance, renewal, and replacement. User costs include the costs of delays, detours and accidents.

The model has been designed so that the input which is required of the user consists of information which should be available to the bridge engineer. Required input includes bridge-specific information such as average daily traffic, average daily truck traffic, expected life of the bridge, and detour length and route. In addition, the user has the option of inputting data on the expected life and construction, maintenance, renewal, and replacement costs of each deck alternative; for the four alternatives included in the prototype model, default values are provided for these inputs.

Efforts thus far have focused on developing a conceptual model of the decision and collecting data which could be used to operationalize the conceptual model. Data was collected through review of relevant literature as well as surveys of and interviews with bridge engineering practitioners, including owners and consultants. Operationalizing the model has involved estimating functional relationships between costs, performance, inflation, and other factors, and developing default values for inputs. The default values have been used to test the reliability and feasibility of the model. While the prototype is currently operational, the relationships and values used thus far in calculating the life-cycle implications of maintenance and renewal costs have been somewhat simplistic. Current efforts focus on improving those relationships and values.

Bridge Management Systems: The federal government has also mandated that states implement bridge management systems (BMS) to manage their bridge inventories. The most popular BMS is Pontis, an AASHTOWare product that was developed for the Federal Highway Administration by Cambridge Systematics. Several states have developed their own BMS, but most are at least considering Pontis. While implementation of BMS is likely to facilitate bridge life-cycle analysis, BMS do not appear well-suited to performing such analysis.

Bridge management systems provide capabilities to improve data management, through a structure for electronic collection, storage, retrieval, and manipulation of construction, inspection, maintenance, and renewal data. On a system level, involving all the bridges in the inventory of a state or a district within the state, a BMS allows a user to determine the budget required to maintain a specified level of service; alternatively, a BMS can be used to allocate constrained funds most effectively within a system.

However, BMS do not appear to be well-suited to performing project-level analysis for a specific bridge. The unit costs for renewal and replacement which are suitable for system-level decisions appear to be, at least at present, too generic for project-level decisions. For example, unit costs may vary widely within a state, between urban and rural areas, as well as seasonally and according to other factors. While use of system-wide unit costs is acceptable for system-level decisions, more specific costs should be used in project-level analysis. As implementation of BMS becomes widespread, reliable, systematic data on maintenance and renewal costs should become available, and can be used in bridge life-cyle engineering.

Expected Deliverables

Several deliverables are expected from this research. It is anticipated that the prototype model for life-cycle engineering of bridge decks will be made available to bridge engineers and owners. In addition, the model will be used to analyse the relative significance to life-cycle costs of construction, maintenance and renewal costs, and the sensitivity of the model to variations in input data, contributing to a better understanding of bridge economics.

The research is also expected to generate a list of decisions which represent opportunities for development of additional bridge life-cycle engineering models, and specification of the data required to apply the prototype and possible additional models. The intent is that the specifications will address the interface between life-cycle engineering models and BMS, specifically Pontis.

Preliminary Results

Preliminary results of the research are positive. A working version of the prototype has been developed. While the current version employs a somewhat simplistic model of maintenance and renewal costs, improvements are underway.

In addition, based on discussions with a variety of bridge engineering practitioners, we feel that results thus far support the basic approach employed in this research and proposed for bridge life-cycle analysis: such analysis should be considered to represent decision support, used by bridge engineers in evaluating the life-cycle cost and performance implications of decisions in design, maintenance, renewal, and replacement.

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FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Establishing Project Level Maintenance Policies for a Network Level BMS

Author(s) and Affiliation(s):

Teresa M. Adams, Joseph Barut and Robert Sianipar
Department of Civil and Environmental Engineering
University of Wisconsin-Madison, Madison, WI

Principal Investigator: Teresa M. Adams

Sponsor(s): Wisconsin Department of Transportation

Research Start Date: August 1, 1994
Expected Completion Date: June 30, 1996

Research Objectives:

The objectives of this study are to adapt and evaluate the Pontis network level BMS for multiperiod planning in Wisconsin, establish condition state transition probabilities for maintenance actions, estimate cost of bridge maintenance actions, evaluate the cost-effectiveness of alternative maintenance actions, and to evaluate the project level effectiveness of recommended network level maintenance policies.

Expected Products or Deliverables:

Recommended cost and condition state transition probabilities for maintenance actions on 25 bridge elements in Wisconsin; methods for evaluating the quality of estimated cost data; the cost effectiveness of alternative maintenance actions; and the sensitivity of network level policies to variations in cost, project factors, and condition state transition.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Teresa M. Adams conducts research in the area of engineering management, construction, and civil infrastructure systems. She has expertise in applied systems analysis, database management, information engineering and geographic information systems. During the past three years, Dr. Adams has worked closely with Wisconsin State Bridge and Maintenance Engineers toward meeting ISTEA regulations for implementing a multiperiod network level BMS. Additional bridge management projects include development of a database program for generating pre-printed bridge inspection forms for the Wisconsin DOT. Dr. Adams is an Associate Professor in the Construction Engineering and Management program in the Department of Civil and Environmental Engineering at the University of Wisconsin-Madison. She has a Ph.D. and MSCE from Carnegie Mellon University and a BSCE from the University of Pittsburgh. She is an Associate Editor of the ASCE *Journal of Computing in Civil Engineering.*

Robert Sianipar is a Ph.D. candidate in the Department of Civil and Environmental Engineering at the University of Wisconsin-Madison. Mr. Sianipar holds Master Degrees in Civil Engineering and Engineering Mechanics, both from the University of Wisconsin-Madison. His research topic is Interpretation and Evaluation of a Network Level Bridge Management System for the State of Wisconsin. Mr. Sianipar has experience as a Project Engineer on infrastructure projects in Batam Island, Indonesia. He was Project Engineer for the construction of the 2,500 meter runway of the Hang Nadim Airport and for the construction of roads and bridges in the Batam Central Business District.

Joseph Barut is a MSCE candidate at the University of Wisconsin-Madison. He holds a BSCE from the University of California-Davis. Mr. Barut has engineering experience in hydrologics and hydraulic modeling.

Principal Investigator's Telephone Number:	(608) 262-5318
Facsimile Number:	(608) 262-5199
E-mail Address:	adams@engr.wisc.edu

ESTABLISHING PROJECT LEVEL MAINTENANCE POLICIES FOR A NETWORK LEVEL BMS

Teresa M. Adams, Joseph Barut, and Robert Sianipar

Department of Civil and Environmental Engineering
University of Wisconsin-Madison, Madison, WI 53706

Research Objectives

The primary objectives of the research are to establish model parameters and to evaluate the performance of the Pontis network level bridge management system (BMS) for multiperiod planning in the State of Wisconsin. The research concentrates on two parameters of the optimization models: probabilities of transitioning from one condition state to another, and cost of MR&R actions.

Network level recommendations of the BMS are based upon network level behavior of bridge elements and are independent of project level parameters. However, cost of maintenance actions vary according to project level factors such as ADT, access, and project size. Other objectives of the research are to evaluate the cost-effectiveness of alternative maintenance actions, and to evaluate the project level effectiveness of the recommended network level maintenance policies.

Research Approach

Background: Current computerized bridge management systems such as Pontis and BRIDGIT employ network level, multiperiod optimization analysis. Successful adoption of a network level BMS requires dramatic changes in the practice of bridge management because the system requires the planning, engineering, and maintenance activities to be integrated more closely than ever before. Furthermore, the BMS recommendation are based upon user parameters and modeling approaches heretofore not used in bridge management. Thus, the necessary data are simply not available and practitioners are generally not familiar with the mechanisms and assumptions of the underlying models.

Overall Approach: The overall research approach involves four tasks: data collection, evaluation of data quality, analysis and simulation.

The scope of the study includes 25 bridge elements, 27 MR&R (maintenance, repair, and rehabilitation) actions, and 21 project scenarios. The bridge elements were selected because of their significance in the Wisconsin network. The selection was based upon the number of elements in the network, the number of bridges with at least one element, and the average cost of the maintenance actions on the element. The final set of 25 elements satisfies three informal criteria: 1) elements with similar MR&R actions were eliminated, 2) a variety of deck, superstructure and substructure element types were included, and 3) element with alternative MR&R actions were included. Each of the 27 MR&R actions is associated with one or more elements. High and low cost project scenarios were identified for the MR&R actions. Each of the 21 project scenarios is associated with one or more MR&R actions.

Data Collection: As a starting point for the implementation of Pontis in Wisconsin, cost of MR&R actions and element transition probabilities were adopted from the Minnesota DOT. These data are estimates. In this research, the Minnesota estimates were updated through elicitation from Wisconsin experts. Two questionnaires were developed for data collection.

An MR&R cost questionnaire was sent to the eight districts of the Wisconsin DOT. The objective of the questionnaire is to collect estimates of ordinary costs of MR&R actions and the range of the costs under extreme project scenarios. Extreme scenarios correspond to expected high and low unit cost of an MR&R action. For example, the replace overlay MR&R action is most costly when an exotic quick setting material is used, the ADT is high, and the project is small. At the other extreme, the MR&R action is least costly when an asphaltic, conventional material is used, the ADT is low, and the project is large.

Standard definitions of bridge elements provide a basis for consistent data collection. The standard definition describes the physical element size for which the unit MR&R cost is estimated. For example, the standard size for a concrete deck element is 200 feet long and 40 feet wide. Standard element definitions for Wisconsin correspond to the most common element size, not the average element size. Standard definitions of 25 elements in Wisconsin were developed for the the questionnaire.

Transition probabilities describe the deterioration behavior of bridge elements. The transition probabilities for the initial implementation of Pontis are estimated by bridge engineers. Observed deterioration statistics gathered by field inspection are used to update the initial expert estimates. In each inspection cycle, new field data updates the probabilities determined in the previous cycle.

As a part of the regular updating procedure in Pontis, the transition probability data evolve to reflect deterioration trends in Wisconsin. The updating procedure is based upon Bayesian probability theory. In some future planning period, the effects of the updating become minimal. At that time, the updated transition probabilities are stable and reliably represent actual deterioration rates. The probabilities for the do-nothing actions evolve faster, because they have the greatest number of observations in the historical database.

A transition probabilities questionnaire was sent to the eight districts of the Wisconsin DOT. The objective of the questionnaire is to collect collect transition probabilities for MR&R actions on 25 bridge elements. Experts are asked to consider the action taken on 100 elements in a particular condition state. They are asked to estimate the percentage of the 100 element in each condition state immediately after the action is taken. At this time, the questionnaire have not been returned.

Evaluation of Data Quality: Raw data on the cost of MR&R actions were evaluated for consistency among the districts. The coefficient of variation was used to measure the clustering tendency in cost estimates for each maintenance action and to allow direct comparison of cost estimates for one maintenance action to those for another. Of the 369 estimated costs, 17% had a coefficient of variation greater than 0.3, which means the standard deviation of the estimates is ±30% of the mean estimate. For these cases, the raw data were evaluated and outliers were eliminated. For the adjusted data, the maximum coefficient of variation is 0.29.

Analysis and Simulation: The analysis of variance was used to determine whether there is statistically significant evidence of variation in estimates of MR&R costs. The data were stratified by element and maintenance action in each cost scenarios. Based upon the analysis, particular elements and maintenance actions with significant variation in the mean estimates can be identified. The one-way analysis of variance indicates whether the variation in MR&R costs for an element is greater than the variation among all elements. Similarly, the analysis indicates whether the variation in cost estimates for an MR&R action is greater than the variation among all MR&R actions.

The transition probabilities for each element can be evaluated several ways. First, average change in condition state for each maintenance action will be computed. Expected results are that more significant actions cause greater average changes. Deviations from the expected result indicate possible anomalies in the transition probability data.

Second, the cost effectiveness of alternative MR&R actions will be evaluated. The cost effectiveness is measured in terms of cost per unit condition state change. The cost per unit condition state change is defined as the ratio of the marginal product to the marginal cost. It is computed by dividing the cost of an action by the average change in condition state is produces. For two alternative MR&R actions, the most cost effective has the lowest cost per unit change. The cost per unit change should follow an increasing trend for optimal actions in increasing condition states. The increasing trend is intuitively reasonable and in keeping with the motivation for preventive maintenance.

Simulation and sensitivity analysis evaluates Pontis results due to variations in MR&R cost and transition probabilities. The variation in MR&R costs reflects the range due to high and low cost project scenarios. The variation in transition probabilities reflects the uncertainties in the estimated rates of deterioration. The output to be evaluated are the recommended MR&R actions, the recommended MR&R policy and the steady state condition. The evaluation will lead to identification of maintenance policies that can be optimized according to high and low cost project scenarios.

Expected Products or Deliverables

The primary results of this research are recommended costs and condition state transition probabilities for MR&R actions on 25 bridge elements in Wisconsin. As a result of the analysis of MR&R costs, elements and maintenance actions with significantly high or low variations in cost will be identified and a range of cost will be determined according to project scenarios. The analysis of transition probabilities will identify optimal policies that are sensitive to variation in MR&R costs under high and low cost project scenarios.

In addition, the research will produce methods for evaluating the quality of estimated cost data; the cost effectiveness of alternative maintenance actions; and the sensitivity of network level policies to variations in cost, project scenarios, and condition state transition.

Preliminary Results

The variation in cost estimates within elements and MR&R actions is less than the variation among all the elements and all the MR&R actions, respectively. This indicates that, for certain MR&R actions, the estimated cost is more reliable than for others. Similarly, for certain elements, the estimated cost of maintenance is more reliable than for others.

For alternative MR&R actions, the cost per unit condition state change is a reliable measure of cost effectiveness. The actions with higher cost per unit change will never be included in the optimal maintenance policies.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Fatigue Impacts on Bridge Cost Allocation and Truck Size and Weight Restrictions

Author(s) and Affiliation(s):

Jeffrey A. Laman, Department of Civil and Environmental Engineering
Thomas E. Boothby, Architectural Engineering Department
The Pennsylvania State University, University Park, PA

Principal Investigator: Jeffrey A. Laman

Sponsor(s): Federal Highway Administration

Research Start Date: January 1996
Expected Completion Date: August 1996

Research Objectives:

The cost allocation model currently under development by the Federal Highway Administration must consider costs attributable to various fleet mixes of weight and vehicle classes based on the costs of repair, rehabilitation, replacement, and safety improvement. This study will develop a tool to apportion damage due to fatigue in steel bridges and cumulative damage to concrete decks as a function of the truck class, weight group, and highway class. The current cost allocation model contains 20 vehicle classes with 30 weight groups. This matrix of vehicle class and weight groups is considered on each of twelve functional classes of highway. A sample of bridges will be selected from the National Bridge Inventory to represent steel bridges for the simulations.

Expected Products or Deliverables:

This work will result in a general tool which is capable of simulating the movement of any number of trucks from a predefined database over any number of bridge structures. The bridge structures must be of a type that can be modeled by the semi-continuum method. Results will be immediately transferable to a highway cost allocation study.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Jeffrey A. Laman is Assistant Professor of Civil Engineering at The Pennsylvania State University in the Structural Engineering Division and a Faculty Associate at the Pennsylvania Transportation Institute. He has more than ten years of professional structural engineering consulting experience in addition to his research experience for the Michigan and US Departments of Transportation and the Great Lakes Center for Truck and Transit Research. His background includes experience in structural design, highway truck load modeling, field testing of large structures, and reliability analysis and code calibration. Dr. Laman's professional experience ranges from design of heavy industrial facilities to institutional and commercial facilities. His research experience has involved field testing of bridges and development of bridge live load and fatigue models. He is a registered Professional Engineer in Michigan and active in ASCE, the Transportation Research Board, American Institute of Steel Construction, and ACI.

Thomas E. Boothby is Assistant Professor of Architectural Engineering at The Pennsylvania State University, a position he has held for the last four years. Previously, he was a post-doctoral researcher at the University of Nebraska-Lincoln, and a structural designer for various consultants in Albuquerque, NM. He has earned a Ph.D. in civil engineering from the University of Washington, an MSCE from Washington University in St. Louis and a BA in Architecture from Washington University. His research specialty is assessment, maintenance, repair, and rehabilitation of old and historic bridges and other structures.

Principal Investigator's Telephone Number:	(814) 863-0523
Facsimile Number:	(814) 863-7304
E-mail Address:	jlaman@psu.edu

FATIGUE IMPACTS ON BRIDGE COST ALLOCATION AND TRUCK SIZE AND WEIGHT RESTRICTIONS

Jeffrey A. Laman[1], P.E. and Thomas E. Boothby[2], P.E.

[1]Department of Civil and Environmental Engineering
The Pennsylvania State University; University Park, PA 16802

[2]Architectural Engineering Department
The Pennsylvania State University, University Park, PA 16802

Research Objectives

The cost allocation model under development by the Federal Highway Administration (FHWA) must consider costs attributable to various fleet mixes of weight and vehicle classes based on the costs of repair, rehabilitation, replacement, and safety improvement. This study will develop a tool to apportion damage due to fatigue in steel bridges and cumulative damage to concrete decks as a function of the truck class, weight group, and highway class. The current cost allocation model contains 20 vehicle classes with 30 weight groups. This matrix of vehicle class and weight groups is considered on each of twelve functional classes of highway. A sample of bridges will be selected from the National Bridge Inventory (NBI) to represent steel bridges for the simulations.

Research Approach

Background: Bridge cost allocation determines the consumption of infrastructure by the various classes of users as a function of the resources required to construct and maintain the system. The purpose of this study is to assess users equitably on the basis of the expenditures required to support the users on the infrastructure. Costs can be allocated on the basis of user classes, by size and weight, or by other distinctions.

The identification of bridge deficiencies under the current FHWA Highway Cost Allocation Study (HCAS) and truck size and weight (TS&W) scenarios encompasses basic types of structural behavior. These basic structural behaviors are manifested in a number of distinct bridge and component types. Structural behavior includes: (1) fatigue strength under repeated loading; (2) static strength under a single event, including the effect of impact; and (3) cumulative damage, deterioration, and the interaction of environmental effects, particularly as it relates to reinforced concrete deck slabs. The main bridge components to be considered in this study are deck slabs and AASHTO steel fatigue details.

Under-capacities in fatigue have different economic consequences from under-capacities of static strength. Fatigue life expectancy is an exponential function of the induced stress range, (e.g., a 10% increase in stress range will result in a 25% reduction in the expected life of the component, assuming no increase in the average daily truck traffic (ADTT)). This study will utilize the stress-life approach modified as a special case of the linear elastic fracture mechanics approach as defined Saklas, et al. 1988.

Depending on the endurance limit at existing AASHTO designed details, changes in the mix of weight and vehicle classes may result in accelerated fatigue damage of the bridge inventory. Although past studies have indicated that fatigue damage accounts for a relatively small portion of current overall bridge maintenance concerns, a relatively small increase in truck loads could significantly change this picture.

<u>Methodology</u>: Many different bridge formulas, tridem-axle load limits, GVW limits, and risk criteria are available for consideration. Candidate configurations will be provided from both FHWA and the trucking industry for analysis and evaluation. Of interest is bridge response to the various TS&W criteria, particularly with regard to fatigue and cumulative damage. The response (both long and short term) of the bridge is a function of the bridge type and materials. Steel girder bridges, concrete girder bridges, steel truss bridges, slab bridges, and other types will exhibit different long and short term responses to a given load spectrum resulting from a revised TS&W criteria.

This study for fatigue and cumulative damage will involve the following tasks:

Task 1: Review the available literature pertaining to cumulative damage of deck slabs subject to load and corrosive environments. Based on the current state-of-the-art a methodology will be developed to assess the relative damage to deck slabs as a result of repetitive loading, load magnitude, overloads, and environmental factors. Past methods of assigning damage responsibility may be improved due to recent research in this area.

Task 2: Review the available literature pertaining to fatigue evaluation of bridge details. The methodology developed by Saklas et al. (1988) will be reviewed and updated to the current state-of-the-art of bridge fatigue evaluation. The same fundamental approach will be followed; that being the basic premise of the AASHTO Bridge Design Specification. AASHTO follows the stress-life model, which has been shown to be a special case of the linear elastic fracture mechanics (LEFM) model under certain limiting conditions (Saklas et al, 1988).

Task 3: Establish the parameters for evaluation and allocation of damage to the defined vehicle types. AASHTO defined fatigue details and corresponding specifications are based on the equivalent stress concept, or root mean cube equation which converts the variable amplitude stress ranges to equivalent constant amplitude stresses. This approach forms the basis of the relative fatigue damage distribution by vehicle type established by Saklas, et al. (1988), and therefore follows the basic philosophy of the AASHTO fatigue specification for steel bridges.

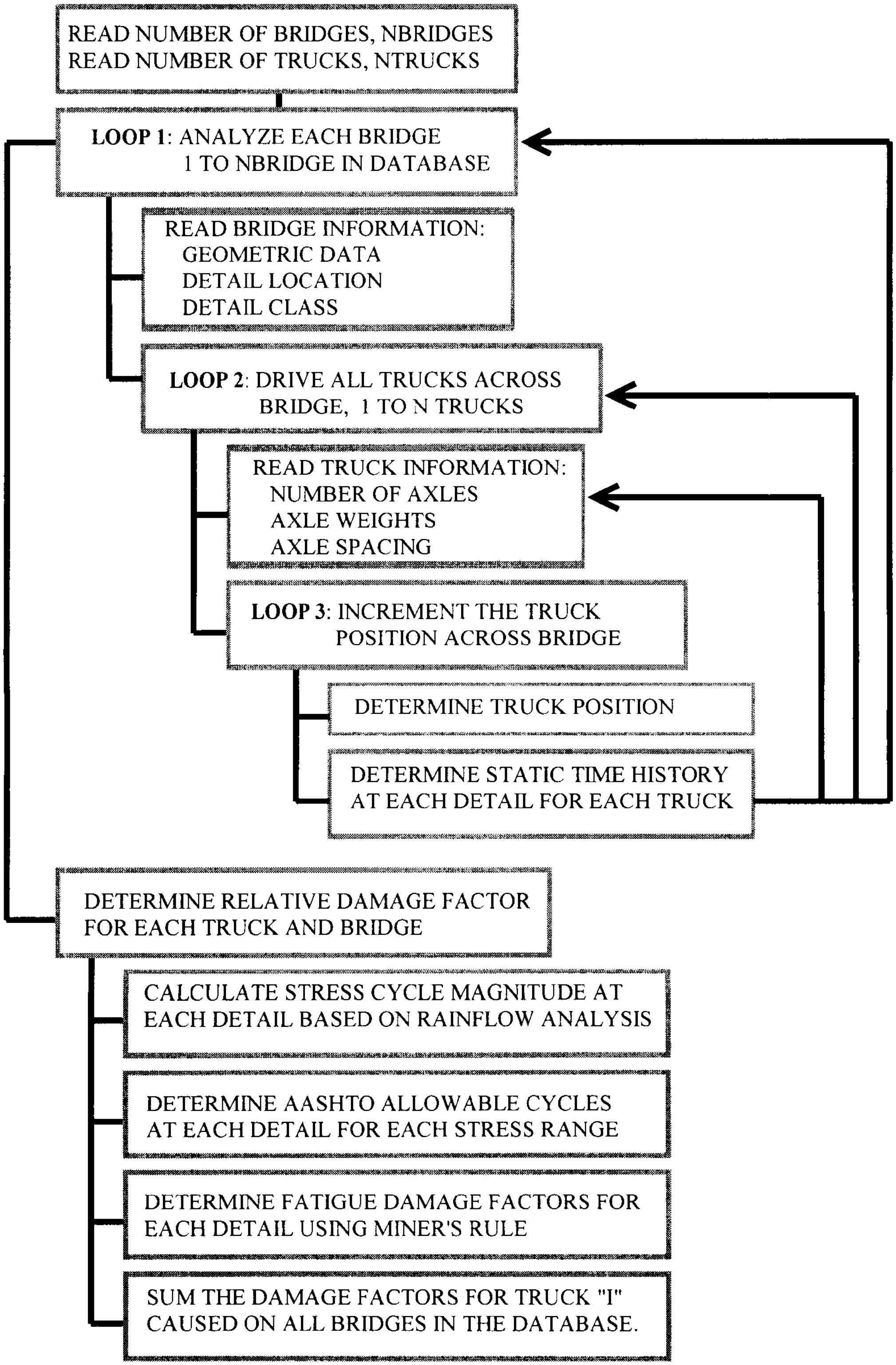

Figure 1. Flow Chart of Simulation Procedure.

Task 4: Develop analytical bridge models from actual bridge plans statistically selected from the NBI (Saklas et al 1988). These bridge plans are available from the 1988 bridge fatigue cost allocation study by Saklas et al., 1988. In the absence of these plans, a limited review and selection of NBI bridges and plans for analysis will be performed. The selected population of bridge plans will be designed to reflect increased use of continuous bridges since 1985.

Task 5: Perform an analysis of the representative population bridges chosen from the NBI. Simulations of truck crossings over bridge analytical models developed from plans will be performed using the semi-continuum methodology. The simulations will assess bridge response and relative fatigue damage for each of the critical AASHTO fatigue details and the established deck slab criteria by vehicle type and functional highway class. The study will be independent of fleet mix and will address relative damage attributable to each weight and vehicle class.

Task 6: Establish a scheme to determine relative damage factors for each of the vehicle classes/weight groups on a given functional highway class. Input variables may be VMT, ADTT, or PCEVMT or other statistic suitable for an intra-increment allocation. The procedure will be developed as a general tool for evaluation of the national bridge population subject to a given fleet mix.

Expected Products

This work will result in a general tool which capable of simulating the movement of any number of trucks from a predefined database over any number of bridge structures. The bridge structures must be of a type that can be modeled by the semi-continuum method. Results will be immediately transferable to a highway cost allocation study.

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FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Segment-Based Reporting for Element-Level Bridge Inspections

Author(s) and Affiliation(s):

George Hearn and Dan M. Frangopol
Department of Civil, Environmental and Architectural Engineering
University of Colorado, Boulder, CO

Principal Investigator: George Hearn

Sponsor(s): Colorado Department of Transportation

Research Start Date: June 1994
Expected Completion Date: August 1997

Research Objectives:

New procedures for element-level inspection of highway bridges together with new quantitative definitions of element condition ratings are developed and demonstrated. New procedures and condition ratings are needed to support quantitative evaluations of bridge condition directly from data collected in element-level bridge inspections. Quantitative evaluations include estimates of load capacity, structural safety, remaining service life, and rates of deterioration of bridge elements. The direct use of condition ratings means that quantitative evaluations of bridges can be made within bridge management systems (BMS). This allows BMS to consider the impact of repair programs on strength and safety of individual bridges. The research proceeds from two ideas: first, that structural engineering evaluations must be a part of bridge management systems and, second, that data gathering to support the evaluations must be compatible with routine inspection procedures.

Expected Products or Deliverables:
Research products include the development of relations between condition states of elements and damage or remaining strength in bridge members, development of graphics-based reporting forms for use in field inspections, a field demonstration of new segment-based inspection procedures, and development of automated procedures for load rating from condition ratings for segments. The research also has important applications in automated calibration of deterioration models from bridge inspection data, and automated estimates of remaining service life.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

George Hearn is Assistant Professor of Civil Engineering in the College of Engineering and Applied Science at the University of Colorado at Boulder. Professor Hearn specializes in bridge engineering with emphasis in bridge assessments, inspections, and the development of quantitative evaluations of strength and safety within bridge management systems. Professor Hearn is the principal investigator for projects that include the creation of procedures for reliability-based design of inspection programs for bridges, the automated generation of NBI condition ratings from bridge management system data, for the enhancement of element condition reports to allow load capacity evaluation of bridges within BMS, the use of transient response of bridges in support of condition assessment, and the study of the vulnerability of bridges to differential settlements at foundations. Professor Hearn works closely with the Colorado DOT in the design and evaluation of transportation structures, which has resulted in two US patents: one for cable-net rockfall barriers and the second for flexible facing systems for mechanically stabilized earth walls. Professor Hearn is a member of TRB and ASCE, and is active in numerous committees in both organizations. Before joining the faculty at Boulder, he was a consulting engineer working on rehabilitation of suspension bridges. Professor Hearn is a licensed professional engineer in New York and Colorado.

Dan M. Frangopol is Professor of Civil Engineering at the University of Colorado at Boulder. He received his Ph.D. in civil engineering from the University of Liege, Belgium in 1976. His background covers academic-oriented research and development in the fields of structural reliability, safety evaluation of existing structures, bridge engineering and structural optimization, and professional practice including design of masonry, steel, reinforced and prestressed concrete structures. Professor Frangopol is currently teaching graduate courses in the fields of structural and geotechnical reliability, structural optimization and inelastic theory of structures. He is also a researcher and consultant to industry in the fields of structural reliability and bridge engineering. A Fellow of ASCE and a member of ACI and TRB, Professor Frangopol currently serves as chair or member of many committees. He has lectured at several world renowned universities and published more than 150 papers on the applications of probabilistic procedures to structural engineering. He is the editor of two books, co-author of a book on reliability of steel structures and editorial board member of three international journals. Professor Frangopol has performed research and consulting for the NSF, Federal Highway Administration, Colorado DOT, Pennsylvania DOT, TRB, US Department of the Interior, and US Army Corps of Engineers.

Principal Investigator's Telephone Number: (303) 492-6381
Facsimile Number: (303) 492-7317
E-mail Address: hearn@stripe.colorado.edu

SEGMENT-BASED REPORTING FOR ELEMENT-LEVEL BRIDGE INSPECTIONS

George Hearn and Dan M. Frangopol
Dept. of Civil, Environmental and Architectural Engineering
University of Colorado
Boulder, CO 80309-0428

Period: June 1994 to Aug. 1997
Sponsor: Colorado Dept. of Transportation, Denver CO

RESEARCH OBJECTIVES

New procedures for element-level inspection of highway bridges together with new quantitative definitions of element condition ratings are developed and demonstrated. New procedures and condition ratings are needed to support quantitative evaluations of bridge condition directly from data collected in element-level bridge inspections. Quantitative evaluations include estimates of load capacity, of structural safety, of remaining service life, and of rates of deterioration of bridge elements. The direct use of condition ratings means that quantitative evaluations of bridges can be made within bridge management systems (BMS). This allows BMS to consider the impact of repair programs on strength and safety of individual bridges. The research proceeds from two ideas. First, that structural engineering evaluations must be a part of bridge management systems and second, that data gathering to support the evaluations must be compatible with routine inspection procedures.

RESEARCH APPROACH

This project is developing a new practice for coding and recording conditions of bridge members that will make possible meaningful automated evaluations of load capacity and safety. There are opposing goals of simplicity in inspections and of precision in strength and safety evaluations. Detailed data on the nature, severity and location of deterioration in a bridge allow precise computations of load capacity and safety, but detailed data require costly inspections. Routine, largely visual, inspections are relatively inexpensive, but offer no real opportunity for evaluations of load capacity and safety. Automated evaluations must be both economical and meaningful. Therefor, the data currently gathered in routine inspections must be enhanced, but modifications to inspection practice must be kept to a minimum.

The issue is data content. Load capacity is a comparison of member strength with load demand. Safety, apart from vulnerabilities, is a similar comparison, but includes the recognition of variability in strength and in demand. Data from bridge inspections must indicate the current strength of bridge members, and must allow for a determination of load demand. In the United States, bridge conditions are reported as qualitative, integer-valued condition ratings. This is true for the National Bridge Inventory protocol that has existed for more than 25 years, and also true for more recent element-level inspection procedures required by management systems such as

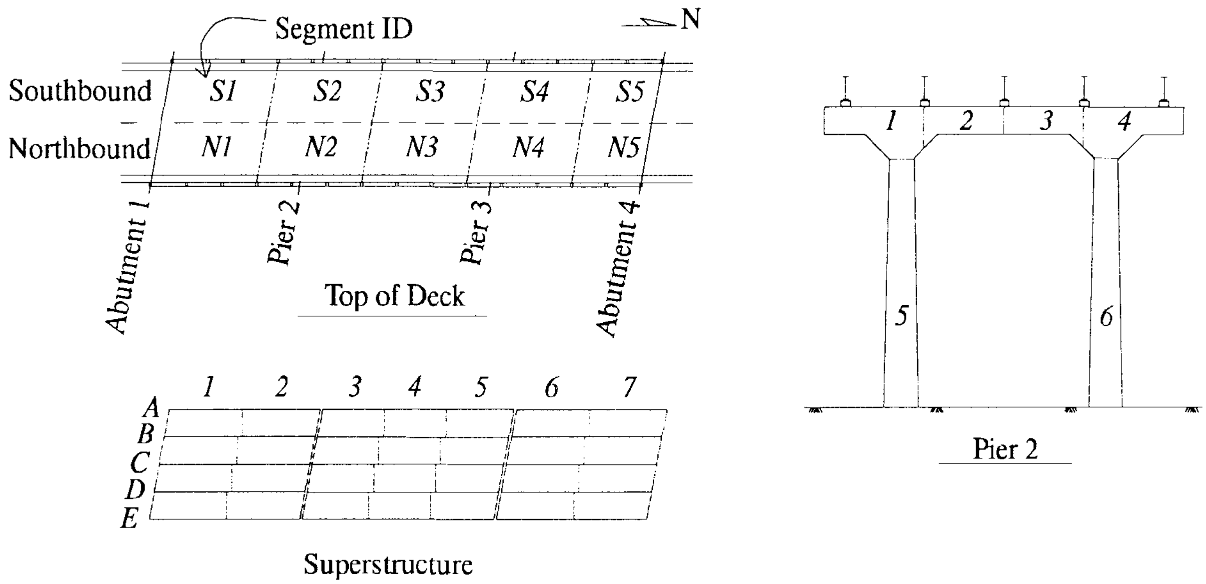

Figure 1 - Segments of Bridge Members

Pontis. These reporting practices do not provide either member strength or load demand.

Two enhancements are introduced. First, condition ratings are each defined as a range of damage in members [Hearn et al. 1995a]. For steel beams and steel reinforcing bars subject to corrosion, damage is defined as a normalized loss in thickness or in diameter. Strength of members can be estimated from data on normalized loss together with original design information. Damage in concrete is defined as normalized area of spalls, and as size and spacing of cracks. Damage can also be defined as normalized loss of any type. In this way, condition ratings can be linked to increasing concentration of chloride ions in concrete, or deterioration of paint on steel members. Such definitions do not indicate a loss of member strength, but they are useful in tracking life cycles of bridge members.

The second enhancement is a procedure for segment-based reporting. Segment-based reporting links condition ratings to locations, and it is this linkage that allows for the automated identification of load demand, and the automated comparison of load demand and current strength of bridge members.

Segments have three properties. Each segment has known material, cross section, and condition. From these data, segment strength can be estimated. Each segment has a known location, and therefor load demand in each segment can be estimated. Examples of segments are shown in Figure 1 for a three span continuous beam bridge. Segments for decks are bounded by lane stripes, by deck joints, by expansion joints in parapets or by other visible features. Segments in beams are defined along beam lines and bounded by diaphragm locations and by bearing points. Segments in pier caps are bounded by beam lines, and pier columns are each one segment. During a bridge

Route, Year Built & Type			Deck Segments No.	Deck Segments Area SF	Superstr. Segments No.	Superstr. Segments Len LF	Substr. Segments No.	Substr. Segments per Sub
US 50	1925	Steel thru truss	100	128	107	14	35	5
I-25	1959	P/S concrete multibeam	116	188	85	24	31	8
I-76 Serv	1935	Timber multibeam	8	202	28	14	42	21
I-25 Serv	1938	Steel pony truss	49	140	58	13	12	6
SH 56	1961	RC continuous beam	68	196	32	28	31	6
US 50	1957	Riveted plate girder	141	154	64	20	35	5
Wadsworth Av	1951	Steel continuous beam	82	130	90	10	33	7
SH 93	1950	Steel continuous beam	49	172	35	19	24	6

Table 1 - Field Study: Bridges and Segment Models

inspection at least one condition rating is assigned to each segment. For most beam segments, two condition ratings are assigned; one rating at each end of the segment.

Field inspection is accomplished using graphical reporting forms that show assemblies of the bridge, and that identify boundaries of segments and locations where condition ratings must be recorded. The interpretation of condition ratings as member strength, the identification of load demand in members, and the summation of quantities of elements in each condition state are all completed in automated procedures after the segment-based report is input to the bridge management system.

PRODUCTS OF THE PROJECT

Products of the research include the development of relations between condition states of elements and damage or remaining strength in bridge members, development of graphics-based reporting forms for use in field inspections, a field demonstration of new segment-based inspection procedures, and development of automated procedures for load rating from condition ratings for segments. The research also has important applications in automated calibration of deterioration models from bridge inspection data, and automated estimates of remaining service life.

PRELIMINARY RESULTS

Quantitative Condition Ratings: Quantitative condition ratings are obtained by relating each condition rating to an upper bound of damage in a member. Damage may be determined as a loss in cross section, particularly as a loss of thickness in parts of steel beams, or a loss in area of reinforcing steel. The Colorado Dept. of Transportation uses a set of Rust codes and Scale codes to define condition ratings [*BMS* 1995]. The definitions of condition ratings as damage are selected to be compatible with simple, verbal description of condition states.

Field Study: Segment-based inspections were executed at eight highway bridges in Colorado. The bridges range in age from 25 years to 70 years and include steel beam bridges, steel truss bridges, a timber bridge, reinforced concrete beam bridges and a prestressed concrete beam bridge (Table 1). The experience of field inspectors using segment-based reporting was very positive. The segment-based procedure demands the assignment of many condition ratings, but it relieves inspectors of the time-consuming task of quantity measurement required for element-level inspections. Quantities are computed from the array of segment condition ratings and from the known quantity for each segment. Overall, Colorado inspectors judged the segment-based techniques to be at least as fast, and possible faster, than element-level inspections required for Pontis.

Automated Load Rating: Data from segment-based inspections are used to compute load capacity ratings for all bridges in the field study. In addition, assumed deterioration rates are used in an exercise to study the potential change in load ratings over time [Hearn et al. 1995b]. Load ratings are determined from the comparison of strength with load demand. For beam bridges in the field study, there were four to six segments per beam line per span yielding eight to twelve condition ratings per beam line per span. With this density of condition ratings, the peak moment and shear demand at each condition rating can be estimated within 10%. Accuracy in beam strength is determined by range of damage covered by condition states. For condition states consistent with the Colorado DOT Rust codes, the uncertainty in remaining moment strength is not more than 5% of the original moment strength [Chakravorty 1995]. Uncertainty in shear strength is also within 5% for rolled shapes with stocky webs. For plate girders, uncertainty can be higher. Overall, load ratings for bridges are within 20% of the correct live load rating for medium span bridges.

Deterioration Modeling: Hearn [et al. 1995a] demonstrated the use of condition ratings to obtain quantitative estimates of deterioration rates. Decision based on life cycle performance (costs) for bridges related to need arising form deterioration must be considered using models developed specifically of the bridge network. Moreover, deterioration models are generated for important, representative microenvironments. Separate deterioration models for RC, P/S, and steel beams as well as separate models for each material recognizing differences of exposure are supported.

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FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Implementation of an Optical Fiber Monitoring System in a Full Scale Bridge

Author(s) and Affiliation(s):

Rola L. Idriss, M. B. Kodindouma and Ken R. White
Civil Engineering Department, New Mexico State University, Las Cruces, NM
M.A. Davis, D.G. Bellemore, Alan D. Kersey, M.A. Putnam and E.J. Friebele
Optical Sciences Division, Naval Research Laboratory, Washington, DC

Principal Investigator: Rola L. Idriss

Sponsor(s): National Science Foundation, New Mexico State Highway and Transportation Department, and the Federal Highway Administration

Research Start Date: 1995
Expected Completion Date: 1998

Research Objectives:

The objective of this research project is the development and implementation of a bridge monitoring and evaluation system. Strain measurements at critical locations in the structure, obtained from a bridge monitoring system, can give quantitative, simple and straightforward means of evaluating an existing bridge, its performance under load, and the effect of any damage on its capacity and structural integrity. This could result in better decision making, more efficient bridge management, and tremendous savings.

Expected Products or Deliverables:

The development of a Bridge Monitoring and Evaluation System.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Rola L. Idriss is Associate Professor of Civil Engineering at New Mexico State University. Her area of research is in bridge nondestructive evaluation and testing. She is currently working on the development and testing of structural monitoring systems, development of computer models to predict the response of damaged and deficient structures, and the verification of the models by means of large and full scale experimental testing. She is the recipient of the National Science Foundation Young Investigator Award for excellence in research and teaching - 1994; the Patricia Christmore Faculty Teaching Award - 1994; the Roush Award for excellence in teaching - 1995; and the Bromilow Award for excellence in research - 1995. Dr. Idriss is a member of a number of Transportation Research Board and ASCE committees, and is a member of an NCHRP Project Panel on horizontally curved steel briges. She is also a member of the New Mexico Society of Professional Engineers, the National Society of Professional Engineers, ACI, and the American Society for Engineering Educators.

Principal Investigator's Telephone Number: (505) 646-3818
Facsimile Number: (505) 646-6049
E-mail Address: ridriss@cage.nmsu.edu

IMPLEMENTATION OF AN OPTICAL FIBER MONITORING SYSTEM IN A FULL SCALE BRIDGE

R.L. Idriss[1], M.B. Kodindouma [1], K.R. White[1], M.A. Davis[2], D.G. Bellemore[2], A.D. Kersey[2], M.A. Putnam[2] and E.J. Friebele[2]

[1]New Mexico State University
Civil Engineering department
Box 3CE, Las Cruces, New Mexico 88003-8001.

[2] Optical Sciences Division. Code 5600
Naval Research Laboratory,
Washington, D.C. 20375

Research Objectives

Currently, damage assessment and performance evaluation of bridges are mainly done by a "hands on" visual inspection. This method is labor intensive and subjective since it relies on visual indication of stress rather than quantitative data measurements.

The objective of this research project is the development and implementation of a bridge monitoring and evaluation system. Strain measurements at critical locations in the structure, obtained from a bridge monitoring system can give quantitative, simple and straightforward means of evaluating an existing bridge, its performance under load, and the effect of any damage on its capacity and structural integrity. This could result in better decision making, more efficient bridge management and tremendous savings. This sensor system will provide an objective tool to:

- Assess the loading history
- Evaluate the effects of damages on the capacity and performance of a bridge
- Assess the effectiveness of repairs and maintenance programs
- Check the performance compared with the design assumptions
- Remote monitor critical structures, and provide a warning when abnormal conditions occur.

Research Approach

The multi-disciplinary research study is an on-going testing program, implemented in several phases, (a) large scale component testing (b) full scale bridge testing in the laboratory and (c) full scale implementation in the field on existing bridge structures. Phase (b) is the current phase of the project, and will be covered in this paper. For full results, the reader is referred to [1].

Full Scale Laboratory Bridge: A full scale 40-ft simple span, non composite steel girder concrete deck test bridge (fig.1&2) is built in the laboratory with an integrated sensor system. The sensor locations (fig. 1,2,&3) were optimized using finite element model simulations of the bridge. A network of forty-eight optical fiber sensors was used to monitor the strain in the bridge with sensors both embedded in the concrete slab and bonded to the steel girders. Sensors in the slab were bonded to the tension steel. Optical fiber gages and resistive gages were placed side by side, and data was compared.

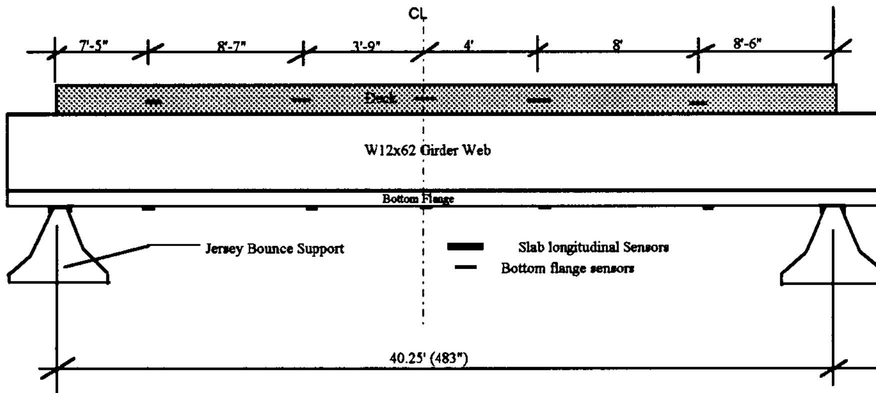

Fig.1 Elevation View and Longitudinal Sensors

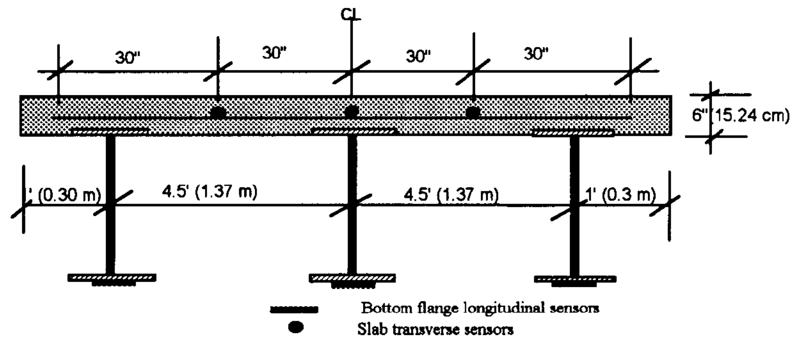

Fig.2 Midspan Cross Section

<u>Optical Fiber Sensors</u>: Besides flexibility, and extremely small size (250 micron in diameter), optical fibers are immune to electric and electromagnetic interference. Probably one of the most attractive features of optical fiber sensors is their inherent ability to serve as both the sensing element and the signal transmission medium, therefore greatly simplifying the instrumentation of large structures.

The Fiber Bragg Grating (FBG) sensors used in the research are intrinsic sensors, i.e. sensing occurs along the fiber itself, while the light remains guided through the fiber. The sensors are written into the core of an optical fiber forming a small periodic modulation of the refractive index. Due to the periodic nature of this perturbation only several discrete optical frequencies will resonate in the structure. Therefore, if broadband light is traveling in the core of the optical fiber the incident energy at such a resonant frequency will be reflected back down the optical fiber, with the remaining optical spectra unaffected. Any strain induced change in the modulation spacing or overall refractive index with cause a shift in the Bragg resonance wavelength. Consequently, strain induced effects on the FBG can be determined by monitoring the corresponding shift in the center Bragg wavelength. One major advantage of the FBG sensors is the ease with which several can be multiplexed along a single optical fiber. This can be accomplished by wavelength division multiplexing (WDM) several FBG sensors, where each grating is written on the fiber with its Bragg resonance at a different optical wavelength [2]. The instrumentation system used here provides the capability to monitor a very large number of Bragg gratings using a common source and scanning narrowband filter. As currently implemented, the system has the capability to monitor 12 FBG sensors along each of 5 separate fibers for a total of 60 sensor elements. The strain resolution of this system was determined to be 0.95 microstrain. The sensors and instrumentation used in this bridge research were developed at the Naval Research Laboratory.

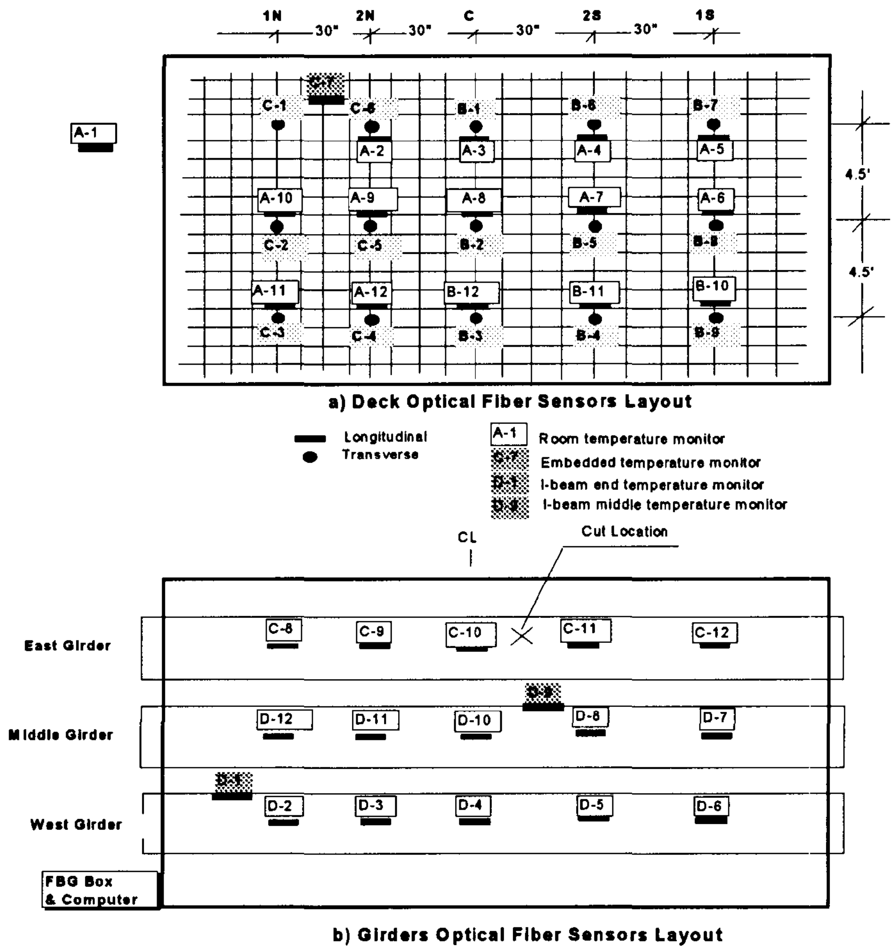

Figure 3. Sensor Layout. Plan Views.

Expected Products

The primary products that will result from this research project:

- Development of a data acquisition system using fiber optics for remote monitoring of large civil structures.
- Adaptation of the fiber optic sensing technology to meet the civil engineering needs.
- Development of a methodology for NDE assisted bridge inspection and evaluation.
- Incorporation of the field data into bridge management. This can be done systematically, since New Mexico State University is the provider for the bridge inspection and management for the State of New Mexico under a contract with the New Mexico State Highway Department.

Preliminary Results

Laboratory results showed the system to be a powerful bridge diagnostic tool. When a fracture was introduced in the bridge, the monitoring system showed a drastic change in the structure's response. It indicated a major damage had occurred, along with the time and the location of the damage (fig. 4). The fracture was near midspan of the east girder,and severed the bottom flange and half of the web. The sudden change in response was most accentuated at the vicinity of the crack. In the intact bridge, dead load strain at midspan, in the bottom flange was at about 350 microstrains for the middle girder, and 200 microstrains for an exterior girder. When fracture occurred, under dead load, the damaged girder carried less load, with about 50 % loss in strain at midspan. The load was shed to the middle girder which picked up a 40% increase in strain at midspan. The west girder showed little change with only a 5% increase in strain. There was also a significant change in the strain pattern in the deck.

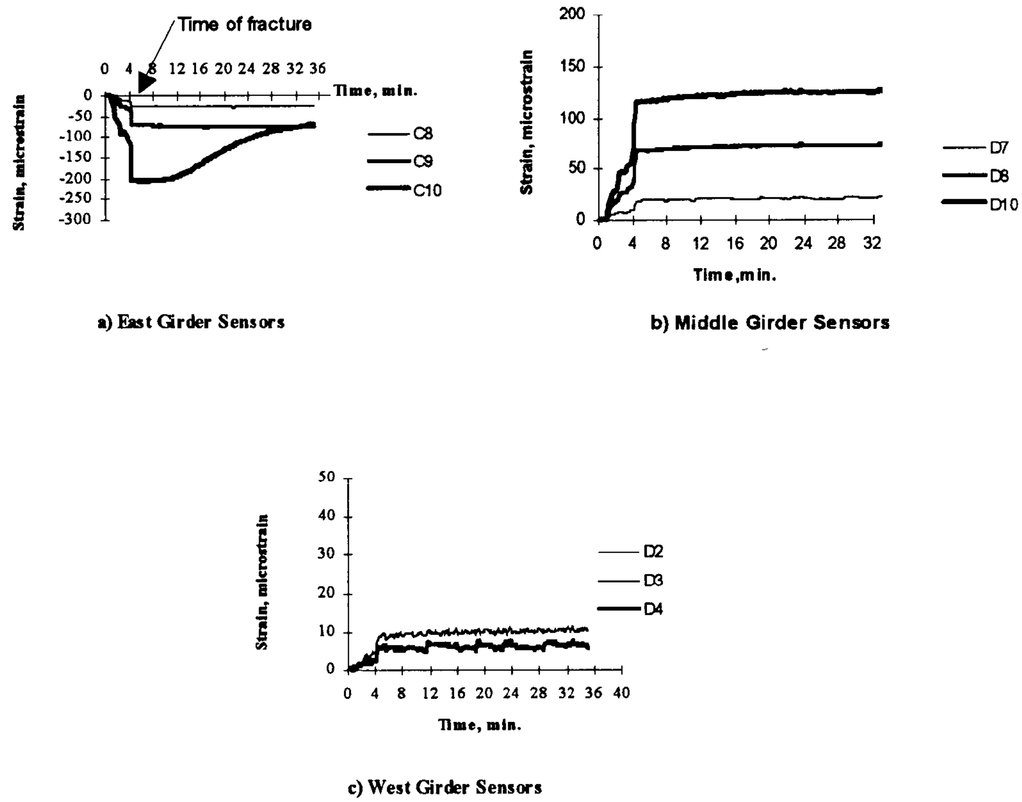

Figure 4. Girders After Fracture Dead Load Strain Response

In the next phase of the project, the "smart bridge" technology using fiber optics will be retrofitted and field tested on an interstate highway bridge in New Mexico.

Acknowledgments

This project is a collaborative research project between New Mexico State University and the Naval Research Laboratory. It is supported by the National Science Foundation grant no CMS-9457604, the New Mexico State Highway and Transportation Department and the Federal Highway Administration.

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FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Nondestructive Testing of Bridges

Author(s) and Affiliation(s):

Vijay Saraf, Andrzej S. Nowak and Sangjin Kim
University of Michigan, Ann Arbor, MI

Principal Investigator: Andrzej S. Nowak

Sponsor(s): Michigan Department of Transportation and the National Science Foundation

Research Start Date: June 1, 1995
Expected Completion Date: August 31, 1996

Research Objectives:

The objective of this study is to develop an integrated field testing procedure for evaluation of bridges. It is often observed during load testing of a bridge that the actual load capacity is higher than that obtained from analytical methods. In certain cases, this extra safety reserve can be utilized to avoid rehabilitation by using nondestructive testing. Moreover, thorough evaluation of the bridge condition also requires the knowledge of actual traffic load and its distribution to different bridge components. Therefore, testing procedures will be developed to determine the bridge load capacity, the actual live load distribution and the fatigue load spectra.

Expected Products or Deliverables:

The proposed procedure for proof load testing was found to be a fast, efficient and economical way to verify the load capacity. The results from weigh-in-motion measurements can be utilized to update the live load models for bridge design specifications. The experimental procedures can be combined together to develop an automated bridge monitoring and management system.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Vijay Saraf is currently a Ph.D. candidate at the University of Michigan. His dissertation focuses on developing reliability based criteria for proof load testing of bridges. He has been conducting proof load tests on medium span highway bridges.

Andrzej S. Nowak received his Ph.D. from Warsaw University of Technology, Poland, and has been at the University of Michigan since 1979. He has been involved in reliability-based development of LRFD bridge design codes in the U.S. and Canada.

Sangjin Kim is a Ph.D. candidate at the University of Michigan. His dissertation involves bridge evaluation based on field measurements. For the past four years, he has been conducting tests on steel bridges.

Principal Investigator's Telephone Number: (313) 764-9299
Facsimile Number: (313) 764-4292
E-mail Address: nowak@engin.umich.edu

NONDESTRUCTIVE TESTING OF BRIDGES

Vijay Saraf, Andrzej S. Nowak and Sangjin Kim

Department of Civil and Environmental Engineering
University of Michigan, Ann Arbor, MI 48109-2125

Research Objective

The objective of this study is to develop an integrated field testing procedure for evaluation of bridges. It is often observed during load testing of a bridge that the actual load capacity is higher than that from analytical methods. In certain cases, this extra safety reserve can be utilized to avoid rehabilitation by using nondestructive testing. Moreover, thorough evaluation of the bridge condition also requires the knowledge of actual traffic load and its distribution to different bridge components. Therefore, testing procedures will be developed to determine the bridge load capacity, the actual live load distribution and the fatigue load spectra.

Research Approach

Load Capacity Evaluation: The load capacity is obtained using the diagnostic test or the proof load test. The purpose of the diagnostic testing is to verify and calibrate the results of an analytical model. A selected load is applied at several locations on the bridge and the response of the structure (i.e. deflection and strain) is measured at designated locations. The results are used to reduce the uncertainties about bridge behavior (e.g. the girder distribution, contribution of non-structural members, unintended composite action, support conditions etc.) in analytical modeling.

For proof load tests, the strain and displacement transducers are placed at selected locations on the bridge. Then, it is incrementally loaded until a predetermined load level or the yield capacity of the bridge is reached. As part of an ongoing study, the proof load tests are being carried out on selected bridges in Michigan. Based on design details and the observed deterioration, the critical limit state is determined for each structure. Considering a factor of safety, the impact factor and few other factors, the considered proof load level is about twice the maximum allowable legal load. Michigan law permits exceptionally heavy loads. The 11-axle truck in Michigan is allowed to carry as much as 77 tons. Therefore, a very high proof load level is required. The M60A3 military tanks are used to load the bridges. Each tank weighs about 60 tons. However, contrary to 11-axle trucks, tanks have a heavy load over a small area. Therefore, they cause very large moment at mid-span. These tanks are provided by the Michigan National Guard. During proof load testing, the mid-span moment is increased in several steps by gradually moving the tanks closer to the mid-span. Tanks are also moved in transverse direction to three different locations : upstream, center and downstream (i.e. in the upstream position the tanks are placed closer to the upstream railing). Stresses and deflections are measured for each load increment. At all stages of proof load testing, the stresses and deflections are closely monitored and compared to analytically predicted values. Low stress values and a linear response are considered as indications of an adequate safety reserve.

Weigh-in-Motion Test: The weigh-in-motion (WIM) test is used to obtain the actual truck load data. Truck weights, including axle loads and spacings, are measured to determine the statistical parameters of the actual live load. The current system uses instrumented bridge girders that offer several advantages over other systems, such as pavement scales. It consists of strain transducers, lane sensors, a processing unit and a portable computer. The strain transducers used for the system are reusable and clamped to the bottom flange of steel girders in the middle third of the span. Two lane sensors are placed on top of the bridge in each lane. Tape switches or the infrared sensors are used as lane sensors. They provide the speed of vehicles, number of axles in a vehicle and the axle spacings. Then, the axle weights are determined by using the strain data together with the truck configuration and influence lines. For each bridge the WIM data acquisition system is calibrated using trucks provided by the Michigan DOT. The measurements can be carried out in up to two traffic lanes at a time. WIM measurements of trucks can be taken discretely during normal traffic, resulting in unbiased data. These measurements indicate that truck loads are strongly site specific. The accuracy of measurements varies depending on the number of axles on the vehicles. For vehicles with up to five axles, gross vehicle weight (GVW) is determined within 5 percent, and for 11 axle trucks within 10 percent. Axle weights are determined within 20 percent.

Fatigue Evaluation: In order to develop meaningful fatigue load models specific to a selected bridge type and its estimated load spectrum, it is necessary to collect unbiased information about the traffic supported by the bridge and the bridge response to these loads. Therefore, in addition to WIM, a data acquisition system from SoMat Corp. is also used for fatigue related study. It collects the strain history under normal traffic and assembles the stress cycle histogram by the rainflow method of cyclic counting, and other counting methods. The data are then stored to memory and downloaded at the conclusion of the test period. The current system is capable of recording up to 4 billion cycles per channel for extended periods in an unattended mode. It can continue testing for as long as three weeks before requiring batteries replacement. The strain transducers are placed at critical locations on all steel girders. The strain data is collected continuously for one weeks periods and reduced using rainflow algorithm. The level of damage induced by a single stress cycle increases significantly with higher levels of stress. In addition to the magnitude of the stress cycles, the number of load cycles is an important parameter in the fatigue life prediction. Therefore, fatigue behavior of bridges is predominantely a function of truck traffic and its parameters. Static load cycles are determined both analytically by counting the stress cycles for different spans, and experimentally for the tested bridges. Fatigue damage is estimated using Miner's rule of linear damage accumulation.

Expected Products

The proof load testing is considered an efficient tool to determine the load capacity of a bridge. However, it is not commonly used due to the complexity involved in conventional approach. Alternatively, the simplified proof load testing procedure described here can be used. Use of tanks was found to be a fast, efficient and economical way to conduct proof load testing.

The weigh-in-motion tests provide the actual live loads experienced by the selected bridge. The results can be used to build an unbiased data base of truck measurements and weights, which can be utilized to update the live load models for bridge design specifications. In addition, collected database will provide a basis for development of site-specific load models in bridge management.

Knowledge of the past and current load spectra, together with predicted future loads, is essential in the fatigue analyses. Comparison of load spectra in different locations can serve as a basis for identification of the fatigue critical components. This information can be useful to focus inspection and repair efforts. The experimental procedures described here can be combined together to develop an automated bridge monitoring and management system.

Preliminary Results

Load Capacity Evaluation: The proof load tests have been carried out on five medium span bridges so far. These bridges are simply supported structures with spans ranging from 6.5 m to 15.2 m. First bridge is a reinforced concrete T-beam structure and the other four are noncomposite steel girder bridges. All bridges are over 60 year old and had varying degree of deterioration. For first bridge only one tank was enough to achieve target proof load level. For other four bridges two tanks were required. Deflections and stresses were measured at mid-span and the quarter-span of selected girders. The resulting stresses were considerably below critical values. Stresses in girders increased linearly with increasing lane moment. The maximum observed stress in lower flange of steel girders was only 30.0 MPa. In general, the experimental deflections were also much smaller than those from analytical methods. During the experiments, the maximum deflection at mid-span of the all bridges was only 4.7 mm. Each bridge was able to sustain the proof load without any sign of distress. Comparison of experimental stresses with analytical stresses indicate the presence of unintended composite action between concrete slab and steel girders.

Weigh-in-Motion Test: Truck weights were measured on selected 20 bridges in Michigan. In general, truck load on bridges is strongly site-specific. There is considerable variation in traffic volume and weight of trucks. The estimated average daily truck traffic (ADTT) varies from 500 to over 2,000 in one direction. The median values of GVW are similar for all locations, varying from 150 kN to 210 kN. The maximum observed GVWs vary from 360 kN to 1,125 kN. The maximum observed axle weights vary from 90 kN to almost 225 kN. The distribution of truck types by number of axles will typically bear a direct relationship to the GVW distribution. The data obtained in this study indicates that between 40 to 80 percent of the truck population are 5-axle vehicles, depending considerably on the location of the bridge. Three and 4-axle vehicles are often configured similarly to 5-axle vehicles, and when included with 5-axle vehicles, they account for 55 to 95 percent of the truck population. Between 1.5 to 7.5 percent of the trucks are two unit 11-axle vehicles in Michigan.

Fatigue Evaluation: Strain histories were collected continuously for one week for five steel girder bridges. These tests indicate that live load stress spectra are strongly component-specific. Each component experiences a very different distribution of strain cycle ranges. The equivalent (root mean cube) stresses were less than 10.5 MPa for all girders and all bridges. The girder that is nearest the left wheel track of vehicles traveling in the right lane experiences the highest stresses in the stress spectra and decreases as a function of the distance from this location. It was observed that a vehicle type that dominates the distribution of vehicle types does not necessarily dominate the fatigue damage of the particular component. However, the vehicle type that dominates the distribution of lane moments will likely dominate the fatigue analysis. Fatigue load models were developed for accurate prediction of fatigue damage caused by normal truck traffic passing over a bridge

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FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Bridge Research in Progress at the University of Cincinnati

Author(s) and Affiliation(s):

Ahmet E. Aktan, M. Baseheart, R. Miller, I. Minkarah, B. Shahrooz, D. Brown, R. Buchanan, N. Jayaraman, A. Helmicki, S. Shelley and G. Simitses
Infrastructure Institute, University of Cincinnati, Cincinnati, OH

Principal Investigator:

Sponsor(s): Ohio Department of Transportation

Research Start Date: Varies (earliest 1994)
Expected Completion Date: Varies (latest 1998)

Research Objectives:

The objectives of these studies are to generate factual data about the actual ranges of state parameters and the loading environments for common bridge types; gain an understanding of how bridges actually respond to many different external and intrinsic loading effects and the mechanisms which trigger the actual (as opposed to envisioned) limit-states; establish the actual capacities of the different mechanisms which resist load effects at the serviceability and damageability levels; and establish the effects of aging and deterioration on both load demands and capacities.

Expected Products or Deliverables:

A database characterizing actual ranges of bridge state parameters; analytical and experimental tools together with use-modes and application rules for condition assessment and reliability evaluation; use-modes and application rules for field instrumentation and data acquisition; fully operational test-beds available for future research projects; and quantification and characterization of the effectiveness of a wide range of indices at detecting aging and deterioration.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Ahmet E. Aktan is Professor of Civil and Environmental Engineering and Director of the Infrastructure Institute at the University of Cincinnati. He received his Ph.D. from the University of Illinois at Urbana in 1973.
T.M. Baseheart is Associate Professor of Civil and Environmental Engineering at the University of Cincinnati. He received his Ph.D. from the University of Cincinnati in 1973.
D.L. Brown is Professor of Mechanical, Industrial and Nuclear Engineering at the University of Cincinnati. He received his Ph.D. from the University of Cincinnati in 1976.
R.C. Buchanan is Professor and Department Head of Materials Science and Engineering at the University of Cincinnati. He received his Ph.D. from Massachusetts Institute of Technology in 1964.
N. Jayaraman is Professor of Metallurgical Engineering at the University of Cincinnati. He received his Ph.D. from the Indian Institute of Science in 1977.
Arthur J. Helmicki is Assistant Professor of Electronic and Computer Engineering at the University of Cincinnati. He received his Ph.D. from Rensselaer Polytechnic Institute in 1989.
R.A. Miller is Associate Professor of Civil and Environmental Engineering at the University of Cincinnati. He received his Ph.D. from Northwestern University in 1989.
I.A. Minkarah is Professor of Civil and Environmental Engineering at the University of Cincinnati. He received his Ph.D. from Rensselaer Polytechnic University in 1977.
B.M. Shahrooz is Associate Professor of Civil and Environmental Engineering at the University of Cincinnati. He received his Ph.D. from the University of California at Berkeley in 1987.
S. Shelley is Research Assistant Professor of Mechanical, Industrial and Nuclear Engineering at the University of Cincinnati. He received his Ph.D. from the University of Cincinnati in 1991.
G.J. Simitses is Acting Dean of Engineering at the University of Cincinnati. He received his Ph.D. from Stanford University in 1965.

Principal Investigator's Telephone Number: (513) 556-3689
Facsimile Number: (513) 556-2599
E-mail Address:

BRIDGE RESEARCH IN PROGRESS AT THE UNIVERSITY OF CINCINNATI

E. Aktan, M. Baseheart, R. Miller, I. Minkarah, B. Shahrooz, D. Brown, R. Buchanan, N. Jayaraman, A. Helmicki, S. Shelley, G. Simitses

Infrastructure Institute, College of Engineering
University of Cincinnati
Cincinnati, OH 45221

in collaboration with

J. Barnhart, W. Edwards, R. Eltzroth, V. Dalal, R. Engel, L. Welker

Ohio Department of Transportation

Research Objectives

The following research contracts and grants on highway bridge research are in progress at the University of Cincinnati:

1. Condition Assessment and Capacity Evaluation of Historic Steel Bridges; Sponsors: NSF and FHWA; Faculty: Aktan, Buchanan, Jayaraman; 1994-1997.
2. Instrumentation, Testing and Monitoring of Reinforced Concrete Deck-On-Steel Girder Bridges, Phases I, II, and III; Sponsors: Ohio DOT, FHWA and OBR; Faculty: Aktan and Helmicki; 1994-1998.
3. Bridge-Type Specific Management of Steel-Stringer Bridges, Phase 1: Validation of Tools; Sponsors: Ohio DOT and FHWA; Faculty: Aktan, Brown, Helmicki and Shelley; 1996-1998.
4. Bond Characteristics of Overlays Placed Over Bridge Decks Sealed With HMWM or Epoxy; Sponsors: Ohio DOT and FHWA; Faculty: Shahrooz; 1995-1997.
5. Full-Scale PC Beam Tests for Evaluating Shearkey Performance; Sponsors: Ohio DOT and FHWA; Faculty: Miller; 1995-1997.
6. The SHRP High-Performance Concrete Showcase: Skewed Box-Beam Bridge; Sponsors: SHRP/FHWA and Ohio DOT; Faculty: Baseheart, Miller and Shahrooz; 1996-1998.
7. Bridge Monitoring By Passive Smart-Sensors; Sponsor: FHWA (Broad Agency Announcement); Faculty: Aktan, Helmicki, Minkarah and Shelley; Industry Partner: Strain Monitor Systems, Atlanta; 1996-1998.

Each individual research project naturally has specific objectives and scope, however, they also share the following global objectives: (a) generate factual data about the actual ranges of state parameters and the loading environments for common bridge types; (b) gain an understanding of how bridges actually respond to many different external and intrinsic loading effects and the mechanisms which trigger the actual (as opposed to envisioned) limit-states; (c) establish the actual capacities of the different mechanisms which resist load effects at the serviceability and damageability levels; and (d) establish the effects of aging and deterioration on both load demands and capacities.

Research Approach

The research is conducted by multi-disciplinary teams of faculty and students under the umbrella of the University of Cincinnati Infrastructure Institute, established in 1988. As an example, the projects listed herein include the efforts of personnel (i.e., the co-authors) from the Departments of Civil and Environmental Engineering; Mechanical, Nuclear, and Industrial Engineering; Materials Science and Engineering; Electrical and Computer Engineering; and Aerospace Engineering and Engineering Mechanics at the College of Engineering at the University of Cincinnati.

Every one of the research projects has a field component, taking advantage of an in-service or decommissioned highway bridge as a prototype test specimen (Figure 1). Each project is conducted in a system-identification framework for optimally integrating the analytical and experimental components of the research. The research projects have been designed to explore unknown-unknowns and generate new knowledge on fundamental bridge behavior, in addition to leading to near-term implementable products. Conducting the research within a partnership of government (agencies and officials responsible from highway bridge operations), industry (design consultants, contractors and materials suppliers) and academe, and by utilizing actual bridge test-beds, researchers maintain a sharp focus on a realistic appreciation of the problems they are exploring as well as the actual state-of-the-practice, so that the research produces near-and-long-term practical benefits.

Expected Products and Results

Project (1) specifically aims to explore the micro-mechanisms which cause aging and deterioration of steel, and whether brittle failure of the material may be predicted by meso-level sampling and scanning electron microscope analyses. In conjunction with material tests at the micro, meso and coupon levels, researchers will be testing built-up members salvaged from 100-year old truss bridges. The studies will lead to a better understanding of the mechanisms that govern the linkage between behavior observed at different scales of material and element levels.

Projects (2), (3) and (7) are focused on condition assessment and reliability evaluation of steel-stringer bridges by developing instrumented monitoring and modal testing as two kernel experimental tools which may be used in many modes. Commercially available active sensors, in conjunction with available data acquisition hardware and software, are being used to monitor several test-beds that are in-service or being constructed. Passive sensors, wireless data acquisition/transmission, programmable graphical user interfaces, and automated/remote video/image processing technologies are being explored for developing the intelligent monitor systems of the future. Project (3) aims at staged destructive testing of a decommissioned test-bed to verify the use-modes and applicability of both modal analysis and instrumented monitoring for damage assessment in conjunction with the algorithms and damage indices that have been proposed in the literature, including modal flexibility, uniform load surface and curvature variation within the uniform load surface which are indices proposed by the writers (Figures 2 and 3). These studies will lead to the development tools (and methodologies for applying them) that will allow for remote, automated health monitoring of highway bridges leading, in turn, to improved, more cost effective management. In addition, data collected and archived will form the basis for a database to be used in bridge-type-specific monitoring/management strategies through trending, baseline development, signature analysis, statistical analysis, and other post-processing methods.

Project (4) involves laboratory and field testing to establish the bond strength between various types of overlays and decks treated with HMWM or epoxy, and untreated decks. Researchers will examine different test techniques that are appropriate for reaching a complete understanding of the behavior and performance of overlay-deck interfaces.

Projects (5) and (6) are directed to better understand the behavior of reinforced concrete deck-on-prestressed concrete (PC) box-girder bridges. Project (6) is a SHRP high-performance concrete showcase, and incorporates a long-term field evaluation of an applicable operating bridge that will be constructed as a test specimen.

Preliminary Results

These projects, particularly projects (2), (3), and (7), have led to the formulation of hierarchal, integrated, systems-oriented design paradigm for the development of an intelligent monitoring system (Figure 4). Under this scenario, information obtained either through controlled testing or during normal operation using environmental sensors and weight-in-motion scales will be processed together with dynamic system models of bridge behavior. It will be capable of the full spectrum of activities from acquiring, conditioning, and archiving data to data analysis and the qualitative evaluation of the status of the structure. In this way the level of deterioration or damage can be weighted in the context of its probable causes, the structure's lifecycle, and present safety requirements.

To date, these research projects have been able to begin the assembly of a database to study and quantify the various cause-and-effect relationships which impact the performance of a bridge. These have included the characterization of (actual) sensor and data acquisition system performance (as opposed to advertized), the development of appropriate calibration and installation methods for obtaining accurate field measurements, the characterization of bridge behavior in response to normal operational traffic, environmental, and controlled input effects, and the quantification of absolute state-of-stress as a consequence of fabrication and constructions practices and methods.

In the future researchers will be testing statistical samples of common steel, RC and PC bridge-types and developing families of baselines for practical and objective global condition assessment. A mobile field laboratory and a bridge commando team, capable of rigorously modal testing and conducting structural identification of about 30 typical multi-span bridges every year, are being developed. The rational knowledge-base which will result from these studies of the steel-stringer and PC box-girder bridges will permit designing optimal management of the corresponding components of the current and future bridge stock.

Fig. 1: Steel-Stringer Test Bridge

Fig. 2: Impact Modal Testing

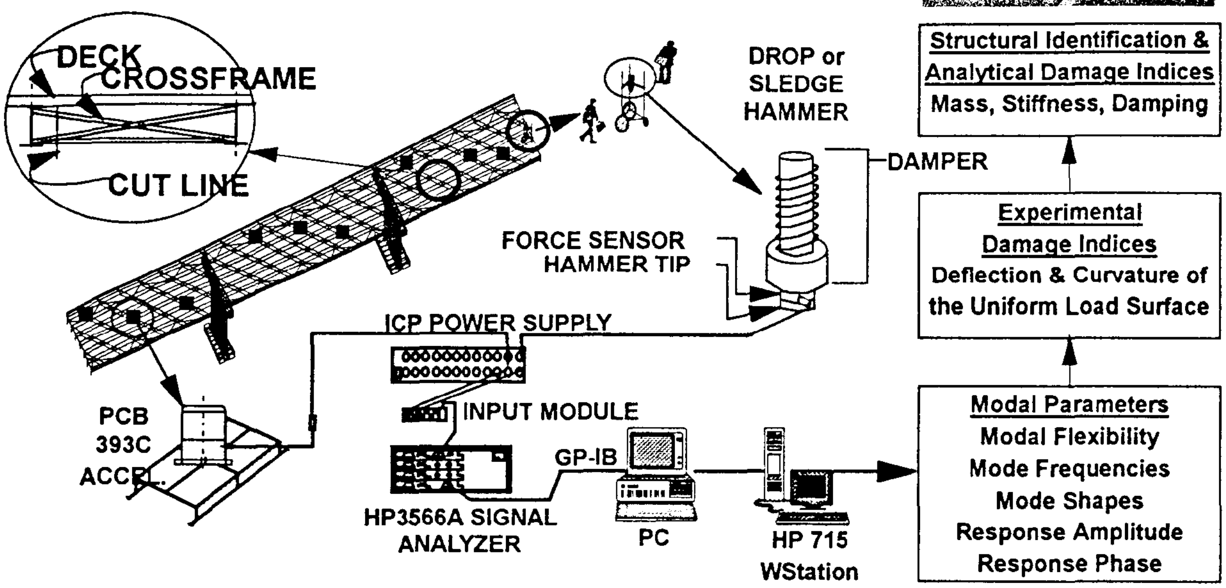


Fig. 3: Modal Impact Test System Configuration for Detection of Induced Damage

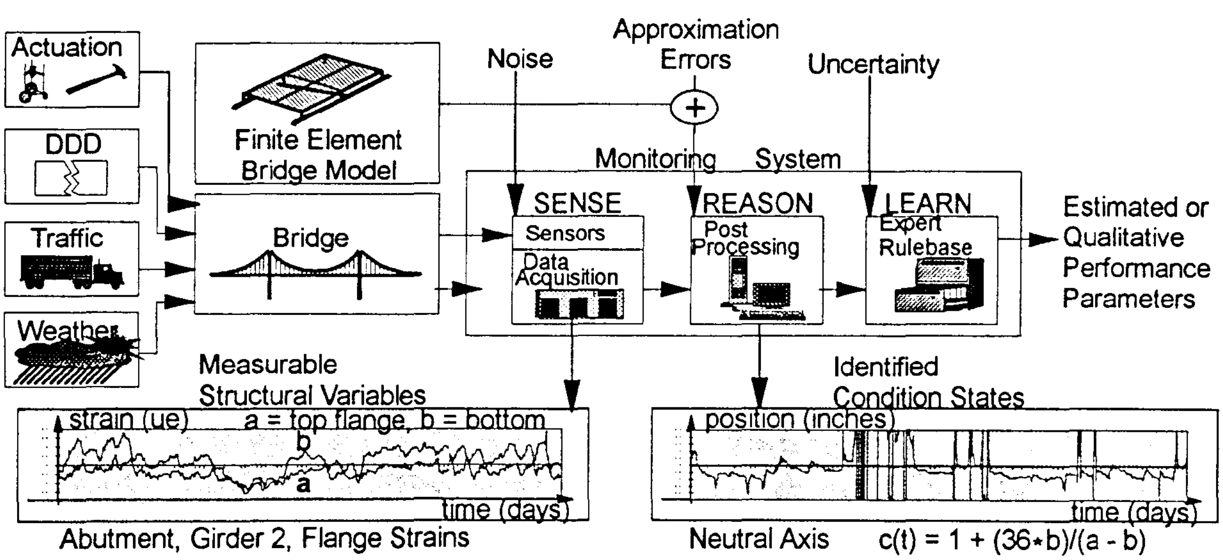


Fig. 4: Instrumentation-Based Monitoring Strategy

Seismic Performance, Design, and Retrofitting (Part 1)

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Seismic Analysis of Bridges Including Soil-Structure Interaction Effects

Author(s) and Affiliation(s):

Raimondo Betti and Kaizan Huang
Columbia University, New York, New York

Principal Investigator: Ahmed M. Abdel-Ghaffar (USC)

Sponsor(s): California Department of Transportation

Research Start Date: October 1, 1995
Expected Completion Date: April 31, 1996

Research Objectives:

In this study, the analysis of soil-structure interaction effects and of multiple support excitation on the seismic response of long-span cable-supported bridges are presented. The purpose of this study is to provide a better understanding of the dynamic performance of cable supported bridges under earthquake excitation, considering the spatial variability of the ground motion and the complex phenomena of soil-bridge interaction, so that such effects can be included in the design and rehabilitation process. Amplification of structural displacements and internal forces will prove that neglecting soil-structure interaction effects leads to inaccurate and unsafe design procedures.

Expected Products or Deliverables:

A technical report document of the research results.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Raimondo Betti is Associate Professor in the Department of Civil Engineering and Engineering Mechanics at Columbia University in New York City. Dr. Betti received his MSCE and Ph.D. from the University of Southern California in 1988 and 1991, respectively, and his "Laurea" in Civil Engineering at the University of Rome "La Sapienza," Italy in 1985. In 1994, he was awarded an NSF National Young Investigator Award. Dr. Betti is a member of a number of technical committees, including the dynamics committee of the ASCE Engineering Mechanics Division, and the committees on seismic effects and seismic performance of bridges of the ASCE Structural Engineering Division. In addition, he is a member of the Transportation Research Board.

Kaizan Huang is a graduate student in the Department of Civil Engineering and Engineering Mechanics at Columbia University, New York, New York. He received his B.S. in Engineering from Tsinghua University, Bejing, P.R. China in 1993 and his M.S. in Engineering from the Florida International University, Miami, Florida in 1995. He is a member of the ASCE, the Civil Engineering Honor Society, and Tau-Beta-Pi.

Principal Investigator's Telephone Number:	(212) 854-6388
Facsimile Number:	(212) 854-6267
E-mail Address:	betti@civil.columbia.edu

SEISMIC ANALYSIS OF BRIDGES INCLUDING SOIL-STRUCTURE INTERACTION EFFECTS

Raimondo Betti and Kaizan Huang
Department of Civil Engineering and Engineering Mechanics
Columbia University, New York, NY, 10027

Research Objectives

In this study, the analysis of the soil-structure interaction effects and the multiple support excitation on the seismic response of a long-span cable-supported bridge is presented. The purpose of such a paper is to provide a better understanding of the dynamic performance of cable-supported bridges under earthquake excitation, considering the spatial variability of the ground motion and the complex phenomena of soil-bridge interaction, so that such effects can be included in the design and rehabilitation processes.

Research Approach

Background: In calculating the bridge seismic response, the assumption of uniform ground motion at the supports of these horizontally extended structures can not be considered valid for the analysis of long–span bridges. First, the bridge may be long with respect to the wavelengths of the ground motion in the frequency range of importance to its earthquake response. In addition, the soil conditions can be quite different from one pier to the other and this drastically alters the input motion. Thus, different portions of the bridge can be subjected to significantly different excitation leading to a complicated dynamic interaction problem between the three-dimensional ground motion inputs and these complex structures. A general procedure is presented to study the dynamic soil-structure interaction effects on the response of long-span suspension and cable-stayed bridges subjected to spatially varying ground motion at the supporting foundations. The foundation system is represented by multiple embedded cassion foundations. The frequency-dependent impedance matrix for the multiple foundations system takes into account also the cross–interaction among adjacent foundations through the soil.

Superstructure In this study, the three–dimensional model of the Vincent-Thomas suspension bridge is used to represent the superstructure. The bridge has a central span of 457.2 m. and two side spans of 152.4 m.. The two central towers (97.3 m. high) and the abutments support the central and side girders and are connected to four embedded foundations. The dimensions of each foundation block are equal to 21.4 $\times$ 30.5, with a depth of 20 m. The superstructure has been represented using an FEM model of 364 joints with 629 elements. Of all these elements, 326 are cable elements which are used to discretize the entire cable system. The stiffness and the mass matrices and the quasi-static functions are determined with respect to the dead-load configuration obtained by an initial nonlinear analysis. A total of 50 modes of vibration has been used to represent the response of the superstructure, in order to provide adequate convergence of the response computations within the range of excitation frequencies considered in this analysis (up to 3 Hz.).

Soil–foundation system With regard to the soil–foundation system, the determination of the impedance matrix for a system of embedded foundations involves the solution of a radiation problem for an excavated half–space. The method used in this analysis represents an alternative formulation of the Substructure Deletion Method considering that the solution of the radiation problem in case of embedded foundations can be derived from the solution of a flat, homogeneous half–space (exterior problem) and from the analysis of the excavated portion of soil (interior problem). For this particular type of embedded foundation system, the effects of the cross–interaction through the soil between adjacent foundations have been considered only for the side spans. Fig. 1 represents the various contribution to the rocking impedance for foundation 1. It is clear that the horizontal components of the motion of the second foundation affect the rocking motion of the first one.

The determination of the foundation input motion requires the solution of a scattering boundary value problem and it can be derived using the results of the analysis of the radiation problem (impedance matrix) and the free–field ground motion. Fig. 2 shows the foundation input motion for foundation 1 subjected to impinging SH-waves propagating along an inclined direction. The angles of incidence in both the horizontal (65°) and vertical plane (20°) have been determined by looking at the location of the bridge with respect to the fault plane.

Expected Results

At this moment, studies are undergoing to combine the two subsystems together so that a global soil-structure interaction analysis can be performed. Comparison of the results with those obtained from an analysis of the superstructure without soil-structure interaction will be conducted to emphasize the differences between the two formulations. The results will prove that the effects of the soil-structure interaction are important and should be included in recommendations for design and rehabilitation.

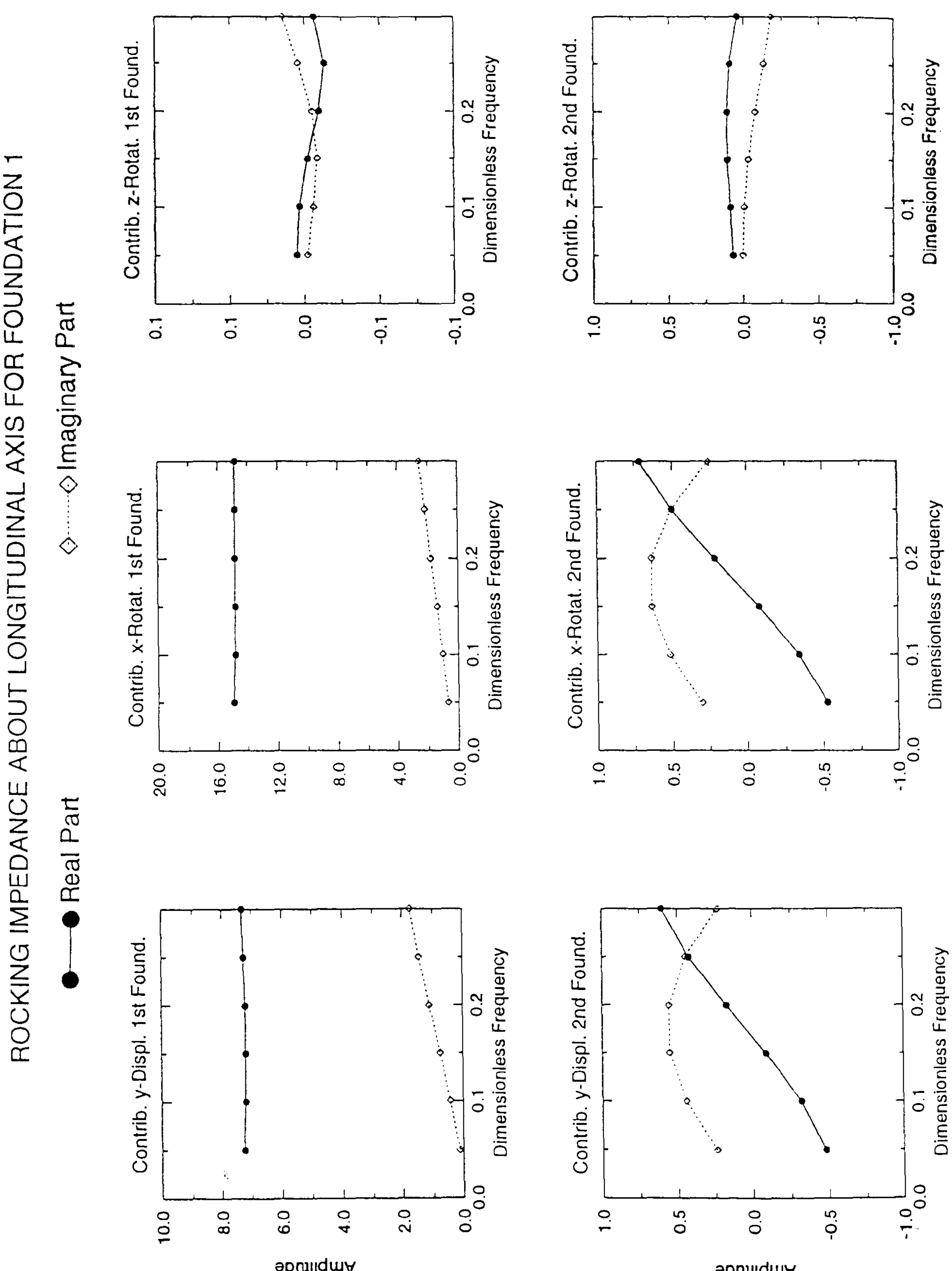
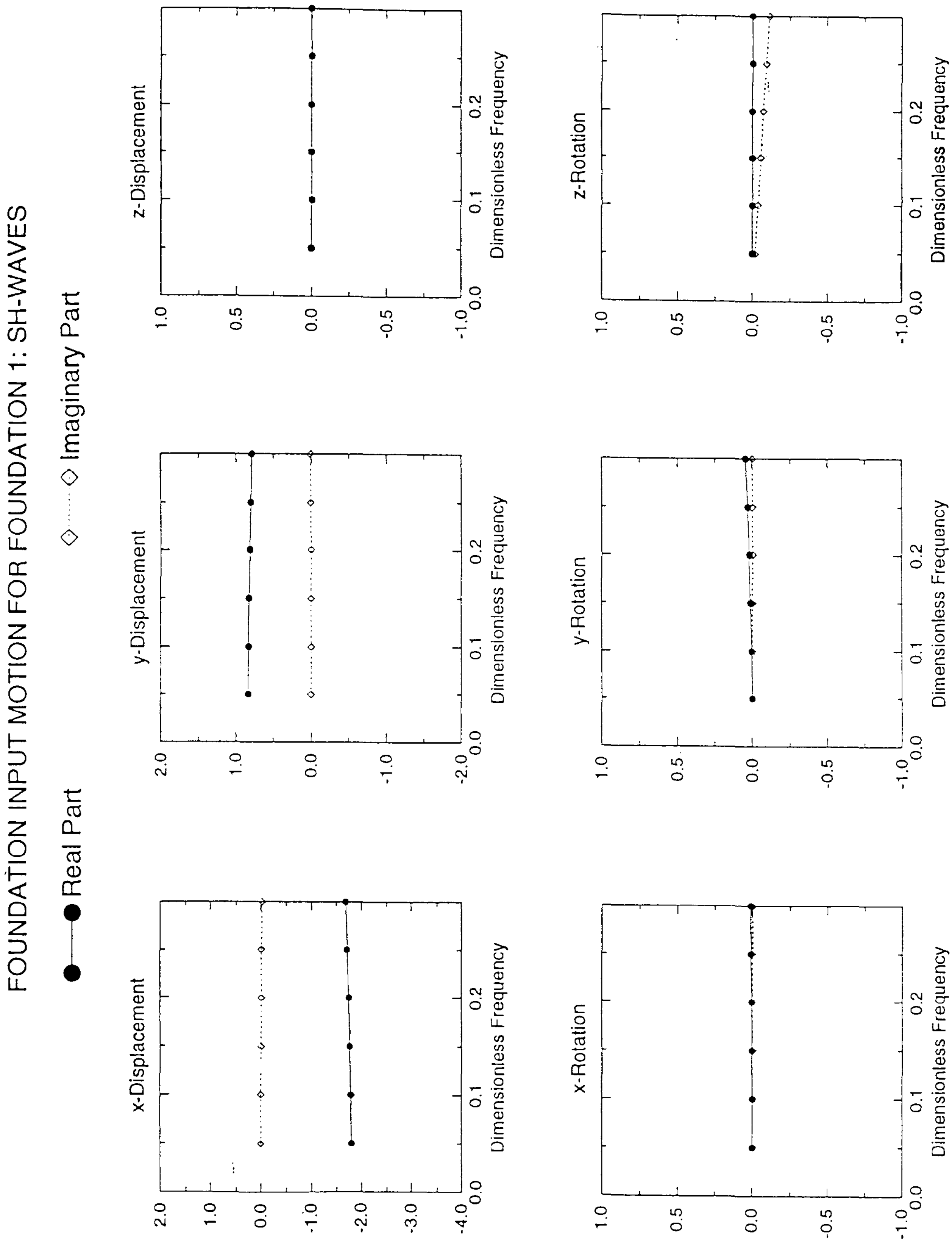

ROCKING IMPEDANCE ABOUT LONGITUDINAL AXIS FOR FOUNDATION 1
Real Part
Imaginary Part
Contrib. y-Displ. 1st Found.
Contrib. x-Rotat. 1st Found.
Contrib. z-Rotat. 1st Found.
Contrib. y-Displ. 2nd Found.
Contrib. x-Rotat. 2nd Found.
Contrib. z-Rotat. 2nd Found.
Amplitude
Dimensionless Frequency

Figure 1



FOUNDATION INPUT MOTION FOR FOUNDATION 1: SH-WAVES
Real Part
Imaginary Part
x-Displacement
y-Displacement
z-Displacement
x-Rotation
y-Rotation
z-Rotation
Amplitude
Dimensionless Frequency

Figure 2

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Practical Nonlinear Time-History Analysis for the Design of Bridge Structures Subjected to Strong Seismic Motion

Author(s) and Affiliation(s):

Robert Dowell, Frieder Seible and M. J. Nigel Priestley
Department of Structural Engineering
University of California, San Diego, La Jolla, CA

Principal Investigator: Frieder Seible

Sponsor(s): California Department of Transportation

Research Start Date: 1994
Expected Completion Date: 1997

Research Objectives:

The primary objective of the current research is to investigate the practicality of replacing traditional elastic seismic response bridge analysis with more refined nonlinear time-history methods, with emphasis on reinforced concrete bridge structures. Currently, such efforts are only employed for large or important structures, leaving the majority of bridges to be designed by linear elastic methods. The resistance to adopt nonlinear time-history methods for the design of bridges is understandable, considering the added effort required. However, the benefits of such analyses, even for simple structures, may outweigh the added expenditure as long as reliable analytical tools and analysis procedures are available. The development of required analytical tools for the current research is discussed.

Expected Products or Deliverables:

It is expected that through the use of the new fiber model (extending the existing experimental database), the required parameters of the pivot hysteresis model will be provided in graphical form, simply as a function of longitudinal steel percentage and axial load ratio (for a given section). It is hoped that this research will lead to wider acceptance of nonlinear tools for the design of bridges subjected to large seismic motion.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Robert Dowell is a Ph.D. candidate at the University of California, San Diego. He received his BSCE from San Diego State University in 1988 and his MS in structural engineering from the University of California, San Diego in 1995. He worked as a Bridge Engineer with Caltrans from 1988 to 1993. Mr. Dowell's research interests are in seismic behavior of reinforced concrete bridges and the application of advanced nonlinear time-history methods for the design of typical bridges subjected to large seismic events.

Frieder Seible is Professor of Structural Engineering at the University of California, San Diego. He received his Dipl. Ing. (Civil Engineering) from the University of Stuttgart, West Germany in 1976; his MSCE from the University of Calgary, Alberta, Canada in 1978; and his Ph.D. from the University of California, Berkeley in 1982. His research interests are in analysis and design of reinforced/prestressed concrete bridges, evaluation and rehabilitation of existing bridge structures and buildings, and development of new computer models to predict dynamic and static nonlinear response of reinforced and prestressed concrete structures under service overload and failure loads. Dr. Seible also has experience in verification of computer models by means of large or full-scale experimental testing, earthquake resistant design of reinforced concrete and concrete masonry structures, development of large-scale structural testing techniques, seismic assessment and retrofit of bridges, and the application of Polymer Matrix Composites (PMC) in civil structures. He has more than 170 papers and 90 technical reports mainly related to seismic design of bridges and buildings. Dr. Seible is a registered Professional Engineer in California.

M.J. Nigel Priestley is Professor of Structural Engineering at the University of California, San Diego. He received his B.E. in 1964 and his Ph.D. in 1967 in Civil Engineering from the University of Canterbury, New Zealand. His primary research interests are related to the design and analysis of structures to withstand seismic attack. In particular, his current research emphasis is on seismic assessment and retrofit of existing bridges; seismic design of concrete bridges; seismic design philosophy for precast concrete structures; seismic design of reinforced and unreinforced masonry structures; and the development of realistic dynamic testing methods to simulate inelastic response under seismic loading.

Principal Investigator's Telephone Number:	(619) 534-4640
Facsimile Number:	(619) 534-6373
E-mail Address:	seible@ucsd.edu

PRACTICAL NONLINEAR TIME-HISTORY ANALYSIS FOR THE DESIGN OF BRIDGE STRUCTURES SUBJECTED TO STRONG SEISMIC MOTION

Robert Dowell[1], Frieder Seible[2] and M.J. Nigel Priestley[2]

[1]Graduate Research Assistant, University of California, San Diego, La Jolla, CA 92092-0085
[2]Professor of Structural Engineering, University of California, San Diego, La Jolla, CA 92092-0085

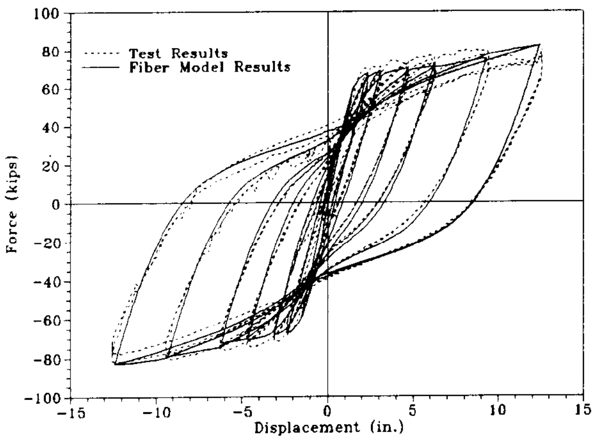
Research Objectives

The primary objective of the current research is to investigate the practicality of replacing traditional elastic seismic response bridge analysis with the more refined nonlinear time-history methods, with emphasis on reinforced concrete bridge structures. Currently, such efforts are only employed for large or important structures, leaving the majority of bridges to be designed by linear elastic methods. The resistance, by the profession, to adopt nonlinear time-history methods for the design of bridges is understandable, considering the added effort required. However, the benefits of such analyses, even for simple structures, may outweigh the added expenditure as long as reliable analytical tools and analysis procedures are available. This paper discusses the development of the required analytical tools for the current research.

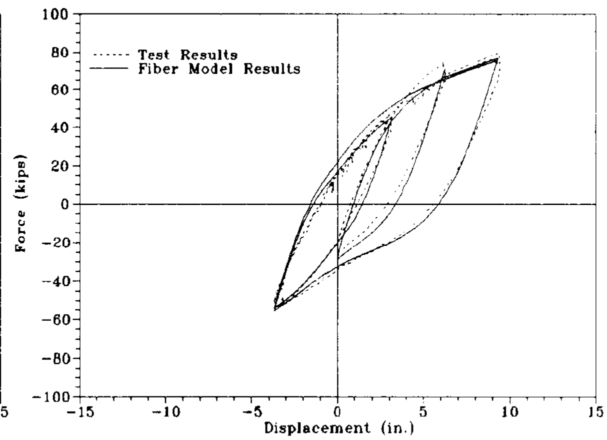
Research Approach

Background: For nonlinear time-history analysis to be used for design on a daily basis, it must be practical. This requires the computer program to have simple input and output, and clearly defined nonlinearities. Since new bridge structures are typically designed based on capacity design principles, forcing all member nonlinearities into the ends of ductile columns, only a few, predetermined, nonlinear elements are required. This is what makes the idea of using nonlinear analysis for bridges so attractive. Other nonlinearities required in the analysis include, opening and closing of expansion joints, and failure of soil behind abutments. Typically, column plastic hinges are included in a nonlinear model by use of a force-displacement or moment-rotation hysteresis model. The shortcoming of many of these models is that a varying axial load (transverse bridge response), non-symmetric section (different amount of tension steel in the two loading directions), and sudden strength loss (shear or confinement failure) are not available. Also, determination of the parameters which define the hysteretic behavior is often based on experimental results, which may not be available for the member in question (percentage of steel, axial load ratio, and section shape), especially for the general bridge designer. To accurately capture the nonlinear response of a bridge structure, a new hysteresis model is required which addresses the shortcomings listed above. Also, a fiber model is required to develop all relevant parameters of the hysteresis model (since the experimental database is not complete).

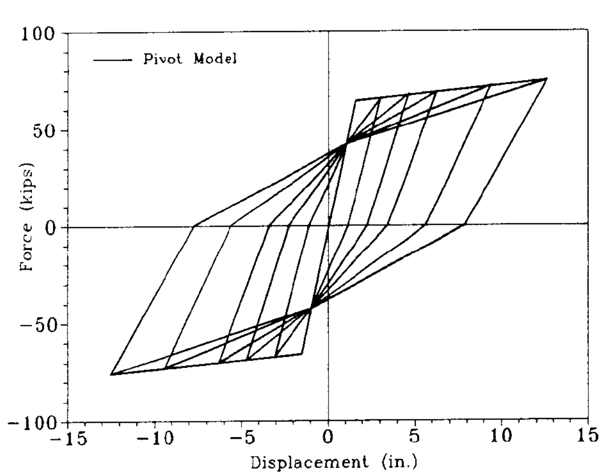
Fiber Model: A new fiber model is developed to directly provide all the required Pivot Hysteresis Model (described below) parameters for any section, steel percentage, and axial load ratio. The cyclic stress-strain rules of the concrete fibers are a simplified version of that presented by Mander et al. (1), with one important exception: the wedging action of concrete. Following large tensile strains, compressive stresses occur prior to closure of the initial gap. This has been reported by Bolong et al. (2), and thought to be attributed to aggregate dislocation in the cracks. Without the wedging action of concrete, the response is severely pinched.



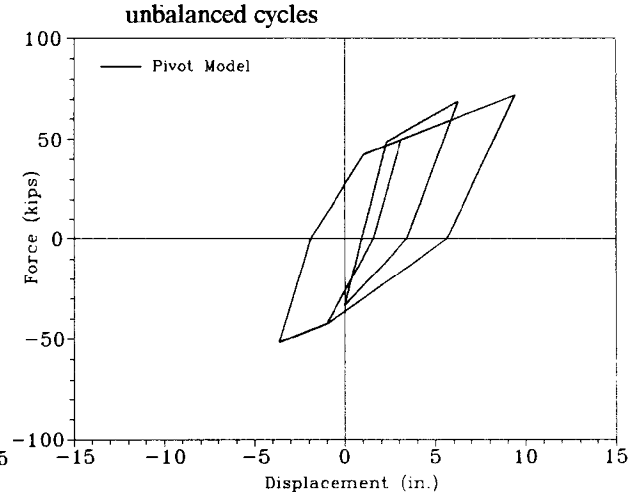
(a) Experimental and fiber model results for complete test



(b) Experimental and fiber model results for unbalanced cycles

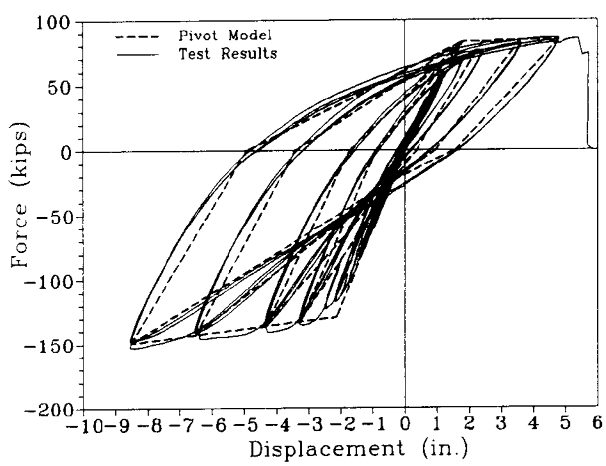


(c) Behavior of Pivot Model for complete test

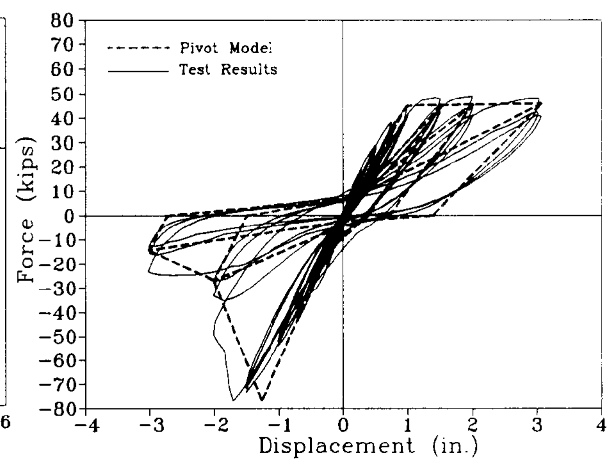


(d) Behavior of Pivot Model for unbalanced cycles

Figure 1 Experimental, fiber model, and Pivot Model results for the Cantilever Column Test

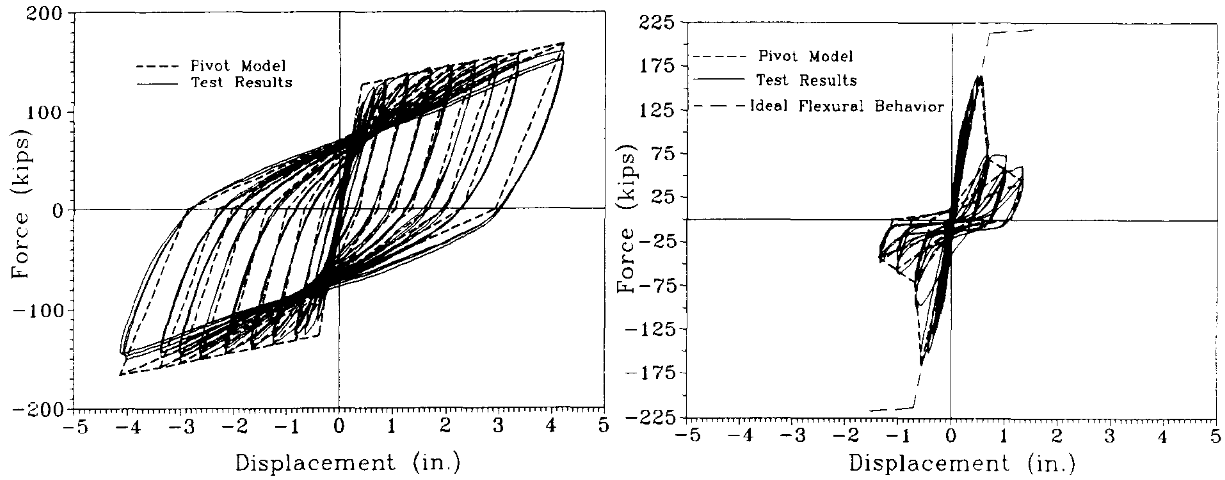


(a) Retrofitted specimen



(b) Original specimen

Figure 2 Experimental and Pivot Model results for Outrigger bent cap tests



(a) Flexural response of Retrofitted shear Column (b) Shear response of unretrofitted shear column

Figure 3 Experimental and Pivot Model results for shear column tests

The cyclic stress-strain rules of the reinforcing steel are similar to those presented by Tjokrodimuljo (3). However, the shape of the Bauschinger curve is governed by parameters used by Taucer et al. (4), utilizing the model first proposed by Menegotta and Pinto (5). Also, the softening of the modulus of elasticity (with maximum plastic strain) suggested by Dod and Restrepo (6) is included. This has an important effect on the unloading stiffness of members, causing a much softer response. The ability of the fiber model to capture the cyclic response of a symmetric (circular with evenly distributed steel), cantilever reinforced concrete column, described elsewhere (7), is demonstrated in Figures 1a and 1b. The agreement is excellent. Note that the unbalanced cycles are not shown in Figure 1a. For this analysis the column section was discretized by 10 slices (10 steel fibers, confined concrete fibers, and unconfined concrete fibers).

Pivot Hysteresis Model: A new hysteresis model has been developed which accurately responds to a varying axial load and/or sudden strength degradation for symmetric or nonsymmetric sections (8). It is a very simple model with very few rules to follow. The parameters which define the model response may be found by experiment and/or analysis. However, it is more desirable to obtain the parameters analytically since there are many gaps in the experimental database. Although the new fiber model is in the developmental stages, it can accurately produce all the required Pivot Model parameters for a symmetric section (as witnessed by the excellent comparison to the tested column response in Figures 1a and 1b). The behavior of the calibrated Pivot Hysteresis Model is demonstrated for a symmetric column in Figures 1c and 1d. The overall response is very similar to the experimental and fiber model results. Remaining comparisons between Pivot Model and experimental responses are provided to show the possible behavior of the model. As the fiber model is still under development, the hysteresis parameters were obtained from the noted references or directly from the test data. Two outrigger bent tests, reported by Ingham (9), are compared against the Pivot Model response in Figure 2. They have a different initial stiffness, and plastic moment capacity, in the two loading directions. This is attributed to a varying axial load, and different amount of tension steel in the top and bottom of the cap beam. Figure 2a demonstrates that the Pivot Hysteresis Model does accurately trace the measured cyclic response of the retrofitted outrigger. The original outrigger specimen suffered from joint degradation in one direction only, resulting in an interesting hysteretic response with uni-directional strength degradation. As shown in Figure 2b, the Pivot Model response is very similar to the experimental response. Figure 3 shows how the Pivot Model can simulate a column failing in shear, and a retrofitted shear column which responds in flexure. Note that the retrofitted shear column went to a displacement ductility of 10, providing a great opportunity to compare the response of the Pivot Model through the entire range of practical displacements. The shear column tests are described elsewhere (10).

Expected Products

It is expected that through the use of the new fiber model (extending the existing experimental database), the required parameters of the Pivot Hysteresis Model will be provided in graphical form, simply as a function of longitudinal steel percentage and axial load ratio (for a given section). It is hoped that the present research will lead to wider acceptance of nonlinear tools for the design of bridge structures subjected to large seismic motion.

Preliminary Results

The Pivot Model has the ability to closely mimic the response of very complicated hysteretic member behavior, including a varying axial load, nonsymmetric section, and sudden strength degradation. The required parameters of the Pivot Model may be found directly from the new fiber model (currently for symmetric sections only).

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FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: On Near-Field Earthquake Ground Motion Simulation and its Effects on Long-Period Structures

Author(s) and Affiliation(s):

Ruichong Zhang and Masanobu Shinozuka
Department of Civil Engineering
University of Southern California, Los Angeles, CA

Principal Investigator: Masanobu Shinozuka

Sponsor(s): Federal Highway Administration and National Center for Earthquake Engineering Research

Research Start Date: January 1996
Expected Completion Date: December 1996

Research Objectives:

Since the occurrence of large earthquakes in urban areas is highly likely in California (e.g. the 30-year probability for the San Francisco Bay area to be subjected to an earthquake with a magnitude of 7.0 or larger has been estimated to be about 67%), it is important to study the nature of the low-frequency components of the near-field earthquake ground motion and its effects on long-period structures. In this project, near-field Loma Prieta earthquake is simulated with the aid of a seismologically consistent earthquake ground motion model as well as record near-field ground motion data. The simulated earthquake ground motion is then used for computing response spectra, and for seismic response analysis of a realistic bridge with a long period in order to gain insight to both physical and engineering significance of the effects of near-field low-frequency earthquake ground motion on flexible structures.

Expected Products or Deliverables:

This study is expected to show the extent of potential underestimation of response spectra with the use of observed ground motion and the effects of source information such as rupture pattern, slip distribution, and fault directivity on response spectra. Consequently, the earthquake-resistant capability of flexible structures, particularly base-isolated ones, to a near-field ground motion can be known. Moreover, seismic responses of realistic bridges to the simulated near-field Loma Prieta-type earthquake ground motions will be carried out.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Ruichong Zhang is a Research Assistant Professor at the University of Southern California. He received his B.S. and M.S. degrees from Tongji University, Shanghai, China in 1984 and 1987 respectively, and his Ph.D. degree from Florida Atlantic University in 1992. Dr. Zhang's professional experience includes: Post-Doctoral Research Associate, Princeton University, 1992-1995; Research Assistant, Florida Atlantic University, 1988-1992; and Research Assistant, University of Colorado at Boulder, 1987-1988. He is a member of ASCE, ASME, and CUREe.

Masanobu Shinozuka is Fred Champion Chair in Civil Engineering at the University of Southern California. He received his B.S. and M.S. degrees from Kyoto University, Kyoto, Japan in 1953 and 1953 respectively, and his Ph.D. in 1960 from Columbia University. He is registered as a Professional Engineer in Washington, D.C. His professional experience includes: Sollenberger Professor of Civil Engineering, Princeton University, 1989-1995; Director, National Center for Earthquake Engineering Research at SUNY/Buffalo, 1990-1992; Visiting Capen Professor of Structural Engineering, SUNY/Buffalo, 1990-1992; Professor of Civil Engineering, Princeton University, 1988-1989; Renwick Professor of Civil Engineering, Columbia University, 1977-1988; Professor of Civil Engineering, Columbia University, 1969-1977; Associate Professor of Civil Engineering, Columbia University, 1965-1969; and Assistant Professor of Civil Engineering, Columbia University, 1961-1965. He is an Honorary Member of ASCE, a Fellow of ASME, and a Senior Member of AIAA. Dr. Shinozuka's honors and awards include: Member of National Academy of Engineering, 1978; ASCE Theodore von Karman Medal, 1994; Wessex Institute of Technology Medal, 1991; ASCE C. Martin Duke Award, 1991; ASCE Moisseiff Award, 1988' ASCE Nathan M. Newmark Medal, 1985; Ticinense Medal, University of Pavia, Pavia, Italy, 1985; ASCE Alfred M. Freudenthal Medal, 1978; and ASCE Walter L. Huber Civil Engineering Research Prize, 1972. He is a holder of U.S. Patent No. 4,228,759 "Pressure Sustaining Vessel".

Principal Investigator's Telephone Number: (213) 740-9528
Facsimile Number: (213) 740-9529
E-mail Address: shinozuk@mizar.usc.edu

ON NEAR-FIELD EARTHQUAKE GROUND MOTION SIMULATION AND ITS EFFECTS ON LONG-PERIOD STRUCTURES

Ruichong Zhang and Masanobu Shinozuka

Department of Civil Engineering
University of Southern California, Los Angeles, CA 90089-2531.

Research Objectives

Near-field Loma Prieta earthquake is simulated with the aid of a seismologically consistent earthquake ground motion model as well as the near-field ground motion record data. The simulated earthquake ground motion is then used for generating the response spectra. It will also be used for seismic response analysis of a realistic bridge with a long period to gain insight to both physical and engineering significance of the effects of near-field low-frequency earthquake ground motion on flexible structures, particularly on those base-isolated.

Research Approach

Background: Recent large earthquakes, especially the Northridge and the Kobe earthquake, have provided an extremely good lesson to both seismologists and engineers. Firstly, large near-field earthquakes can damage severely the structures designed, constructed and even retrofitted on the basis of seismic codes and guidelines, when they are subjected to near-field earthquakes. Secondly, the nature of near-field ground motion strongly depends on seismic source characteristics such as location, directivity and pattern of shear dislocation. Consequently, the study of the effect of near-field earthquakes on structures is much more difficult and complex than for far-field earthquakes. Thirdly, the existing records associated with the near source quite often distort the nature of the shaking, especially for low-frequency components due to the high-pass filtering in processing the data obtained from seismometers of earlier vintage. The low-frequency ground motion is believed to be capable of causing significantly high percentage of story drift in case of high-rise buildings or very large pad displacement in case of base-isolated buildings (Heaton et al.[1]). Since the occurrence of large earthquakes in urban areas is highly likely in California (e.g. the 30-year probability of San Francisco Bay area to be subjected to an earthquake with a magnitude being 7.0 or larger has been estimated to be about 67%), it becomes important to study the nature of the low-frequency components of the near-field earthquake ground motion and its effects on long-period structures.

Simulation of Near-Field Ground Motion: In this research, the near-field ground motion due to the Loma Prieta earthquake is first simulated, which consists of two parts. The low-frequency part (up to 1 Hz) is synthesized using software SEISMO (developed by Shinozuka and his colleagues at both Princeton University and the University of Southern

[1]Heaton, T.H., Hall, J.F., Wald, D.J. and Halling, M.W. (1995). in *Science*, Vol. 267, January 13, pp. 206-211.

California) in which discrete wave number method is applied to solve a 3D wave propagation problem in a layered half-space subjected to a seismologically-consistent extended shear-dislocation source. SEISMO currently runs on a parallel supercomputing platform and generates the earthquake motion time histories including permanent deformation. The high-frequency part is obtained by modifying real earthquake records by deleting the corresponding low-frequency part from the recorded data and adjusting peak magnitude on the basis of the relative distance away from the observation sites. The reasons why the synthetics and filtered recorded data are combined to form the simulated ground motion are detailed as follows. (1) Many studies indicated that the synthetics of the ground motion by using theoretical models can successfully predict the ground motion in the low-frequency range (usually less than 1.5 Hz). However, the synthetics of higher frequency motion are either time-consuming using the theoretical models or not as good as the prediction of low-frequency motion. (2) The existing records particularly at near field quite often distort the nature of the shaking, especially for low-frequency motion. For example, some accelerometers do not record very low frequency components (e.g. less than 0.1 Hz). In addition, the relatively low frequency components (e.g. less than 0.3 Hz) of the recorded data are underestimated (Iwan and Chen[2]). (3) As far as the long-period structures are concerned, the high-frequency motion is not as important as the low-frequency, especially beyond the frequency larger than 1 Hz.

Effects on Long-Period Response Spectra: The simulated and observed ground motion at selected observation sites are compared in this study. The corresponding response spectra to both simulated and observed ground motion are then calculated so that the low-frequency motion effects on long-period structures can be analyzed. The seismic responses of a realistic bridge to the simulated near-source Loma Prieta-type earthquake ground motion will continue to be carried out (Zhang and Deodatis[3]) to gain insight to and understanding of both physical and engineering significance of such effects.

Expected Products

In this study, six observation sites located within 20 km around the epicenter are selected in relation to the Loma Prieta earthquake. They are Lexington Dam (LEX), Watsonville (WAT), Capitola (CAP), Corralitos (COR), San Jose (SJS) and Santa Cruz (UCS) as shown in Fig. 1. Since these observation sites are located around the epicenter at some typical directions, they are considered to be representative in the study on effects of near field ground motion on structures, including the rupture directivities, slip distribution, observation site location etc. Consequently, the effects of the near-field low-frequency ground motion on the long-period structures can be investigated in a systematic fashion.

Due to the high-pass filtering in processing the data obtained from the seismometers of earlier vintage, the observed ground motion can not always be adequately used to examine its effects on flexible structures. This study thus attempts to show the extent of the possible

[2]Iwan, W. and Chen, X. (1995) in *Proc. 10th Euro. Conf. on Earthquake Eng.*, Austria.
[3]Zhang, R. and Deodatis, G. (1996) in *Int'l J. of Earthquake Eng. and Struc. Dynamics.*

underestimation of response spectra with the use of observed ground motion, effects of source information such as rupture pattern, slip distribution as well as directivity of the fault on the response spectra. The research therefore contributes to the state of knowledge of the earthquake-resistant capability of the flexible structures to a near-field ground motion and ensures more adequate earthquake-resistant structural design and seismic retrofitting under a near-field earthquake.

Preliminary Results

The preliminary results at Lexington Dam (19 *km* away from the epicenter) are presented in Figs. 2 and 3. Although the peak values of the simulated ground motion in horizontal direction at Lexington Dam are somewhat smaller than the recorded data (see Fig. 2), the richer low-frequency contents of the ground motion can be observed in the simulation than in the recorded data, resulting in the larger acceleration responses at moderate to long period (1.5 to 6 *sec*) under simulated motion (see plots on the left hand side of Fig. 3). This means that the use of recorded data underestimates the response of structures with a moderate to long period (1.5 to 6 *sec*). Fig. 3 (right) also shows the acceleration response spectra at Corralitos (COR).

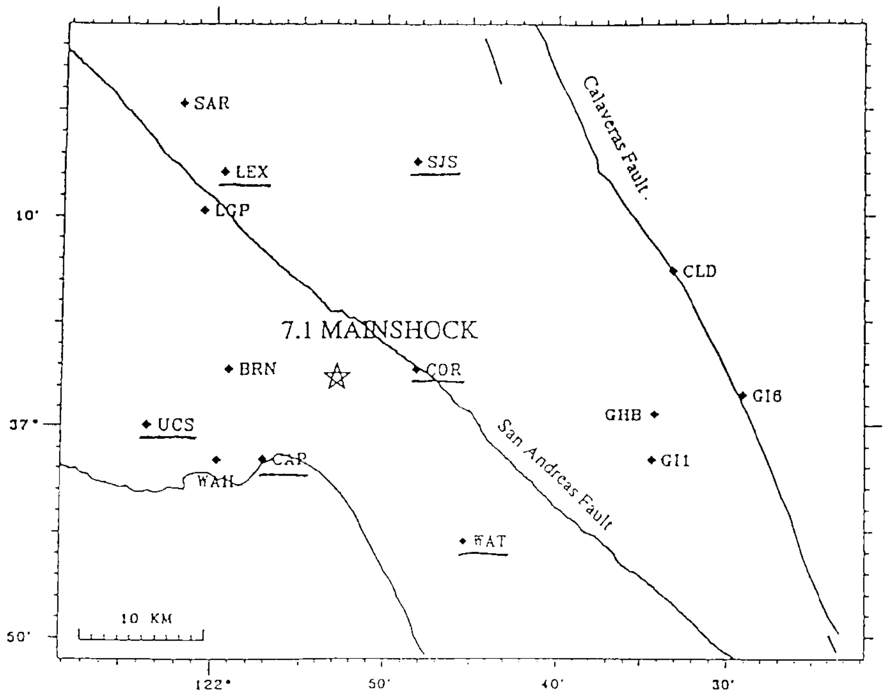

Fig. 1 Map view of strong motion stations for Loma Prieta earthquake

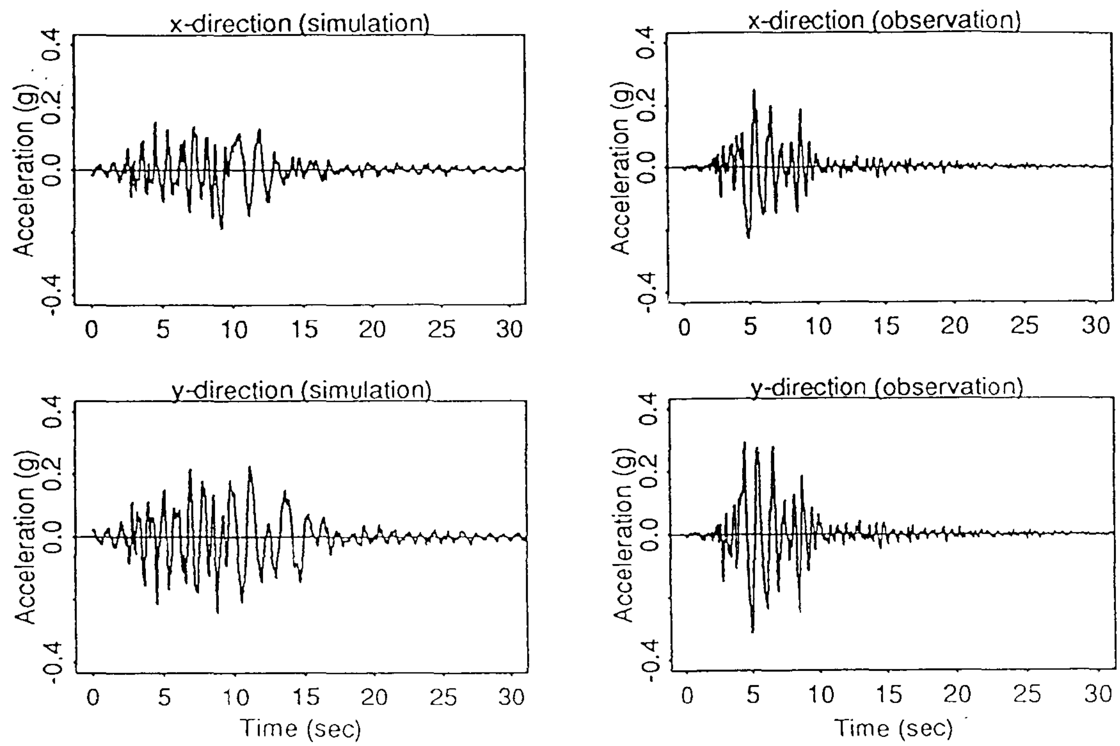

Fig. 2 Simulateed and recorded earthquake ground motion at Lexington Dam

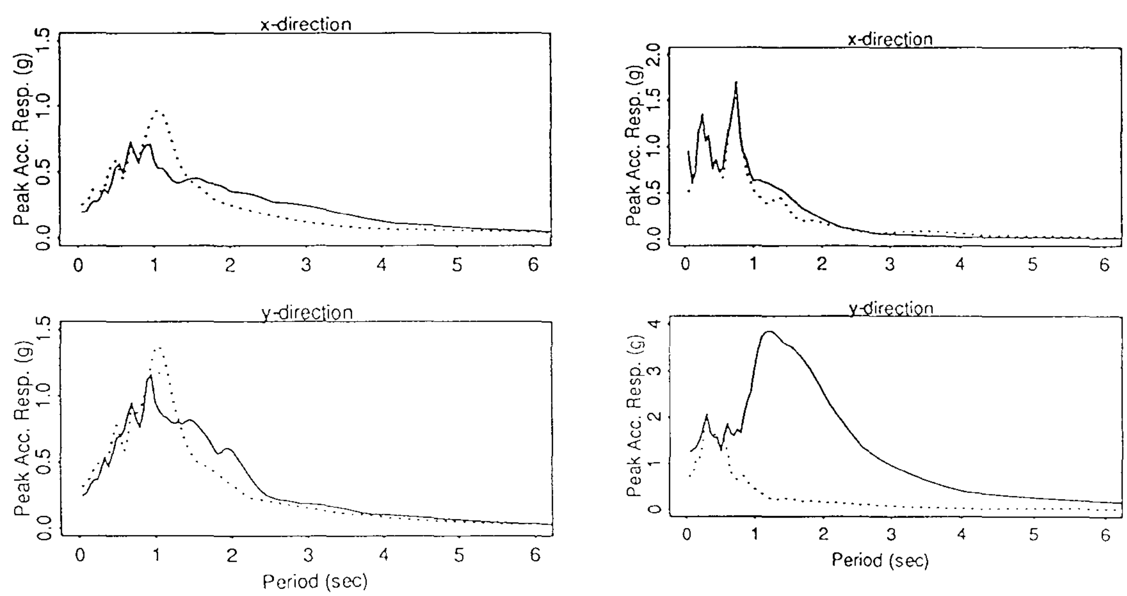

Fig. 3 Acceleration response spectra with damping ratio being 5 % at LEX (left) and at COR (right). (solid line is associated with simulated motion and dotted line with recorded motion)

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Evaluation of Bridge Abutment Stiffness During Earthquakes

Author(s) and Affiliation(s):

Rakesh K. Goel
Department of Civil and Environmental Engineering
Syracuse University, Syracuse, NY

Principal Investigator: Rakesh K. Goel

Sponsor(s): Currently unfunded (Initial research was jointly supported by CSMIP and Caltrans with Dr. A.K. Chopra, University of California, Berkeley, as PI)

Research Start Date: June 1993
Expected Completion Date: June 1998

Research Objectives:

The objectives of the research are to develop a simple procedure for estimating stiffness of abutments in short-span bridges directly from their motions recorded during earthquakes, and to evaluate and improve the current abutment modeling procedures for earthquake analysis and design of such bridges.

Expected Products or Deliverables:

The products expected are a simple analytical procedure to estimate the abutment stiffness of short bridges directly from their motions recorded during earthquakes, an evaluation of current industry practice on estimating the stiffness of abutments in bridges with integral abutments, and recommendations for improving industry practice.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Rakesh Goel is Assistant Professor of Civil Engineering in the L.C. Smith College of Engineering of Syracuse University, where he has taught since 1995. Prior to that he was an assistant research engineer at the Earthquake Engineering Research Center of the University of California, Berkeley. He has a B. Tech. in civil engineering (Indian Institute of Technology, Delhi, 1982), an MS in structural engineering (University of California, Berkeley, 1985), and a Ph.D. in structural engineering (University of California, Berkeley, 1990). Dr. Goel is a registered Professional Engineer in California and is a member of the Earthquake Engineering Research Institute and ASCE.

Principal Investigator's Telephone Number: (315) 443-3347
Facsimile Number: (315) 443-1243
E-mail Address: rkgoel@summon2.syr.edu

EVALUATION OF BRIDGE ABUTMENT STIFFNESS DURING EARTHQUAKES

Rakesh K. Goel

Department of Civil and Environmental Engineering
Syracuse University, Syracuse, NY 13244-1190

Research Objectives

The objectives of the research described in this paper are: (1) to develop a simple procedure for estimating stiffness of abutments in short-span bridges directly from their motions recorded during earthquakes; and (2) to evaluate and improve the current abutment modeling procedures for earthquake analysis and design of such bridges.

Research Approach

Background: Most specifications and guidelines for earthquake design of highway bridges require that abutment-soil systems be included in the analytical model as discrete equivalent linear springs. In design applications, stiffness values of these springs are determined from one of the two iterative procedures:

- The procedure starts with an initial estimate of the abutment stiffness that is obtained from simplified rules involving abutment dimensions and soil properties. The analysis is repeated by successively reducing the stiffness until earthquake-induced force in the abutment becomes smaller than the abutment capacity.
- The procedure starts with an initial estimate of the abutment stiffness equal to the abutment capacity divided by expected abutment deformation during the design earthquake. The bridge system is analyzed to compute the abutment deformation and the abutment stiffness recalculated using the computed deformation. The analysis is repeated until abutment deformations from two successive analyses are within acceptable tolerance.

It is not entirely clear how well the stiffness value thus determined represents the complex behavior of the abutment-soil system, which is influenced by soil-structure interaction and nonlinear behavior of the soil. Therefore, it would be useful to determine and compare 'actual' values of the stiffness and capacity of abutment-soil systems during earthquakes with their 'design' values.

Method of Analysis: The 'actual' values of capacity and stiffness of abutment-soil systems of a continuous two-span bridge with integral abutments are determined from its ground and structural motions recorded during a significant earthquake event. The bridge considered in this study is the US 101/Painter Street Overpass located in Rio Dell California (Figure 1); and recorded motions selected are those obtained during main shock of the April 25, 1992, Cape Mendocino/Petrolia earthquake. To estimate the abutment stiffness, the bridge is idealized as a simple system consisting of the following:

- Road deck girder.

- Spring-dampers along the east abutment, normal to the east abutment, and along the west abutment. The spring represents the stiffness property and the damper accounts for material and radiation damping of the abutment-soil system.
- Two linear elastic springs -- one normal to and the other along the bent -- at location of each column in the central bent; no damper is included because the energy dissipation in the bent should be negligible.

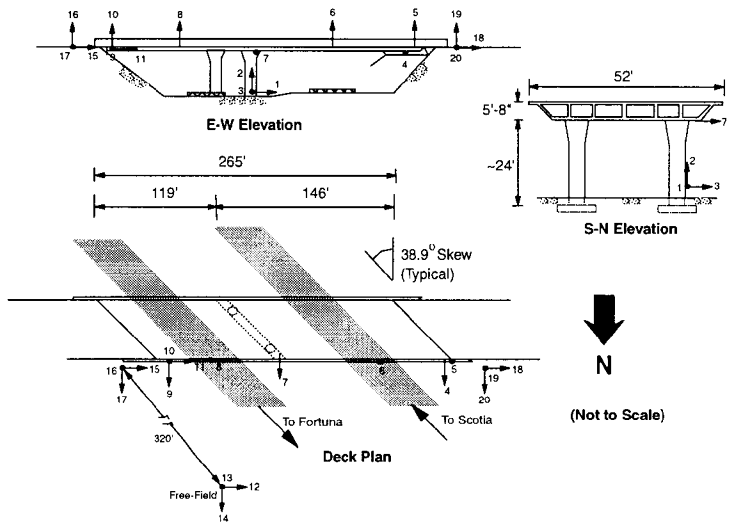

Figure 1. US 101/Painter Street Overpass: Structural details and sensor locations.

The 'actual' stiffness and capacity values of each abutment are estimated from force-deformation relation of the associated spring-damper system. The forces in the three spring-damper systems are obtained by solving three equations of dynamic equilibrium, corresponding to two translational motions and one rotational motion about vertical axis of the road deck, at each instant of time. In these equations, inertia forces of the road deck are computed from its mass properties and recorded accelerations, and column forces are calculated from their known stiffness and deformation values. The deformations at desired locations are obtained by subtracting the free-field displacement from the total displacements calculated by appropriately transforming the recorded motions.

The 'actual' capacity of the abutment-soil system is the yield strength displayed by a flat yield plateau in the force-deformation relation. The 'actual' stiffness of the abutment-soil system is the slope of the force-deformation relation. For linearly visco-elastic behavior, the abutment stiffness is the slope of the major axis of the ellipse in an individual force-deformation loop. For nonlinear behavior, the abutment stiffness is the secant slope of the force-deformation loop.

<u>Evaluation of Current Modeling Procedures:</u> The 'actual' values of abutment stiffness for the US 101/Painter Street Overpass have been obtained by using the above-described procedure. These 'actual' values are then compared with the 'design' values computed by the CALTRANS, AASHTO-83, and ATC-6 procedures. In the CALTRANS procedure, the 'design' value of the abutment stiffness is computed as the ratio of its 'design' capacity and the acceptable

deformation. The abutment capacity is calculated based on the ultimate passive resistance of the backfill equal to 7.7 ksf and failure capacity of each pile equal to 40 kips. Two values of the acceptable abutment deformations are considered: 1 inch and 2.4 inch. The first represents the deformation at which the soil pressure reaches its peak value of 7.7 ksf and the latter represents the limiting value corresponding to incipient damage to the abutment. In the AASHTO-83 and ATC-6 procedures, which are identical, the initial estimate of the abutment is obtained by adding the contributions of the backfill and of the piles. The stiffness due to the backfill is $0.425 \times E_s \times B$, in which $E_s = 1440$ ksf is the elastic modulus of the soil and B is the width of the backwall or effective length of the backwall or the wingwall. The stiffness of each pile is assumed equal to 40 kips/inch.

Expected Products

The following products are expected from the research described in this paper:

- A simple analytical procedure to estimate the abutment stiffness of short bridges directly from their motions recorded during earthquakes. This procedure is appealing because it provides the most direct and economical means of estimating the actual abutment stiffness that includes effects of soil-structure interaction as well as nonlinear behavior of the soil.

- Evaluation of current industry practice on estimating the stiffness of abutments in bridges with integral abutments. The results of this research would also lead to recommendations on improving such industry practice.

Preliminary Results

Compared in Figures 2 and 3 are the 'actual' and 'design' values of the abutment stiffness. The upper and lower bound values of the 'actual' abutment stiffness estimated at various time-instances during the earthquake shaking are shown in solid circles connected by a vertical line whereas the 'design' values are shown either by solid or dashed lines. These results lead to the following conclusions:

Conclusions:

- CALTRANS procedure may overestimate the stiffness normal to the abutment (along the road deck) by a factor of over two. This is indicated by CALTRANS 'design' values for 1 inch abutment deformation being much higher than the 'actual' values. It is appropriate to compare the values for 1 inch deformation, not for 2.4 inch deformation, because 'actual' abutment deformations during the earthquake were of the order of 1 inch.

- CALTRANS procedure leads to a good estimate of the abutment stiffness in the direction along the abutment (transverse to the road deck). The 'actual' values presented in Figure 3 match well with the 'design' values computed for the assumed abutment deformation of 2.4 inch; the 'actual' abutment deformations during the earthquake were also close to 2.4 inches.

- AASHTO-83 and ATC-6 procedure gives an initial estimate of abutment stiffness that is too large in both directions. This is especially so during strong shaking phase of the earthquake.
- AASHTO-83, ATC-6, and CALTRANS procedures would give identical values for the final stiffness because the abutment capacities from these procedures would be identical.
- These conclusions are preliminary in nature because they are based on results from recorded motions of one bridge during single earthquake.

<u>Research In Progress:</u>

- The value of 7.7 ksf for the ultimate passive resistance of the soil used in the CALTRANS procedure appears to be too high and this issue is being further examined.
- Other similar bridges would be investigated to verify these conclusions and to develop generally applicable conclusions that can be used to improve the current design procedures.

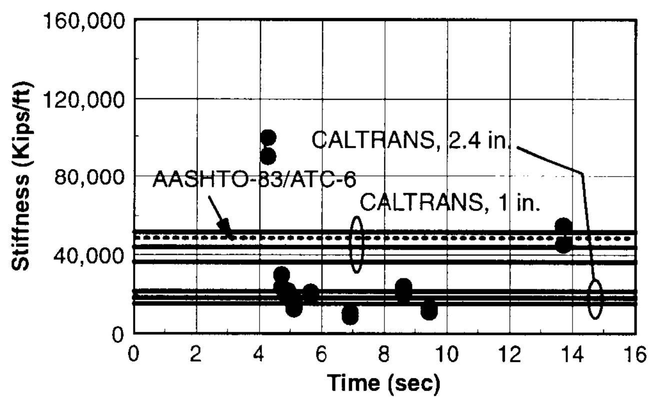

Figure 2. Comparison of 'actual' and 'design' values of longitudinal abutment stiffness (along the road deck); main shock of the 1992 Cape Mendocino/Petrolia earthquake.

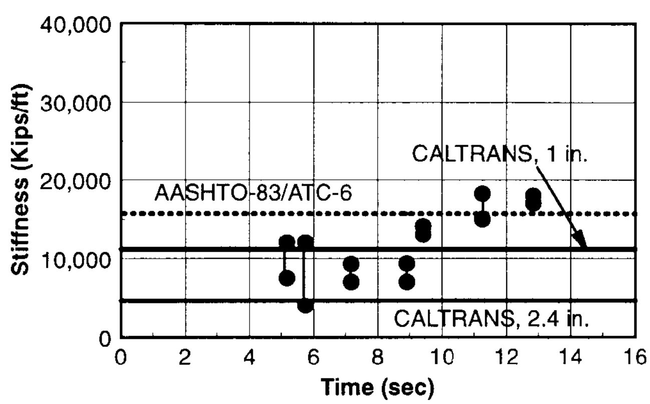

Figure 3. Comparison of 'actual' and 'design' values of stiffness along the west abutment (transverse to the road deck); main shock of the 1992 Cape Mendocino/Petrolia earthquake.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Characterization of Nonlinear Abutment Stiffnesses for Seismic Design and Retrofit

Author(s) and Affiliation(s):

Raj V. Siddharthan and Mahmoud El-Gamal
University of Nevada, Reno, Reno, NV

Principal Investigator: Raj V. Siddharthan

Sponsor(s): National Science Foundation and Nevada Department of Transportation

Research Start Date: March 1994
Expected Completion Date: August 1996

Research Objectives:

Seismic response analyses of highway bridges are often necessary in the seismic safety evaluations of existing bridge structures and new bridge designs. Such studies require realistic characterization of bridge deck-abutment interaction. The current procedure models the deck-abutment interaction using translational springs (longitudinal, lateral, and vertical) located at the deck-abutment support. Past studies that dealt with abutment stiffness have been based on many assumptions that were not quite realistic and, therefore, suffer from many limitations. This study accounts for the following factors: nonlinear soil behavior, earthquake induced strains (i.e., free-field strains), the presence of active and passive conditions and the corresponding differences in soil behavior, the influence of wing walls, and the physical abutment dimensions.

Expected Products or Deliverables:

This study will provide a relatively simple approach to estimate the nonlinear longitudinal, vertical, and transverse spring stiffnesses of seat-type abutments on spread footings. Readily usable design curves will be developed as a function of the abutment dimensions, design excitation level at the site, and foundation and backfill soil properties. Routinely used soil properties are adequate to use the procedure proposed in the study.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Raj V. Siddharthan received his Ph.D. from the University of British Columbia, Canada. He joined the University of Nevada, Reno in 1984 where he is currently Professor of Civil Engineering. Dr. Siddharthan developed a two-dimensional effective stress based computational model (TARA) to study the seismic response evaluation of soil structures. He has authored over 80 publications in journals, national and international conferences, and technical reports. His research contributions have been in many disciplines including geotechnical-earthquake engineering, offshore engineering and transportation engineering. His current research interests include the seismic behavior of soils and structures (liquefaction, site-specific soil response, permanent deformation behavior of flexible and rigid retaining walls and bridge abutments), wave-soil interaction, and pavement material characterization and pavement response analysis.

Mahmoud E. El-Gamal is currently a Ph.D. candidate in civil engineering at the University of Nevada, Reno. He developed an abutment model to evaluate the abutment translational stiffnesses (longitudinal, vertical, and transverse) for seat-type abutments on spread footings and on pile foundations. He has six publications in journals and in national and international conferences. His research contributions and interests have been in soil response under static and dynamic loading, the dynamic behavior of rigid and flexible retaining walls, interpretation of centrifuge model behavior, bridge abutment optimum design and seismic behavior, abutment stiffness and analysis, and bridge abutment movement.

Principal Investigator's Telephone Number: (702) 784-1411
Facsimile Number: (702) 784-4466
E-mail Address: siddhart@unr.edu

CHARACTERIZATION OF NONLINEAR ABUTMENT STIFFNESSES FOR SEISMIC DESIGN AND RETROFIT

Raj V. Siddharthan[1] and Mahmoud El-Gamal[1]

[1] Department of Civil Engineering
University of Nevada, Reno, NV 89557

Research Objectives

Seismic response analyses of highway bridges are often necessary in the seismic safety evaluations of existing bridge structures and new bridge designs. In the case of existing bridge structures, dynamic studies can provide valuable insight into the effectiveness of various seismic retrofit techniques. Such studies require realistic characterization of bridge deck-abutment interaction. The current procedure (e.g., AASHTO, 1983 and CalTrans) models the deck-abutment interaction using translational springs (longitudinal lateral and vertical) located at the deck-abutment support. Recent large-scale field studies conducted at the University of California at Davis clearly indicate that these translational springs are nonlinear even at low displacements (as low as 1 mm). Furthermore, given that soil will also behave nonlinearly above acceleration levels of more than about 0.1g, abutment stiffness is clearly nonlinear and should be treated as such.

Past studies that dealt with abutment stiffness have been based on many assumptions that were not quite realistic (Maragakis and Siddharthan, 1989; Wilson and Tan, 1990; Lam et al., 1991) and, therefore, suffer from many limitations. For example, these methods ignore many of the following factors: nonlinear soil behavior, earthquake induced strains (i.e., free-field strains), the presence of active and passive conditions and the corresponding differences in soil behavior, the influence of wing walls, and the physical abutment dimensions. This study is limited to the seat-type of abutments on spread footings and the cohesionless soil type for the foundation and abutment fill.

Research Approach

Figure 1 shows a sketch of a displaced abutment caused by a longitudinal force, P_L, applied to the abutment. To find the secant abutment stiffness, it is required to find the force needed to cause a certain horizontal displacement (δ_L). The movement of the abutment suggests that active and passive conditions develop in front of and at the back of the abutment as shown in Fig. 1. A close look at the figure suggests that, to estimate the force P_L for a given δ_L, one needs the characterization of (1) the wall movement/lateral earth pressure relationship under active and passive conditions, (2) the rotational resistive moment/rotation relationship, and (3) foundation-soil interface forces (horizontal and vertical) at the bottom of the abutment.

By resolving forces in the vertical direction and taking moments about the center of the abutment base, it is possible to find the force, P_L, required to cause a certain displacement, δ_L. The proposed numerical procedure results in abutment rotation, θ, and P_L as solutions after satisfying all of the conditions of equilibrium (force and moment). The secant longitudinal spring is then given by P_L/δ_L (Siddharthan et al., 1995).

The wall movement/lateral earth pressure relationship is nonlinear and differs substantially between the active and passive conditions. Data derived for compacted medium dense sand from field testing and finite element studies have been used to represent the active and passive pressure-wall displacement relationships. A realistic procedure has been developed to represent the nonlinear resistive moment-rotation relationship taking into account the following factors: the strain-dependent nonlinear behavior of the foundation soil (Seed-Idriss relationship), the ultimate bearing capacity, the possible lift-off of the foundation at the base, and the influence of the free-field strains caused by the excitation (Siddharthan et al., 1995). The abutment secant abutment stiffness in the vertical and transverse directions has also been obtained by undertaking a similar approach. The predictive capability of the proposed approach has been validated using large-scale abutment field tests. Both the longitudinal and the transverse field test results have been utilized in the validation.

Expected Products

This study will provide a relatively simple approach to estimate the nonlinear longitudinal, vertical, and transverse spring stiffnesses of seat-type abutments on spread footings. It is expected that the spring stiffnesses can be presented in a design curve and/or equation form that bridge designers can readily use. The curves will be developed as a function of the abutment dimensions, design excitation level at the site, and foundation and backfill soil properties. Routinely used soil properties are adequate to use the procedure proposed in the study, and the approach is consistent with the procedures adopted in the state-of-practice of bridge abutment design under static and seismic loading.

Preliminary Results

Maroney et al. (1994) provided a detailed description of a well-instrumented testing program undertaken for CalTrans to characterize abutment behavior. They tested to failure two large-scale abutments 1.7m and 2.1m high. These abutments were founded on cast-in-drilled-hole (CIDH) piles and were provided with wing walls. Figure 2 shows the longitudinal and transverse secant abutment stiffnesses of the taller (H = 2.1m) abutment as a function of abutment displacement. The test results clearly show that the abutment stiffness is nonlinear and that a substantial reduction in abutment stiffness exists as the abutment displacement increases. The test results reported above are for an abutment on piles. To obtain only the abutment contribution, it is necessary to subtract the contribution form piles. This was achieved by using the guidelines provided by Maroney et al. (1994). Figures 3a and 3b present the field measured secant abutment stiffnesses along with those computed by the proposed approach. The soil properties were selected based on the data provided by Maroney et al. (1994). In the case of the longitudinal stiffness (Fig. 3a), there is a slight over prediction at the low displacement of about 10%, which is well within the variation accepted in geotechnical designs. At other levels of displacement, the predicted longitudinal stiffness is slightly lower. However, overall, the agreement between the predicted and the computed stiffnesses is excellent.

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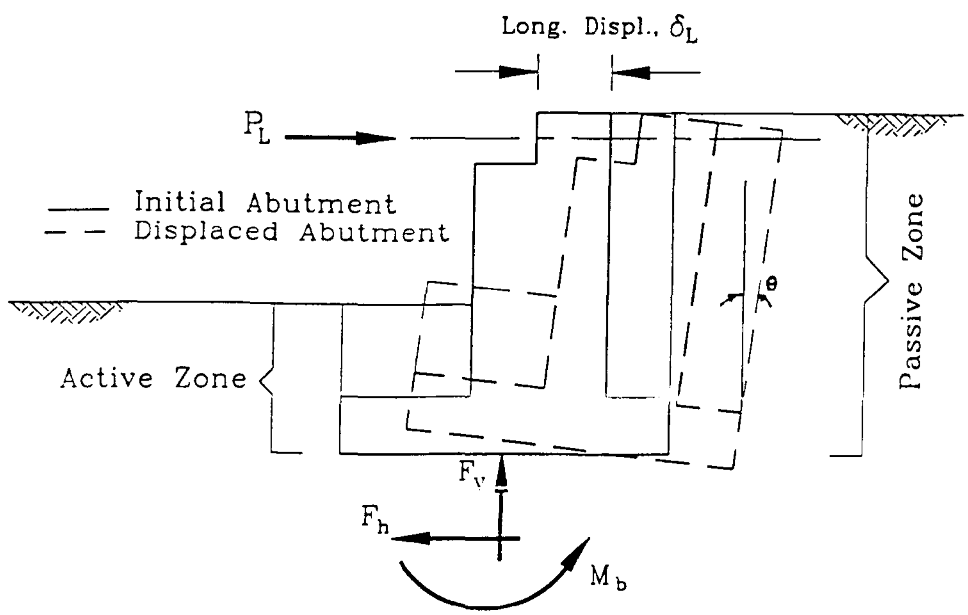


Fig. 1: Sketch of an Abutment Displaced by a Longitudinal Force.

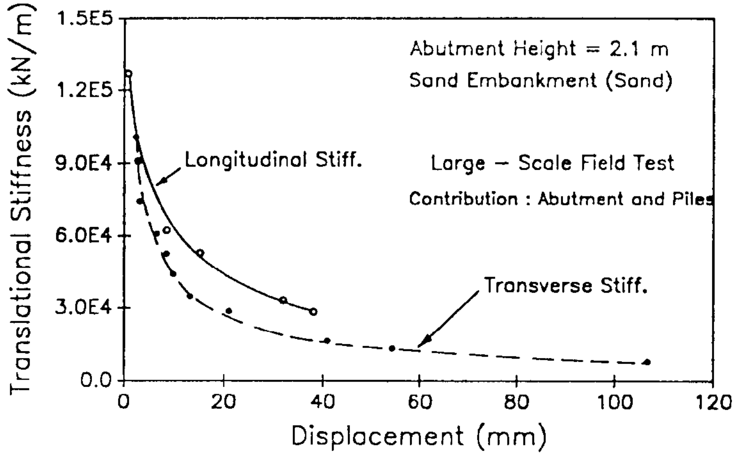


Fig. 2: Field Measured Secant Abutment Stiffnesses: Longitudinal and Transverse (After Maroney et al., 1994).

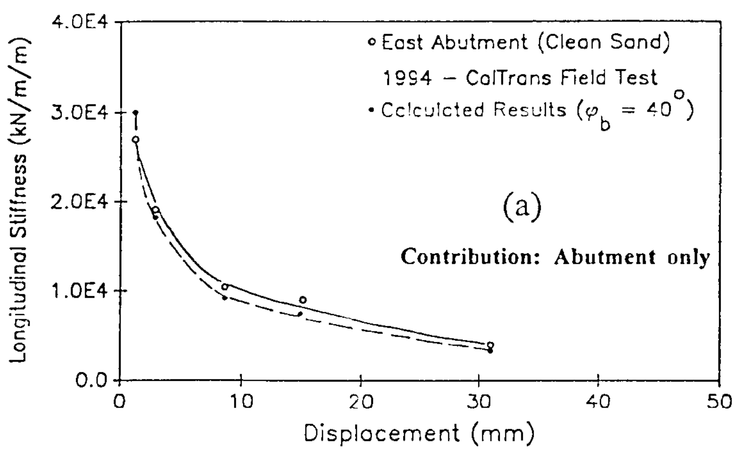


Fig. 3a: Comparison Between Computed and Measured Secant Longitudinal Abutment Stiffness

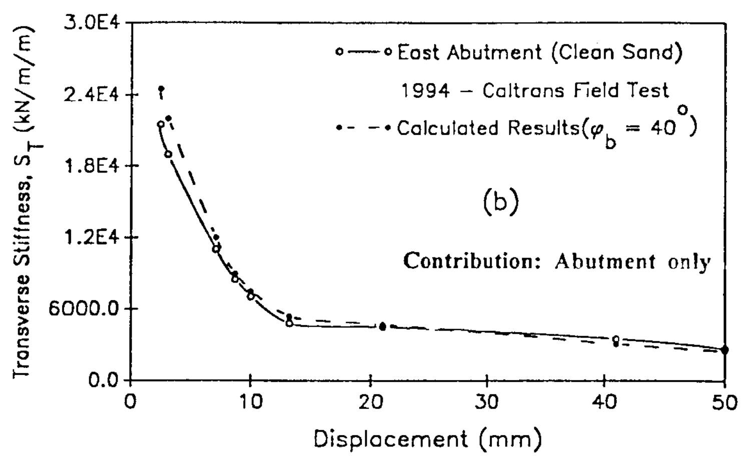


Fig. 3b: Comparison Between Computed and Measured Secant Transverse Abutment Stiffness

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Three-Dimensional Modeling of Stiffness Characteristics for Highway Bridge Abutments

Author(s) and Affiliation(s):

John C. Wilson, Department of Civil Engineering
McMaster University, Hamilton, Ontario, Canada
Michael Justason, Berminghammer Foundation Equipment Ltd.
Hamilton, Ontario, Canada

Principal Investigator: John C. Wilson

Sponsor(s): Natural Sciences and Engineering Research Council, Canada

Research Start Date: September 1993
Expected Completion Date: September 1996

Research Objectives:

The objective of this study is to develop simple techniques for inclusion of abutment stiffness characteristics in seismic bridge analysis.

Expected Products or Deliverables:

Simple relationships describing linear elastic stiffness or typical abutment systems; a basic understanding of the contribution of abutment systems to total bridge response; correlations with seismic data of bridge responses; and a simple yet realistic starting point for including nonlinear effects in abutment analyses.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

John C. Wilson is Associate Professor and Graduate Student Advisor in the Department of Civil Engineering at McMaster University, Hamilton, Ontario, Canada. He is a regiestered Professional Engineer in Ontario and received his Ph.D. from the California Institute of Technology. His research and professional interests are in earthquake engineering and bridge engineering.

Michael Justason is an Engineer at Berminghammer Foundation Equipment Ltd., Hamilton, Ontario, Canada. He has a Master of Engineering degree from McMaster University. His professional interests are in geomechanics and foundation equipment, including development and applications of the statnamic pile load testing device.

Principal Investigator's Telephone Number:	(905) 525-9140 Ext. 24299
Facsimile Number:	(905) 529-9688
E-mail Address:	jcwilson@mcmaster.ca

THREE-DIMENSIONAL MODELING OF STIFFNESS CHARACTERISTICS FOR HIGHWAY BRIDGE ABUTMENTS

John C. Wilson
Dept of Civil Engineering
McMaster University, Hamilton, Ontario, Canada L8S 4L7

Michael Justason
Berminghammer Foundation Equipment Ltd
Hamilton, Ontario, Canada

Research Objectives

Abutments and the soil approach embankments are an integral part of most highway bridges and need to be included, in some manner, for a realistic seismic bridge analysis. Despite the importance of determining the stiffness for bridge abutment systems, there are no simple methods that can be used to evaluate complete 3D stiffnesses at a level of effort consistent with that used to develop dynamic models of the bridge structure.

This research is concerned with the development of new and relatively simple techniques to incorporate the effects of the soil embankments and abutments into seismic analysis of highway bridges. The static and dynamic behavior of soil embankment bridge abutment systems are investigated, and three dimensional finite element models are used to extend earlier two-dimensional models developed by the first author. Of principal concern in this study are the transverse and longitudinal stiffness characteristics of the abutment systems. These are calculated for a wide range of abutment geometries. The results are used to develop simple equations describing static stiffnesses. The results are in a form simple enough to be incorporated as simple boundary conditions in models of the full bridge for purposes of practical seismic analysis of the superstructure and substructures.

Research Approach

<u>Background:</u> An illustration of the type of highway bridge and abutment system considered in this study is shown in Figure 1. It shows both the bridge superstructure and the soil embankments as approaches to the bridge. In this study the term "abutment" refers to the soil embankments and all other structural components that are typically contained within the soil (ie., wingwalls).

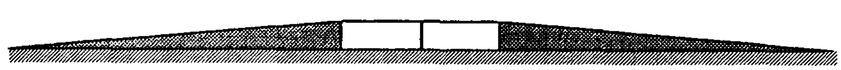


Figure 1 A typical bridge structure and soil abutment system

In this study, the soil embankment is modelled in three dimensions as a homogeneous mass, as shown for a typical configuration in Figure 2. The modelling is completely linear elastic, in order to afford insights into the basic static and dynamic behavior of bridge abutment systems. This is, admittedly, a significant limitation as far as modelling soil behavior under seismic loads is concerned and the possible interactions between the soil and the bridge structures. Nonetheless, this approach has afforded many insights into the behavior of these system that were, heretofore, unavailable. It also provides a firm foundation for future studies investigating the nonlinear response of bridge abutment systems under seismic loads.

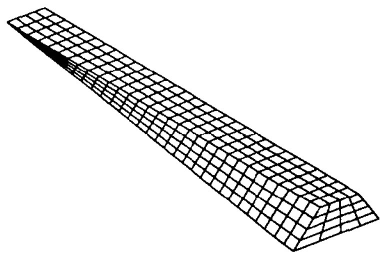


Figure 2 Typical 3-D abutment model

Methods of Analysis: Static analysis are used to determine equivalent static stiffnesses of 3-D abutment systems in both the transverse and longitudinal directions (abutments are typically very stiff in the vertical direction and, because effects of horizontal ground shaking tend to be more significant, the behavior in the vertical direction was not investigated). To determine the stiffnesses, a force (transverse or longitudinal) was applied at the top front edge of the abutment. Figure 3 illustrates some of the extreme geometric configurations examined in this study, where H=abutment height, W=top width, 1/S=side slope. A study of these geometric extremes has enabled rather simple relationships to be developed to relate the 3-D stiffness to stiffnesses developed in earlier studies using simple 2-D plane strain analyses of the abutment cross-sections.

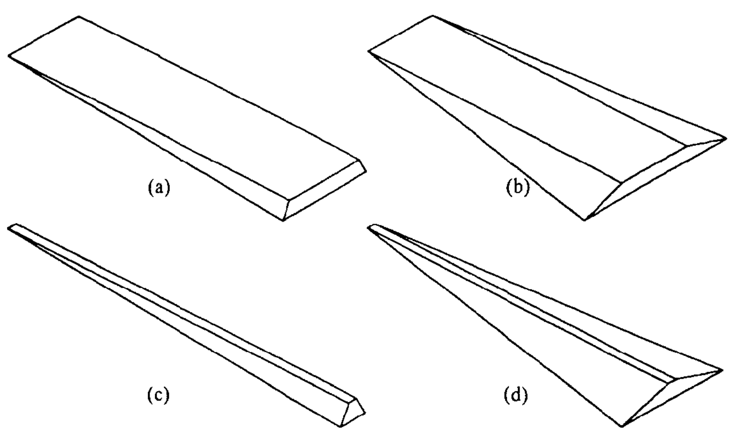


Illustration of extreme values of H/W and S for 3D abutment models for a fixed H, (a) H/W=0.2, S=0.5; (b) H/W=0.2, S=3.0; (c) H/W=2.0, S=0.5; (d) H/W=2.0, S=3.0

Figure 3 Extremes of abutment geometries considered in the study

Expected Products

The expected products from the study include:
1 - simple relationships describing linear elastic stiffness of typical abutment systems
2 - a basic understanding of the contribution of abutment systems to total bridge response
3 - correlations with seismic data of bridge responses that include abutment effects
4 - a simple yet realistic starting point for including nonlinear effects in abutment analyses.

Preliminary Results

Transverse Static Stiffness: In this study, the concept of an 'effective depth' (d_{eff}) was introduced. This provides a measure of the extent of the soil embankment system that contributes to the stiffness at the face of the abutment, and is illustrated in Figure 4.

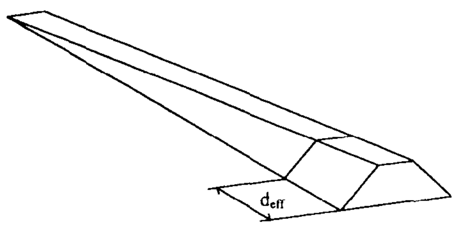


Figure 4 'Effective depth' of abutment for computation of stiffness

The effective depth is a function of the height H of the abutment. On its own is of limited general use. However, by normalizing H with respect to d_{eff}, the relations r_t shown in Figure 5 are produced.

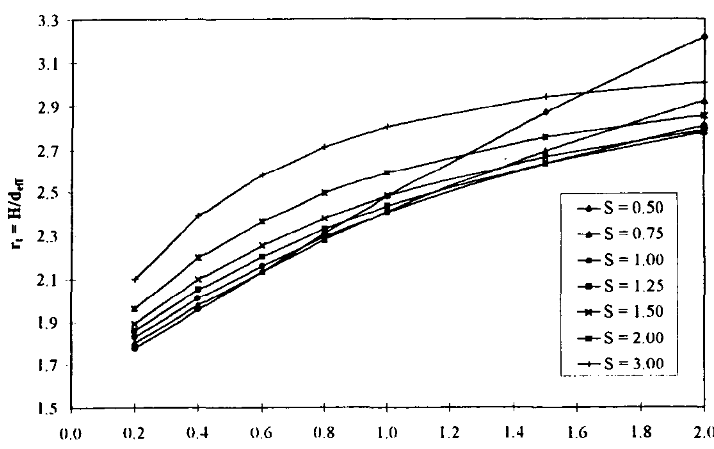


Figure 5 r_t vs H/W characterizing the transverse stiffness for any abutment height H

This is an extremely useful result because the normalized quantity r_t can be used in the equation

$$k_{t3D} = (H/r_t)/(2GS/\ln(1+2SH/W))$$

to obtain an estimate of the transverse stiffness of the three-dimensional finite element model of the abutment soil mass (G is the soil shear modulus). This means that a good estimate of abutment stiffness (within limitations of the modelling process) can be made without resorting to a full model of the abutment.

Longitudinal Static Stiffness: A similar analysis has been developed for longitudinal stiffness. However, in this case the longitudinal stiffness of the three-dimensional model is related to the transverse stiffness through a similar non-dimensional parameter. The resulting stiffness equation is of the same form as the transverse stiffness equation above.

Comparison of Transverse and Longitudinal Stiffnesses: Figures 6a and 6b show transverse and longitudinal stiffnesses for various geometric configurations for an abutment of height 5m. These results show that the transverse and longitudinal stiffnesses exhibit similar characteristics, and tend to be of similar value, for typical abutment geometries. Abutments with larger H are stiffer than abutments with smaller H. Also, for abutments with a fixed H, steeper side slopes (lower S) produce abutments with lower stiffness than abutments with shallow side slopes.

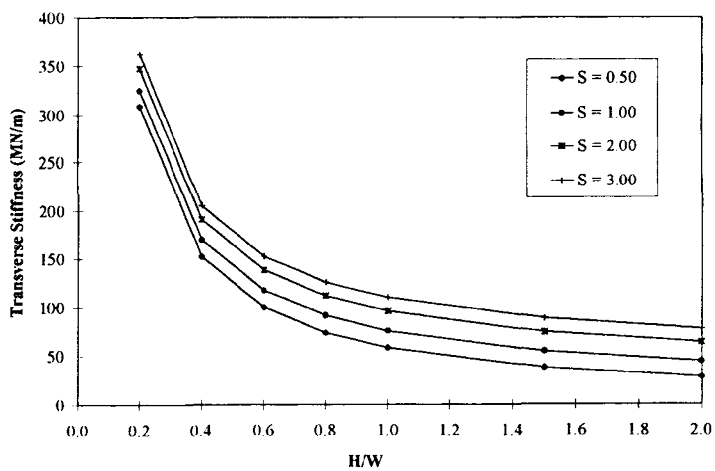


Figure 6a Transverse 3D stiffnesses for a H=5m abutment with various geometries

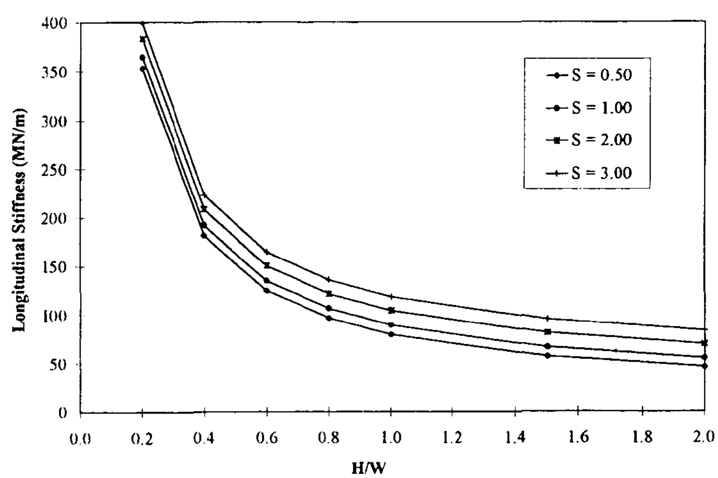


Figure 6b Longitudinal 3D stiffnesses for a H=5m abutment with various geometries

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Behavior of Laterally Loaded Piles Supporting Bridge Abutments

Author(s) and Affiliation(s):

J. Harold Deatheridge, Edwin G. Burdette and David W. Goodpasture
University of Tennessee, Knoxville, TN

Principal Investigator: Edwin G. Burdette

Sponsor(s): Tennessee Department of Transportation and the Federal Highway Administration

Research Start Date: April 1996
Expected Completion Date: July 1997

Research Objectives:

To determine the behavior of the interface between piles and concrete pile caps and to determine the distribution of moments along the length of the pile.

Expected Products or Deliverables:

Comprehensive report describing results of the research.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Edwin G. Burdette is Professor of Civil Engineering at the University of Tennessee, Knoxville. He is a registered Professional Engineer in Tennessee. His primary teaching efforts are related to the behavior, analysis and design of reinforced and prestressed concrete structures. He currently consults in forensic analysis of safety related accidents. He has been actively involved in full scale testing of bridge structures since 1970 and has published extensively in this research area.

J. Harold Deatherage is Professor of Civil Engineering at the University of Tennessee, Knoxville. He is a registered Professional Engineer in Tennessee. His primary teaching efforts are related to the construction means and methods of highway, bridge and building structures. He currently consults in construction related litigation. He has been actively involved in component and full scale testing of bridge structures since 1985 and has published extensively in this research area.

David W. Goodpasture is Professor of Civil Engineering at the University of Tennessee, Knoxville. He is a registered Professional Engineer in Tennessee. His primary teaching efforts are related to the behavior, design, and analysis of steel structures. He currently consults in forensic analysis of structural failures. He has been actively involved in full scale testing of bridge structures since 1970 and has published extensively in this research area.

Principal Investigator's Telephone Number: (423) 974-7704
Facsimile Number: (423) 974-2669
E-mail Address: eburdett@utkvx.utcc.utk.edu

BEHAVIOR OF LATERALLY LOADED PILES SUPPORTING BRIDGE ABUTMENTS

J. Harold Deatherage[1], P.E., Edwin G. Burdette[1], P.E., and David W. Goodpasture[1], P.E.

[1]Department of Civil and Environmental Engineering
University of Tennessee
Knoxville, Tennessee 37996-2010

Principal Investiagtor: Edwin G. Burdette

Sponsors: Tennessee Department of Transportation
Federal Highway Administration

Start Date: April 1996
Completion Date: July 1997

Research Objectives

The Tennessee Department of Transportation (TENN DOT) pioneered the use of jointless bridges in the 1970's and is continuing to be a national leader in this area. In some of these bridges, the abutments are fixed; in other, typically longer, bridges, provisions are made for longitudinal movement of the superstructure at the abutments. In the case of fixed abutments supported on piles, the piles are subjected to lateral deflections due to thermal expansion and contraction of the bridge and possible seismic loading. In order to determine the overall behavior of such a bridge, including the calculation of forces and moments, the bridge designer must be able to calculate the response of the abutment piles.

Two aspects of the piles' behavior are of particular interest: (1) the behavior of the interface between the pile and the concrete pile cap and (2) the distribution of moment along the length of the pile. The behavior of the pile-pile cap interface determines the amount of moment that can be developed at that location. A definitive understanding of the distribution of moments along the pile would allow the bridge designer to develop a realistic mathematical model of the abutment. Both of these aspects of behavior, particularly the distribution of moment along the pile, are dependent to some degree on the number of cycles of load to which the pile has been subjected, particularly when these load cycles have produced large lateral displacements of the top of the pile.

Two types of piles are of particular interest: steel H-piles and prestressed concrete piles. Initially, the proposed research will focus on the behavior of steel H-piles as these present fewer experimental and analytical problems than the prestressed piles. If the work on steel H-piles is successful, tests on prestressed concrete piles will be performed at a later date.

The objective of the research is to achieve a better understanding of the behavior of abutment piles both in the embedment zone of the piles in the abutment and in the soil which supports the piles. From this enhanced understanding a bridge designer can predict more accurately the forces and moments acting on the piles.

Research Approach

In order to achieve the stated objective, lateral load tests will be performed on piles in which the piles are subjected to several incremental cycles of loading. In the final cycles the lateral deflection of the pile cap will be as much as two inches.

The piles tested will be HP 10x42 steel piles. Each pile will be instrumented along its length with strain gages (and the gages protected) before driving. A gaged pile will then be driven 25-ft. to 30-ft. into the soil. A pile cap (stiffness approximately ten times the stiffness of the pile) will then be cast. The pile will project approximately 1-ft. into the cap. The ends of the cap in the direction of pull will be supported on rollers against bearing pads on the ground. Strain gages will be used to measure the strains in the pile just below the cap, and load cells will measure the vertical reactions at the ends of the pile cap in order to calculate the moment at the pile. LVDT's will be used to measure deflections of the pile cap and pile (above ground). Current plans are to test two piles, one in September - October of 1996 and a second in February - March of 1997 after data from the first test have been thoroughly studied.

In each test the pile cap will be subjected to a lateral load required to produce a predetermined deflection. All gages, LVDT's, and load cells will be continuously monitored during loading. Then the load will be removed. This sequence will be repeated two more times, after which the same procedure will be done for larger deflections.

From the data obtained, information in the two areas noted earlier will be obtained. The behavior of the pile-pile cap interface will be studied, and the deflected shape of the pile below ground will be determined. Both of these areas are of particular interest when piles are subjected to a large number of cyclic loads. Additionally, analytical studies will be performed using avaliable software to compare the measured and computed responses of the pile to the applied loading.

Expected Products

A comprehensive report describing the results of the testing and the implications of the results will be written. It is expected that the stress distribution along the vertical axis of the pile will be obtained. This will enable the researchers to better understand the interaction between the soil and driven piles. Additionally, the embedment zone of the pile/abutment interface will be studied in detail. It is of particular interest to determine the minimum embedment of the pile into the abutment which will support the full plastic moment of the pile.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: The Use of Reticulated Micropile Groups for Bridge Foundations

Author(s) and Affiliation(s):

Fred H. Kulhawy, School of Civil and Environmental Engineering
Cornell University, Ithaca, NY
James A. Mason, California Department of Transportation, Sacramento, CA

Principal Investigator: Fred H. Kulhawy

Sponsor(s): Federal Highway Administration

Research Start Date: June 1996 (estimated)
Expected Completion Date: June 2000

Research Objectives:

The objectives are to: investigate the effectiveness of reticulated micropile groups (RMG) in dry sands; investigate the basic mechanics of the components of a RMG; investigate the influence of contractiveness and/or pilativeness of the soils upon the performance of the RMG; compare the performance of the RMG to traditionally installed vertical deep foundation elements; and, develop predictive analytical modeling tools.

Expected Products or Deliverables:

Confirmation of the global surface response of RMG, under axial and lateral loading; confirmation of the engagement of the confined soil mass; determination of optimal trends for spacing (density), batter, and inclination of RMG; and equations, computer code and general design methodology for the design of RMG written and presented for the general foundation engineer.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Fred H. Kulhawy is Professor in the School of Civil and Environmental Engineering and Graduate Faculty of Geological Sciences at Cornell University. He received his BSCS and MSCE from the New Jersey Institute of Technology in 1966 and his Ph.D. from the University of California, Berkeley. Professor Kulhawy has been on the teaching and research staff at New Jersey Institute of Technology, University of California, Syracuse University, and Cornell University, and has been a Visiting Professor at the Universities of Cambridge (U.K.), Sydney (Australia), Hawaii, Hong Kong, and Queensland (Australia). He has been at Cornell since 1976 and has led the Geotechnical Engineering Group since 1977. His teaching has included many areas in geotechnical engineering, with current emphasis on foundation engineering, rock engineering, embankment dams, underground structures, and geotechnical design practice. His research efforts have focused on foundations, dams, coastal and marine structures, soil-structure interaction, underground openings, soil and rock behavior, geological modeling, and computer applications in geotechnical engineering. He has been a primary investigator on research contracts worth over $7.4 million (U.S.), and he is the author, or co-author of over 210 published technical papers and reports. He has lectured widely, giving over 480 presentations in 58 cities within the U.S. and in 40 other cities around the world.

James A. Mason is a Bridge Engineer with the California Department of Transportation. He received his BS in Physics in 1986 from the California State University, Hayward and his MSCE in 1993 from California State University, Sacramento. Mr. Mason has been working with Caltrans since 1990. He is currently a design team member for the seismic retrofit and upgrade of the San Francisco/Oakland Bay bridge. His scope of work has included all aspects of the structure, from evaluation of the original timber piled foundation and towers, to the superstructure with isolation device retrofits. His retrofit experience includes numerous multi-span viaducts throughout California, each with unique geotechnical problems, from the soft clays of the San Francisco Bay area, to liquefiable sand along the northern California coast. All of the dynamic analyses for these structures have included non-linear foundation springs such that a complete seismic system was modeled. In 1992, Mr. Mason designed and managed a testing program of full-scale deep foundation elements that were founded in soft San Francisco clays. This test program included various element types and included several installation methods.

Principal Investigator's Telephone Number: (916) 227-8103
Facsimile Number: (916) 227-8379
E-mail Address:

THE USE OF RETICULATED MICROPILE GROUPS FOR BRIDGE FOUNDATIONS

Fred H. Kulhawy, P.E., G.E.[1] and James A. Mason, P.E.[2]

[1]Cornell University
School of Civil and Environmental Engineering, Ithaca, N.Y. 14583-3501
[2]California Department of Transportation
Engineering Service Center, Sacramento, CA. 95816

Research Objectives

The use of micropiles, i.e., small diameter deep foundation elements, as primary foundation components for bridges in the United States is becoming more common. Their usage is typically as a load transfer strut from the pilecap to competent subsurface strata. Yet, the greatest benefit to be realized from micropiles is from the unique installation method known as reticulation. A reticulated micropile group is constructed by installing two closely spaced concentric pile groups around the perimeter of the foundation pilecap. The inner row is installed battered out and inclined, roughly at a 20° angle for batter and inclination. The outer row is installed just a few feet farther out, battered at the same angle, and inclined in the opposite direction. It is the reticulation, the inter-weaving, that allows the soil to arch from pile to pile, in effect forming a soil membrane, a "Quilted Soil Surface". Both the encased soil mass as well as the loaded soil wedge will provide resistance to loading. Reticulated micropile groups have been successfully used under seismic loading in Europe. Yet, a rational design methodology has not been realized. This research fulfills that objective.

Research Approach

The objective of the experimental program is to investigate the effectiveness of reticulated micropile groups. One quarter (1/4) scale models, with micropile diameters of 40 mm, will be constructed for the testing of: (a) the basic components of reticulated groups and then (b) the complete reticulated system. The depth to diameter ratio (D/B) of the test micropiles will be large enough to have complete flexural response, as opposed to rigid body rotation. By using the quarter scale models, problems that could arise from to the scaling factors of smaller diameter testing elements would be eliminated.

The baseline performance of the individual elements, as well as the performance of the integrated reticulated micropile groups, will be compared with traditional deep foundation components and systems under equivalent loading cases. The first series of tests will be replications of Lizzi's original tests. Those tests compared vertically installed micropiles with the reticulated groups for two soil types: cohesive and non-cohesive. It has been the original Lizzi tests that have generated such intense interest in reticulated groups, and their inclusion in this study can not be overemphasized.

The other major question to be addressed is whether or not the reticulated group works as well in contractive soils as in dilative soils. The method of addressing this issue is by testing three density states of drained sands: loose, medium, and dense. By observing the trends of the responses to loading of the various elements, the question about applicability of reticulated micropile groups in different soil conditions can be addressed.

Analytical modeling and design guidelines are an integral component of this project. The modeling will be conducted concurrently with the experimental testing. Feedback and refinement between the two teams, the experimental team and the analytical modeling team, should be occurring daily. The computer program that will be utilized for the modeling is still to be determined.

Proposed Load Tests: The scope of this proposed study will range from the basic components of the reticulated group to the complete system, i.e., from individual micropiles to the reticulated micro-pile group. All tests will be conducted in drained sands. There will be three density states of the sands investigated for the majority of the tests: loose, medium, and dense. In addition to investigating the performance of the various components in the three density states, the influence of element spacing will be tested for the interaction of the basic multiple element components: (a) vertical installation - two piles - in-line interaction - vertical & skewed installation, (b) reticulated installation - two piles - node experiment - in a cross-over configuration - vertical & inclined orientation, (c) reticulated installation - four piles - soil diamond experiment - in a cross-over configuration - vertical & inclined orientation. By investigating the spacing phenomena for these tests, optimum spacing conditions will be defined for the remainder of the tests.

Equipment Design & Soil Property Characterization, Phase 1: The Phase 1 work is subdivided into two parts: (a) test equipment design, and (b) soil property characterization. The test equipment will be able to facilitate one quarter scale models of a reticulated micropile group.

The second part of Phase 1 will be determining the static in-situ sand properties for the three density states. This characterization will be accomplished by: (a) various index and shear tests of the test box material, and (b) mini-probe testing (CPT). Also the structural properties of a few micropiles will be determined for inclusion with the analytical analyses.

The Original Lizzi Tests, Phase 2: Phase 2 will be repeat tests of the original tests performed by Lizzi, as mentioned previously. These will be replicas of the tests that have generated the intense interest by foundation engineers worldwide. The tests were of : (a) a three pile vertical group, (b) an eighteen pile vertical group, and (c) an eighteen pile reticulated group.

Performance Tests of Individual Micropiles, Phase 3: The Phase 3 tests are to establish the performance of the individual micropiles in vertical installation. These tests will have the same per pile loading cases as for the reticulated systems. The basic mechanics of the individual micropile must first be investigated. The load shed characteristics for axial loading, compression and tension, must be rigorously understood before any attempt to pursue the complexities of reticulation. These tests will also help to calibrate the analytical model for the system interaction.

Performance Tests of the Reticulated Micropile Group Components. Phase 4: The Phase 4 tests are of the multiple element systems, both vertically installed and reticulated systems. A description of the components follows.

The Node: The node is the most basic form of pile-soil-pile interaction. In the cross-over configuration two adjacent micropiles have just one node point of direct interaction, i.e., pile-soil-pile interaction. It must be remembered that two adjacent micropiles have opposite batter angles. Therefore, there would be a minimum of interaction at the nodal cross-over point for longitudinal loading.

The Soil Diamond: The next larger-scale basic component of the reticulated micropile system to be investigated is the "Soil Diamond". The soil diamond is developed from two sets of two parallel inclined micropile groups, with opposite inclination. Just as for the two pile nodal cross-over soil arching mechanism, the soil diamond develops arching from the lead to trailing row.

The Quilt: The "Quilt" is the next larger-scale basic component and is a system of soil diamonds. The expansion of the diamond pattern is continued in both width and depth. It is easy to visualize the "quilt" as a "soil membrane" that is developed by the arching action between the reticulated micropiles, see Figure 1.

The Reticulated Micropile System: The testing of the reticulated micropile system is the overall goal of the project. The reticulated micropile system is a closed-form geometry of the "Quilted Wall" system. The membrane analogy is again appropriate for this structure. This is, in effect, a truncated cone or a surface of revolution. The loading conditions would be combined axial and lateral. Also at this stage, it should be clear how dilatant or contractive soil behavior affects the performance of the system. At this point in the research program, the analytical modeling should be in a form of near-completion. The design methods should be close to finished for the researchers and design aids should be clearly in-site.

Expected End Products

This research will combine a robust regime of load tests that will include the basic components of the reticulated group to the complete system, i.e., from the individual micropile to the reticulated micro-pile group. The testing of this system will quantify the mechanics of interaction of the reticulated micropile system and will bring it to a new level of understanding. Perhaps more importantly, this research would bring a technology to the American market that could be readily used. American companies and the state and federal departments of transportation could utilize the completed research in many of their current design / build projects. From the retrofit and strengthening of historic buildings on the east and west coasts to the seismic retrofit of bridges throughout the nation, the utilization of reticulated micropiles will solve difficult installation problems and provide substantial load capacity for various soil and foundation conditions. Ultimately, the combination of load testing with design software will provide designers with the tools to confidently utilize this unique technology. Design guidelines will be a deliverable product with the two other components of the final report, i.e., the experimental and modeling results.

Preliminary Results

None.

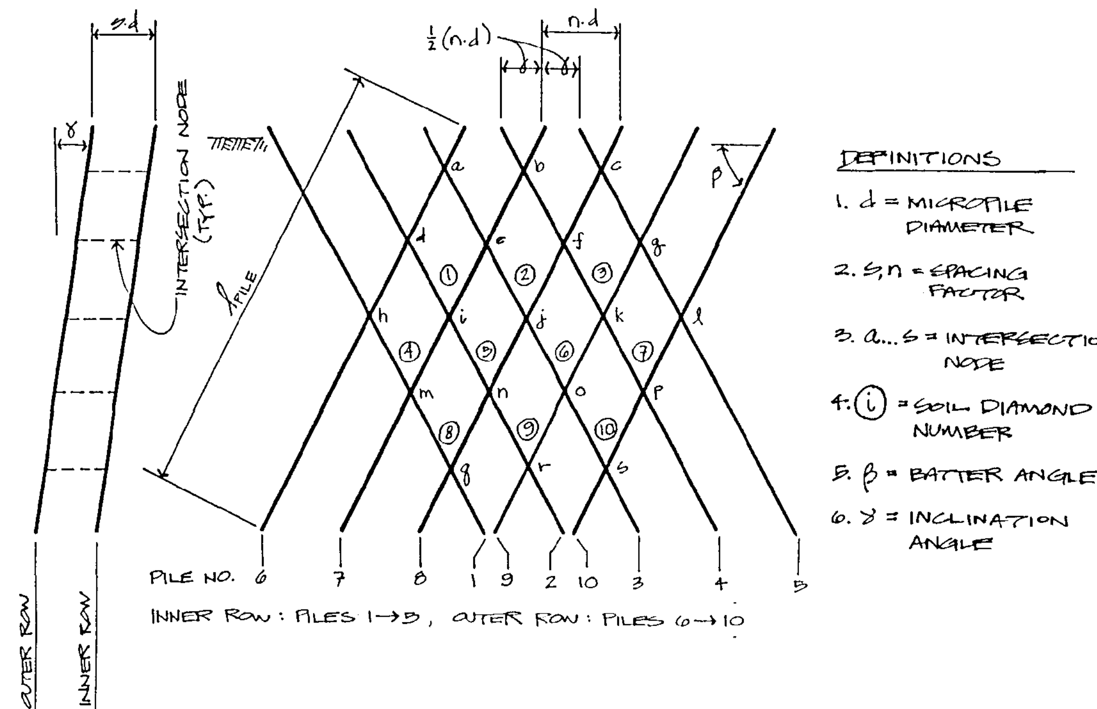


Figure 1. 10-Pile Reticulated Test Wall: "Quilted System"

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS RESEARCH ABSTRACT

Title: Repair of Damaged Pier Walls

Author(s) and Affiliation(s):

Medhat A. Haroun, Gerard C. Pardoen,
Hussain Bhatia, Santosh Shahi and Robert P. Kazanjy
Department of Civil and Environmental Engineering
University of California, Irvine, Irvine, CA

Principal Investigator: Medhat A. Haroun and Gerard C. Pardoen

Sponsor(s): California Department of Transportation

Research Start Date: July 1, 1995
Expected Completion Date: October 31, 1996

Research Objectives:

Large accelerations in a severe earthquake may force a pier wall to experience force levels beyond the elastic range, causing substantial localized damage. Instead of demolishing the damaged pier wall and building a new one, it may be beneficial to repair the damaged wall resulting in substantial savings in material and labor along with a quick restoration time to its intended use. The main objective of this research is to repair damaged pier walls from a previous Caltrans testing program and to compare the strength, ductility and cross-tie performance of the repaired walls with the undamaged pier walls. Six damaged pier wall samples will be used to test the effectiveness of proposed repair techniques. Each repaired sample will be cyclically loaded as in earlier investigations.

Expected Products or Deliverables:

Design recommendations for the repair of earthquake-damaged pier walls.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Medhat A. Haroun is Professor in the Department of Civil and Environmental Engineering at the University of California, Irvine. He received his Ph.D. from the California Institute of Technology. Dr. Haroun is currently serving as the Egypt Study Center Director of the University of California Education Abroad Program.

Gerard C. Pardoen is Professor in the Department of Civil and Environmental Engineering at the University of California, Irvine. He received his Ph.D. from Stanford University.

Hussain Bhatia is a graduate research assistant and Ph.D. candidate in the Department of Civil and Environmental Engineering at the University of California, Irvine.

Santosh Shahi is a graduate research assistant and Ph.D. candidate in the Department of Civil and Environmental Engineering at the University of California, Irvine.

Robert P. Kazanjy is Laboratory Director of the Structural Engineering Test Hall at the University of California, Irvine. He received his BS degree from the University of California, Irvine.

Principal Investigator's Telephone Number:	(714) 824-7094
Facsimile Number:	(714) 824-2117
E-mail Address:	gpardoen@uci.edu

REPAIR OF DAMAGED PIER WALLS

Medhat Haroun[1], Gerard Pardoen[1], Hussain Bhatia[2], Santosh Shahi[2] and Robert Kazanjy[3]

[1]Professor
[2]Graduate Student Researcher
[3]Senior Development Engineer
Department of Civil and Environmental Engineering
University of California, Irvine, CA 92717

Research Objectives

Large accelerations in a severe earthquake may force a pier wall to experience force levels beyond the elastic range, causing substantial localized damage. Instead of demolishing the damaged pier wall and building a new one, in some cases it may be beneficial to repair the damaged wall resulting in substantial savings in material and labor along with a quick restoration time to its intended use. The main objective of this research is to repair damaged pier walls from a previous testing program and to compare the strength, ductility and cross-tie performance of the repaired pier walls with the undamaged pier walls.

Research Approach

Background: Six half-scale pier walls with a height of 127 inches, a width of 96 inches and a thickness of 10 inches had been built to present Caltrans specifications [1]. The dimensions of the footings used to support the pier walls were 18 inch thick, 116 inch long and 56 inch wide, and were fixed to the strong floor with pre-tensioned rods. The six specimens were divided into two groups. One group had a low vertical reinforcement ratio of 1.3 percent (L, which is typical in Caltrans specifications) while the other group had a high vertical reinforcement ratio of 2.3 percent (H, an upper limit). For each group, three different cross-tie distributions were used. The pier walls were designated as N: no cross-ties; U: uniform cross-ties; and P: partial cross-ties (in the calculated plastic hinge zone). The walls were tested under displacement controlled cyclic lateral loading about the weak axis and had varying degrees of damage at the end of the tests. For instance, the concrete cover had fallen off exposing the buckled vertical reinforcements and most of the 135° legs of the cross-ties, in walls with cross-ties, had opened. In the HP pier wall, a vertical rebar had also broken at the base of the wall. The reinforcement details of the original walls are shown in Figure 1.

Repair Scheme: The underlying idea behind the repair scheme was to provide more confinement in the damaged plastic hinge zone. All pier walls, except HN, were repaired as shown in Figure 2. A 25 inch height of concrete was removed from the pier wall, although the plastic hinge length was calculated to be 21.25 inches. Six layers of horizontal reinforcement were provided at an interval of 4.5 inches using # 3 Grade 60 steel. Additionally, two Grade 90 D5-wire with alternating 90° and 135° legs were placed at each location. The lengths of the 135° legs were also increased to provide better anchorage in the concrete core of the pier wall. The concrete was recast in the lower portion of the pier wall. The cross section dimension was increased to provide a 1 inch clear concrete cover over the buckled vertical reinforcements. Thus, the thickness of the repaired wall in the plastic hinge zone increased to 12 inches. Pier wall HN will be repaired by providing a steel-plate jacket around the damaged section with bolts going through the wall.

Testing Scheme: Four of the six repaired pier walls have been tested to date using the same cyclic lateral displacement test protocol as the original pier walls. An axial load of 5 percent of the axial load carrying capacity of the section was applied as in the original pier wall test.

Preliminary Results

The envelopes of the hysteresis loops of the original [2] and the repaired pier walls are shown in Figure 3. The displacement ductility factors and the strengths of the original and the repaired walls are compared in Table 1.

Ductility: The L walls reached about the same displacement ductility factors as the original walls. The repaired HU pier wall had a displacement ductility factor that was slightly lower than the original HU wall but the displacement was about the same. The repaired pier wall HP had a welded vertical rebar that broke at low lateral displacement, causing surrounding vertical rebars to break at the base of the pier wall. The test was stopped after 4 vertical reinforcing bars broke.

Strength: All four samples achieved about the same or higher strengths than the original pier walls. At low lateral displacements, the increased cross section of a repaired pier wall experiences higher lateral loads. However, once the cover falls off, the behavior is essentially that of the concrete core and therefore, the lateral load falls to the same levels as the original pier wall.

Cross-ties: None of the 135° legs of the cross-ties used in the L walls opened while some of the 135° legs opened in the H walls towards the end of the test.

Pier Wall	Original Pier Wall			Repaired Pier Wall		
	Max Lateral Load	Lateral Disp.	Disp. Ductility	Max Lateral Load	Lateral Disp.	Disp. Ductility
LU	41.9 kips	10.2 in.	5.69	49.0 kips	10.2 in.	5.25
LP	41.3 kips	11.0 in.	6.14	51.5 kips	10.2 in.	5.44
HU	63.8 kips	10.5 in.	4.07	67.2 kips	10.0 in.	3.29
HP	62.5 kips	12.1 in.	4.51	79.3 kips	6.9 in.	2.69

Table 1: Comparison of Original and Repaired Pier Walls

Conclusions

The repaired L walls behaved more like the original L walls (ductility and strength) than the repaired H walls when compared to the original H walls. Based on the tests to date, the repair of damaged pier walls is a cost effective alternative to new construction.

References

[1] Department of Transportation, State of California, "Bridge Design Specifications," June 1990.

[2] Haroun, M.A., Pardoen, G.C., Shepherd, R., Haggag, H.A., and Kazanjy, R.P., "Cyclic Behavior of Bridge Pier Walls for Retrofit," Final Report to the California Department of Transportation, December 1993.

[3] Haroun, M.A., Pardoen, G.C., Shepherd, R., Haggag, H.A., and Kazanjy, R.P., "Assessment of Cross-Tie Performance in Bridge Pier Walls," Final Report to the California Department of Transportation, December 1994.

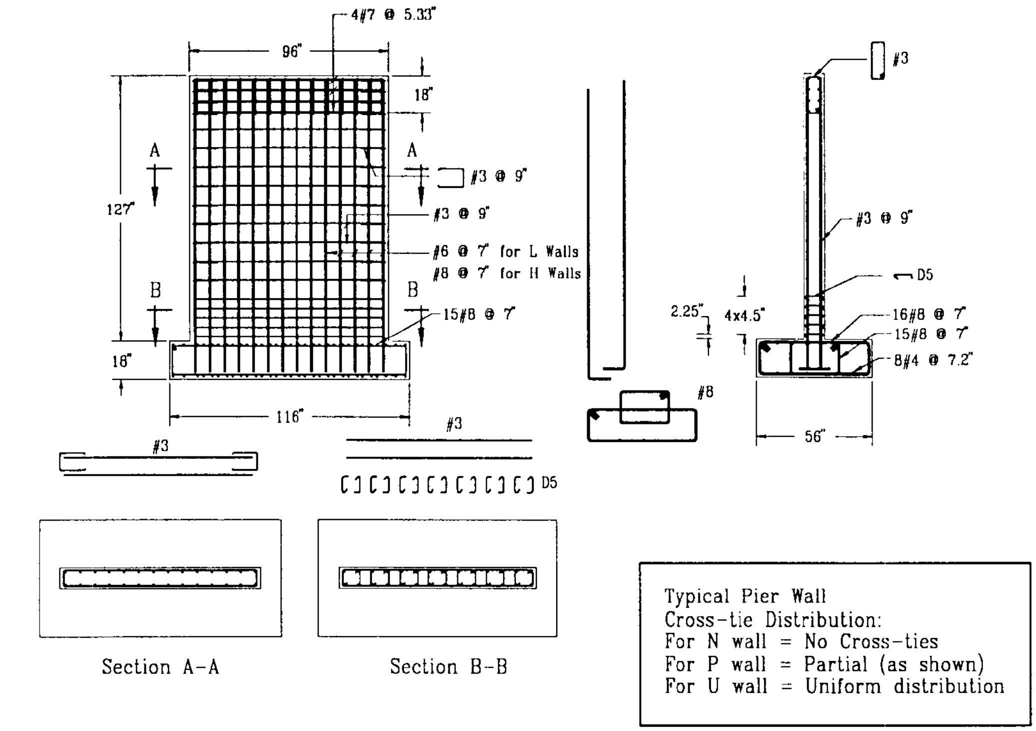


Fig. 1: Reinforcement Details of Original Pier Walls

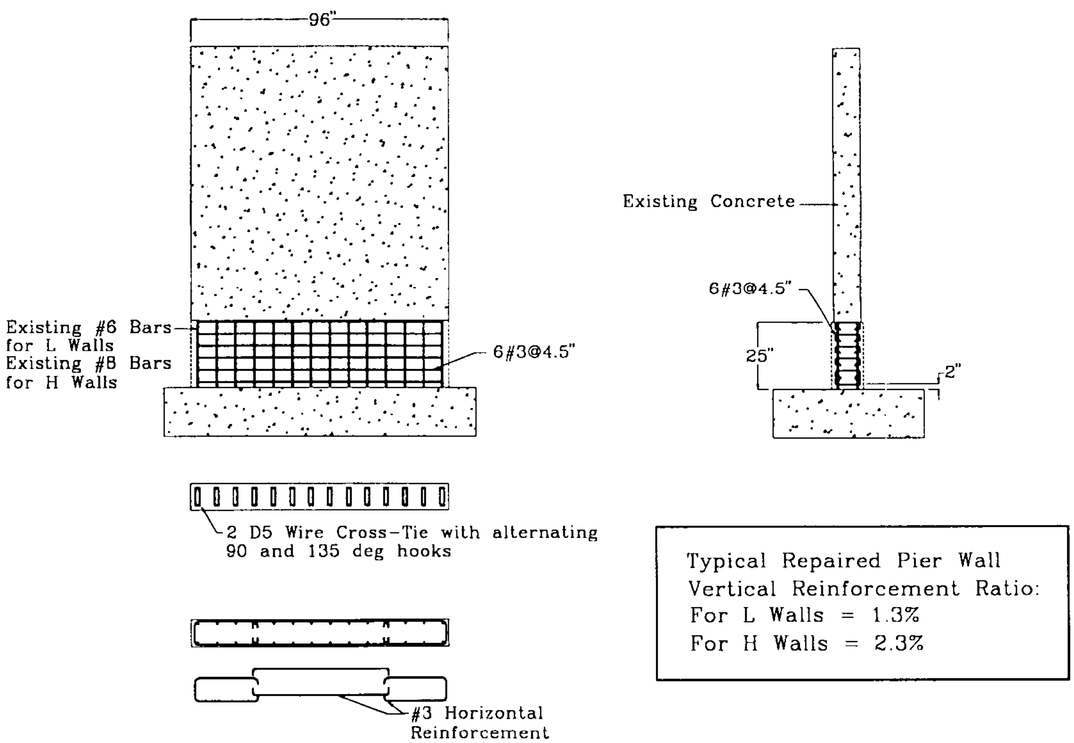


Fig. 2: Reinforcement Details of Repaired Pier Walls

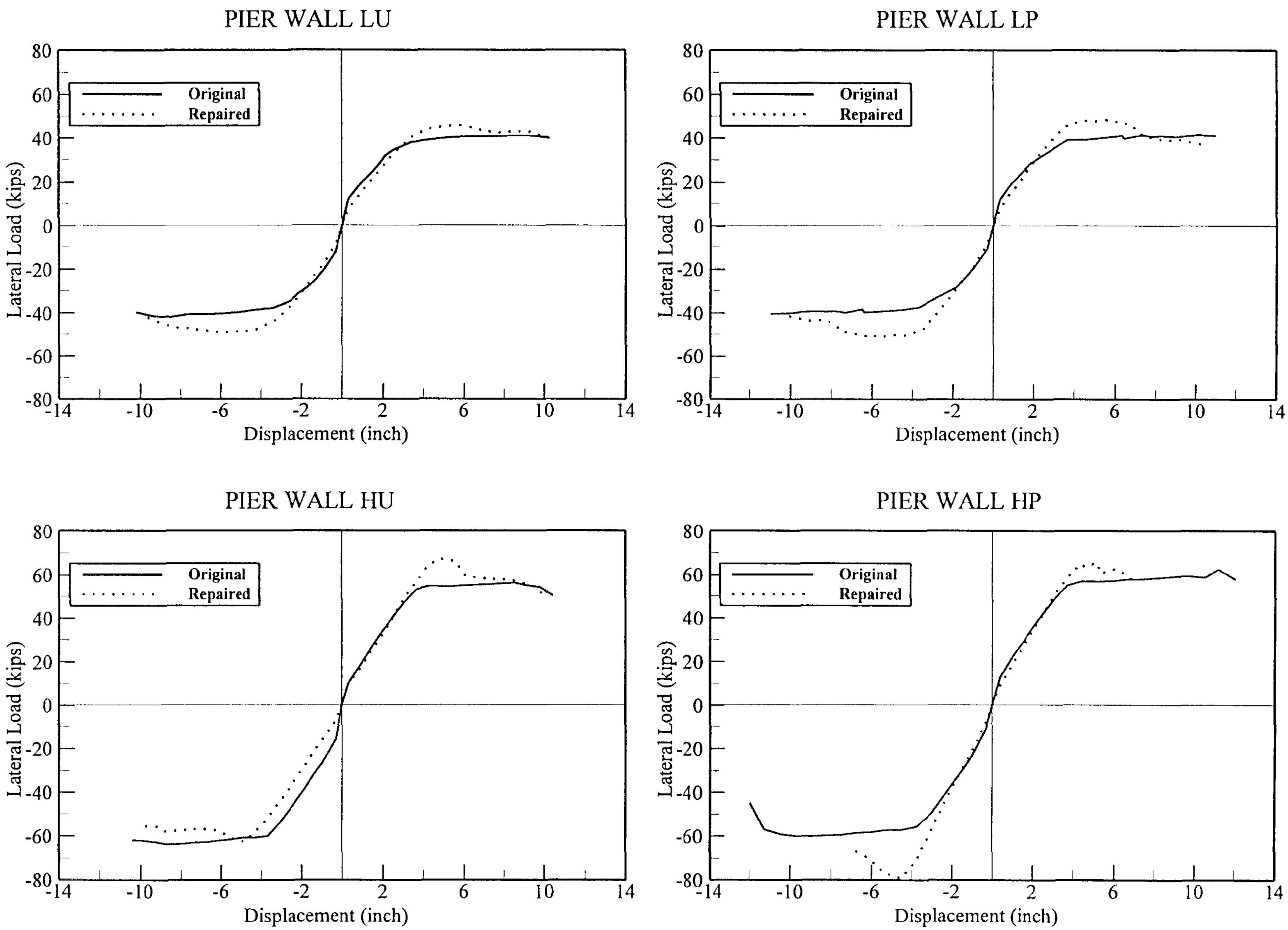


Fig. 3: Envelope of the Hysteresis Loops of the Original and Repaired Pier Walls

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS RESEARCH ABSTRACT

Title: Modeling Seismic Damage of Circular Reinforced Concrete Bridge Columns

Author(s) and Affiliation(s):

Andrew Taylor and William Stone, National Institute of Standards and Technology
Gaithersburg, MD
Ashraf El-Bahy and Sashi Kunnath, Department of Civil Engineering
University of Central Florida, Orlando, FL

Principal Investigator: Andrew Taylor

Sponsor(s): Federal Highway Administration, National Center for Earthquake Engineering Research, and California Department of Transportation

Research Start Date: October 1994
Expected Completion Date: September 1996

Research Objectives:

The long term objective of this project is to develop performance-based approaches to the seismic design, retrofit, and repair of reinforced concrete bridge columns. The objectives of the current task, Phase II, are to use the experimental data obtained in Phase I to evaluate and calibrate existing analytical damage models for reinforced concrete columns, derive improved damage models, and develop methods of using these damage models in practical design applications.

Expected Products or Deliverables:

The primary product of this study will be an analytical damage model for circular, flexure-dominated, reinforced concrete bridge columns which reflects damage caused by controlled laboratory loading and damage caused by random earthquake loading. Based on the findings of the damage model, a performance-based approach for design and evaluation of well-confined circular, flexure-dominated bridge columns will be proposed. It is anticipated that this model will be based on a comparison of damage capacity and damage demand.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Andrew Taylor is a Research Structural Engineer in the Earthquake Engineering Group of the Building and Fire Research Laboratory at the National Institute of Standards and Technology. He has been at NIST since 1990. Dr. Taylor received his BS degree in 1983 and his MSCE in 1985 from the University of Washington, and his Ph.D. in civil engineering from the University of Texas at Austin in 1990.

Ashraf El-Bahy is a Ph.D. candidate in the Department of Civil and Environmental Engineering at the University of Central Florida. Mr. El-Bahy received his BS degree in civil engineering at Cairo University in 1985, and MS degree in civil engineering from Cairo University in 1988.

Sashi Kunnath is an Assistant Professor in the Department of Civil and Environmental Engineering at the University of Central Florida. He has been at UCF since 1991. Dr. Kunnath received his BSCE degree from Bangalore University, India, his MS degree in structural engineering from the Asian Institute of Technology, Bangkok, and his Ph.D. from the State University of New York at Buffalo.

William Stone is a Research Structural Engineer in the Earthquake Engineering Group of the Building and Fire Research Laboratory at the National Institute of Standards and Technology. He has been at NIST since 1980. Dr. Stone received his BS degree in 1974 and his Master of Engineering degree in 1975 at Rensselaer Polytechnic Institute, and his Ph.D. in civil engineering from the University of Texas at Austin in 1980.

Principal Investigator's Telephone Number:	(301) 975-6078
Facsimile Number:	(301) 869-6275
E-mail Address:	ataylor@nist.gov

MODELING SEISMIC DAMAGE OF CIRCULAR REINFORCED CONCRETE BRIDGE COLUMNS

Andrew Taylor[1], Ashraf El-Bahy[2], Sashi Kunnath[3], and William Stone[4]

[1,4]Research Structural Engineer, Structures Division, Building and Fire Research Laboratory
National Institute of Standards and Technology, Gaithersburg, Maryland, 20899
[2]Graduate Student, and [3]Assistant Professor, Department of Civil and Environmental Engineering
University of Central Florida, Orlando, Florida, 32816

Research Objectives

The long-term objective of this project is to develop performance-based approaches to the seismic design, retrofit, and repair of reinforced concrete bridge columns. The objectives of the current task, Phase II, are to use the experimental data obtained in Phase I to evaluate and calibrate existing analytical damage models for reinforced concrete columns, to derive improved damage models, and to develop methods of using these damage models in practical design applications.

Research Approach

Background: In earthquake engineering studies of reinforced concrete bridge columns, laboratory test specimens have traditionally been loaded with a controlled, cyclic lateral displacement pattern with gradually increasing amplitude. However, in actual earthquakes bridge columns are exposed to random cyclic lateral loading patterns, which are much different from the typical laboratory loading patterns. Current American Association of State Highway and Transportation Officials (AASHTO) and California Department of Transportation (Caltrans) seismic design provisions are based almost entirely on tests in which traditional, controlled laboratory loading patterns have been applied. The differences in the effects of these two types of loading - controlled, cyclic lateral loads, and random earthquake type loads - have never been explored systematically. In this study both types of loading were applied to a series of twelve circular, cantilever flexural columns. The differences in observed damage are now being studied, and a model which relates damage from controlled laboratory loading, and damage from random earthquake loading, is being investigated.

Experimental Program: In Phase I of this study tests were performed on twelve nominally identical cantilever bridge columns. The test specimens were 1:4 scale models of prototype single column bridge bents designed according to Caltrans standards. The height to diameter ratio of the columns was 6, so the behavior of the columns was dominated by flexure. A constant axial load, simulating the dead load of a superstructure, was applied in all tests. The primary variable in the twelve column tests was the lateral displacement history applied at the top of the column. In the first six tests "benchmark" displacement histories were applied: monotonic pushover to failure; a "standard" cyclic displacement pattern (gradually increasing amplitude to failure, as shown in Figure 1); and four constant-amplitude cyclic tests at $\pm 2\Delta y$, $\pm 3\Delta y$, $\pm 4\Delta y$, and $\pm 5\Delta y$ to failure. In the final six tests series of random displacement patterns, simulating earthquake loading, were applied to each column. Displacement histories for earthquakes of various magnitudes were calculated using the inelastic analysis program IDARC. A series of 4 or 5 events, of varying severity, was applied to each column, as shown by the example in Figure 2. The order of the events was varied. For example, one specimen was subject to two minor events followed by a major, then a severe event; while another specimen was subject to a major event, followed by two minor events, and finally a severe event. In all twelve tests observations of the progress of damage were made both visually and with instruments.

Currently, in Phase II, the damage data gathered during Phase I are being evaluated, and damage model hypotheses for well-confined, flexure-dominated columns are being developed. The damage data was first compared with several existing damage models. Most of these models had significant shortcomings. However, initial review of the data indicates that for well-confined columns, damage can be expressed satisfactorily as a function of low-cycle, high-amplitude fatigue of the longitudinal reinforcing bars. Allowance must also be made for failure of the confining reinforcement, although the method for doing this has not yet been determined.

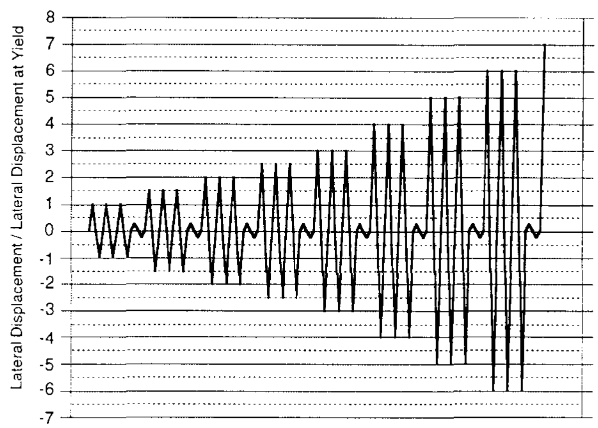


Figure 1: Typical laboratory displacement history (applied to specimen 2)

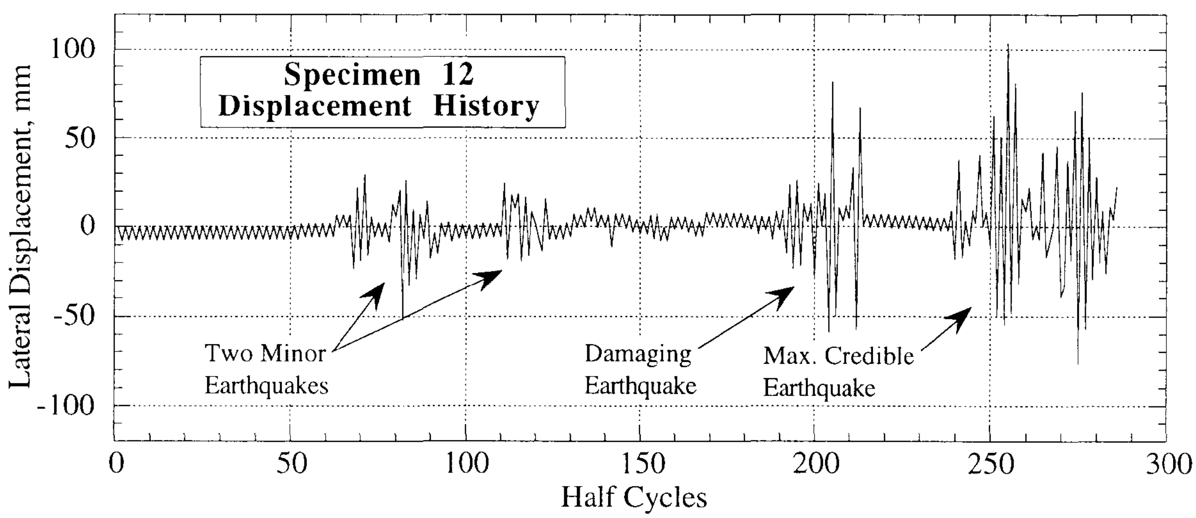


Figure 2: Typical earthquake displacement histories (applied to Specimen 12)

Expected Products

The primary product of this study will be an analytical damage model for circular, flexure-dominated, reinforced concrete bridge columns which reflects equally well the damage caused by controlled laboratory loading and the damage caused by random earthquake loading.

Based on the findings of the damage model studies in Phase II, a performance-based approach for design and evaluation of well-confined, circular, flexure-dominated bridge columns will be proposed. It is anticipated that this model will be based on a comparison of damage capacity and damage demand. Damage capacity might be expressed as the maximum number of equivalent inelastic cycles that could be sustained by a column at several different levels of constant displacement ductility; damage demand might be based on the anticipated (or recorded) inelastic cycles at a site, converted to a series of constant-amplitude inelastic cycles.

Preliminary Results

Constant Amplitude Tests: It was observed that repeated cycling of specimen 3 at a displacement amplitude of ±2Δy caused almost no degradation of stiffness and strength of the column. After the first full cycle at ±2Δy (during which initial yielding and cracking occurred) the hysteresis loops remained extremely stable, lying nearly on top of one another for 150 cycles. The test was stopped at 150 cycles not because the column failed, but because it was believed that no further useful information could be obtained by continuing the test. At the other extreme, specimen 6 was subjected to cycles of ±5Δy and exhibited a rapid decrease in strength and stiffness, nearly completely losing lateral stiffness and load capacity after only 5 cycles. This rapid deterioration is illustrated in Figure 3. Specimen 5, which was cycled at ±4Δy, exhibited a gradual decrease in strength and stiffness as cycling progressed, but the decrease was not nearly as rapid as for specimen 6. This is illustrated in Figure 4.

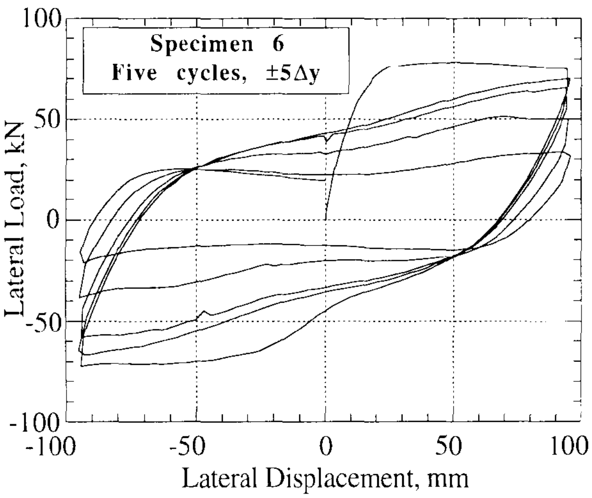


Figure 3: Load-displacement plot, spec. 6

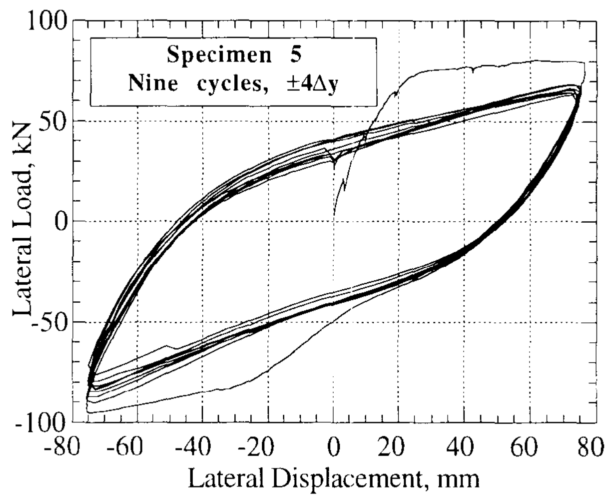


Figure 4: Load-displacement plot, spec. 5

Thus, for the specimens tested in this program there appears to be a threshold ductility level, of about 4Δy, above which deterioration is rapid and severe. It could be surmised that earthquakes which induce displacements of less than 4Δy in the columns tested in this program would cause much less damage than those which induce displacement greater than 4Δy. Indeed, subsequent testing under earthquake displacement patterns (specimens 7 to 12) confirmed this observation: a series of several minor events, which caused few excursions greater than 2Δy, would result in very little damage to a column, while a single earthquake with a few excursions greater than 4Δy would result in rapid deterioration of strength and stiffness. The apparent threshold of 4Δy applies only to the columns tested in this program. It could be expected that similarly-designed circular columns, with high ratios of confining reinforcement (such as those designed under the Caltrans specifications), and which were dominated by flexural rather than shearing deformations, would also exhibit such a threshold, but the level of the threshold might not necessarily be 4Δy.

Another significant observation from the constant amplitude tests was that cumulative dissipated energy (the area contained within the hysteresis loops) is not good predictor of column failure. It was found that the cumulative dissipated energy at failure depended on the amplitude of the sawtooth wave. Figure 5 shows the accumulation of energy to failure for specimens 2, 4, 5 and 6 ("standard" displacement pattern with gradually increasing amplitude, ±3Δy, ±4Δy, and ±5Δy, respectively). Results for specimen 3 are not plotted because the specimen did not deteriorate significantly under 150 cycles at ±2Δy, so a failure state was not achieved. In Figure 5 it can be seen that dissipated energy at failure is strongly dependent on the displacement history. Therefore, because of the highly variable nature of earthquake-induced displacement histories, it is apparent that cumulative energy alone is not an acceptable measure of column damage.

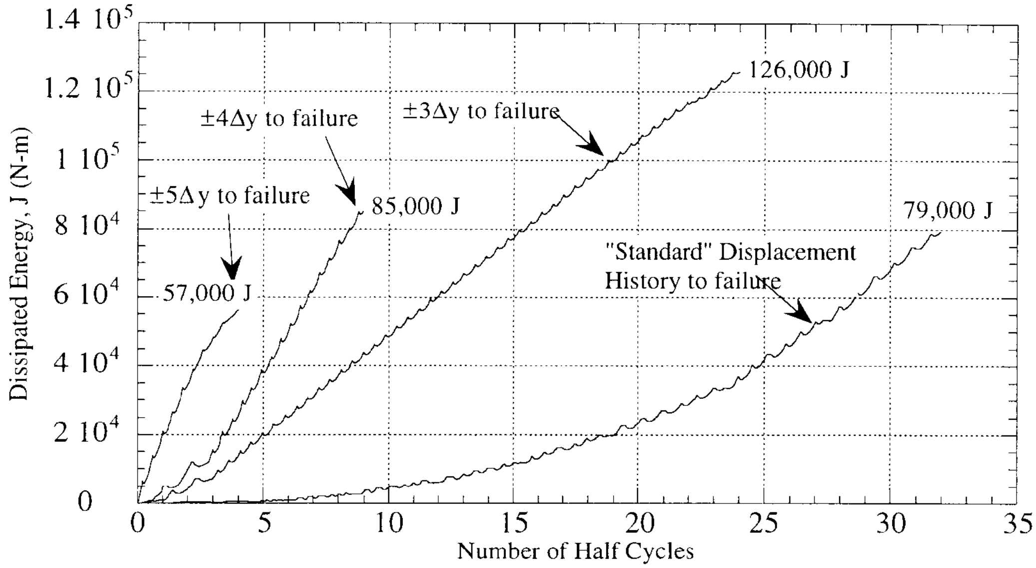


Figure 5: Cumulative energy to failure for specimens 2, 4, 5 and 6.

<u>Earthquake Loading Tests:</u> There are two preliminary findings from the earthquake loading tests:

First, it was observed that minor earthquakes had little or no effect on the ultimate failure of a column. That is, it made no difference if minor events occurred before or after a damaging event: the level of damage following the damaging event was essentially the same in both cases. This has important implications for the management of earthquake damage to bridge columns. It would appear that minor seismic events should be considered to have little effect on well-confined, circular, flexure-dominated bridge columns. It remains for transportation authorities to define precisely what a "minor" event is, but it appears acceptable to not discount the strength and stiffness of well-confined columns which have been subjected to a minor earthquake.

Second, it was observed that failure of the columns could be classified into two general types: failure due to low cycle fatigue of the longitudinal reinforcement, called here a "low cycle fatigue failure"; and failure due to rupture of confining reinforcement, or "confinement failure." Confinement failures were observed to occur more frequently than low cycle fatigue failures. While these two classes of failure have been observed and reported by others, it is interesting to recall that the only variable in these tests was the displacement history. Thus displacement history, rather than the column configuration, determined the failure mode. This illustrates the importance of conducting nonlinear dynamic analyses of bridge columns whenever feasible, and considering a range of possible ground motions at the bridge site, rather than permitting a single earthquake record to determine the design.

With regard to analytical damage modeling, preliminary analyses of the test results indicate that cumulative fatigue models, based on variations of classic fatigue rules for metals, can successfully track damage in columns which fail due to low cycle fatigue of the longitudinal reinforcement. However, this same approach does not apply well to columns which exhibit confinement failure. Other methods of modeling confinement failure are currently being investigated. Furthermore, the columns tested in this study were dominated by flexural behavior, and none of the findings stated here necessarily apply to short, shear-dominated columns. A similar experimental investigation of shear-dominated columns is planned.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Seismic Fragility Analysis of Conventional Reinforced-Concrete Highway Bridges

Author(s) and Affiliation(s):

Chris L. Mullen and Ahmet S. Cakmak
Department of Civil Engineering and Operations Research
Princeton University, Princeton, NJ

Principal Investigator: Ahmet S. Cakmak

Sponsor(s): Federal Highway Administration and National Center for Earthquake Engineering Research

Research Start Date: July 1995
Expected Completion Date: June 1996

Research Objectives:

Seismic fragility curves are constructed for conventional reinforced-concrete highway bridges on the basis of detailed structural analysis. Such curves provide a rational basis for improving the Applied Technology Council's ATC-13 damage-probability matrices that were constructed on the basis of expert opinion alone. The analyses themselves also provide insight into various aspects of highway bridge seismic damage modeling and dynamic response.

Expected Products or Deliverables:

Fragility curves and corresponding damage-probability-matrices for conventional highway bridge structures are constructed for comparison with ATC-13. Simulated time histories, damage state predictions, and damage-intensity relationships of structures analyzed using nonlinear dynamic finite element method provide new knowledge on damage behavior of these structures.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Chris L. Mullen is a Ph.D. candidate in the Mechanics, Materials, and Structures Program at Princeton University. His dissertation is on the 3D nonlinear-dynamic finite element analysis of seismic damage in reinforced-concrete bridges as applied to the Meloland Road Overcrossing. He is a Professional Engineer registered in New York where he worked on a variety of structural engineering rehabilitation, monitoring, and inspection projects for major East River suspension bridges that carry both highway and subway traffic. Prior to his work on bridges, he was involved in computer-aided fatigue analysis of offshore structures, welded tubular connections, and fatigue-sensitive engine components. His current research interests are in large-scale 3D seismic damage analysis of structure-foundation systems using supercomputers and in the development of in-service performance monitoring of structural systems using motion and local strain measurements compatible with finite element and system identification models. Mr. Mullen is a member of ASCE and IABSE.

Ahmet Ş. Çakmak is Professor of Civil Engineering and Chairman of the Department of Civil Engineering and Operations Research at Princeton University. His tenure at Princeton has spanned over 30 years during which time he served as the Associate Dean of the School of Engineering and Applied Science and Visiting Professor at Boğazici University, University of Southampton, California Institute of Technology, and Kyoto University. His current research interests are the application of materials technology to the past and future seismic resistance of ancient masonry construction in the Hagia Sophia structure, seismic damage assessment of reinforced-concrete structures, and stochastic response analysis of nonlinear multi-degree-of-freedom structures with random properties subjected to random excitation. Dr. Çakmak is a Professional Engineer registered in New Jersey and is a member of ASCE, ASME and the Society of Rheology. He is the founding editor of the *International Journal of Soil Dynamics and Earthquake Engineering* which he continued to edit for 11 years, chair and proceedings editor of a series of six International Conferences on Soil Dynamics and Earthquake Engineering that began in 1982 and continues to the present, author of books on the structure of the Hagia Sophia and on Applied Mathematics in Engineering Analysis, and a reviewer for the National Science Foundation and ASCE journals.

Principal Investigator's Telephone Number: (609) 258-4601
Facsimile Number: (609) 258-1309
E-mail Address: ahmet@tremor.princeton.edu

SEISMIC FRAGILITY ANALYSIS OF CONVENTIONAL REINFORCED-CONCRETE HIGHWAY BRIDGES

Chris L. Mullen and Ahmet Ş. Çakmak
Department of Civil Engineering and Operations Research
Princeton University, Princeton, NJ 08544, USA

Research Objectives

Seismic fragility curves are constructed for conventional reinforced-concrete (RC) highway bridges on the basis of detailed structural analysis. Such curves provide a rational basis for improving ATC-13 Damage-Probability-Matrices (DPM) that were constructed on the basis of expert opinion surveys alone. The analyses themselves also provide insight into various aspects of highway bridge seismic damage modeling and dynamic response.

Research Approach

Bridge Classes: Fragility is defined as the conditional probability of exceeding a specified damage limit state given excitation of a specified intensity level. Seismic fragility is studied for three important classes of RC multi-cell box-girder highway bridges.

1. The short, straight two-span overpass with a single-column central pier, no deck expansion joints or hinges, no girder bearings, and no cable restrainers.
2. The long, curved multi-span connector with single-column piers, deck expansion joints and hinges, girder bearings, and cable-restrainers.
3. The intermediate length, straight, multi-span overpass with two-column piers, highly skewed deck expansion joints and hinges, girder bearings, and cable-restrainers.

The first class is represented in the study by the Meloland Road Overcrossing (MRO) which serves as a base case since it sustained no significant externally visible superstructure damage during the M= 6.4 Imperial Valley event even though it was located only 1 km from the epicenter. Furthermore, the bridge was equipped with an extensive 3D array of accelerometers on the superstructure during the event whose time history records have been given thorough analysis by a variety of other researchers. The second class is represented by the SR14/I5 Separator and Overhead which suffered major column and deck collapse during the M= 6.7 Northridge event and was located only 12 km from the epicenter. The third class is represented by the Gavin Canyon Undercrossing (UC) which suffered major deck collapse during the Northridge event and was located only 14 km from the epicenter.

Bridge Modeling: Time history analysis is performed for each structure-foundation system using 3D nonlinear-dynamic finite element modeling that includes the following special considerations:

1. Fiber model for RC column damage allowing 3D interaction of axial force and biaxial bending moments on the cross-section.
2. Lumped spring/dashpot representation of soil-structure interaction and expansion joint behavior.
3. Seismic input motions include the effects of spatial variation of the propagating waves.

The columns of the bridges are discretized into beam-column elements in a pattern that captures effects of plastic hinging and significant changes in cross-section shape or reinforcement. The multi-cell box-girder is modeled using plate elements permitting accurate description of the mass and stiffness distribution affecting the global response modes that contribute to structural damage. For the SR14/I5 and MRO both of which have single-column piers, the important modes are dominated by lateral bending and torsion about the longitudinal axis of the deck. Vertical bending and torsion about the vertical axis of the deck are important for the two-column piers of the skewed Gavin Canyon UC.

Simplified interaction of the superstructure with the pier foundations and the embankment soil (SSI) is included using truss elements that mimick a lumped spring/dashpot system. Figure 1 shows the SSI model for the MRO.

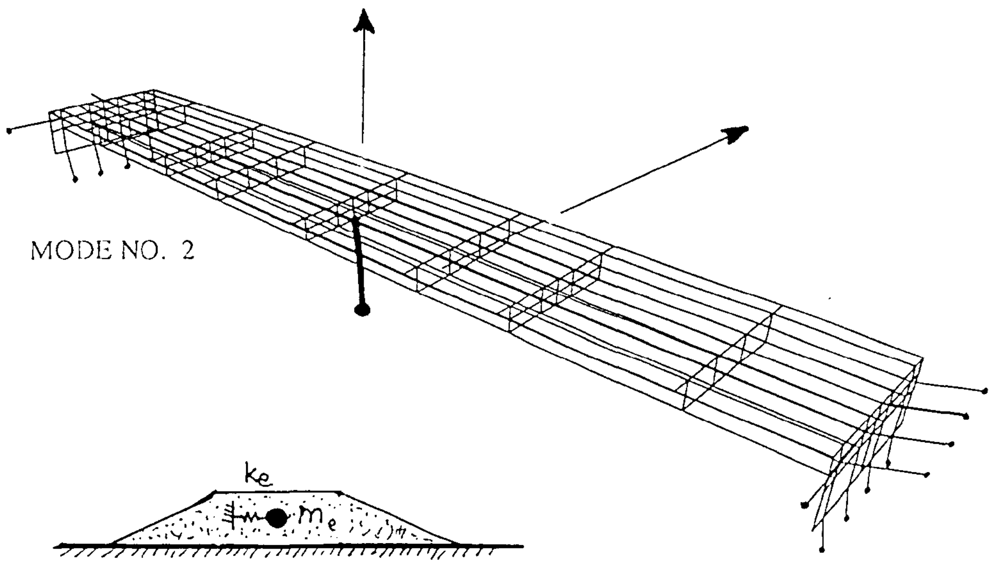


Figure 1. Soil-structure interaction model for the Meloland Road OC.

A more detailed SSI in which the soil mass is discretized as a 3D continuum requires use of a supercomputer which is not feasible for this study. Input motions are applied to grounding points of the truss elements. The effective embankment mass is also lumped at a joint with lateral freedom that is tied to the abutment through one of the truss elements.

Overall 3D dynamic characteristics of the bridge/foundation system are reflected in the eigenfrequencies and eigenmodes which are matched with the best available information for the given structure. In the MRO case, the above modeling approach achieved excellent agreement with system identification results using only engineering estimates of the embankment mass/stiffness, elastic moduli in the deck plate elements reflective of cracked-section properties for the overall deck, and measured strengths of the concrete and steel materials in the column element.

Damage Modeling: The damage modeling focuses on the flexural behavior of the column through the fiber beam-column element. The principle of the element is that the composite action of the steel and concrete materials may be idealized through the behavior of interacting longitudinal fibers that resist normal stress and strain at each cross-section. The fibers obey a common strain-compatibility relationship at each cross-section but may observe different constitutive relationships based on the material type and, in the case of the concrete, the level of effective confinement. Complex cracking, plastic loading, softening/crushing, and spalling behaviors in the concrete may be represented in an effective manner through the specification of 1D stress-strain envelope curves and path-dependent, loading/unloading rules that are keyed on a few key strain parameters.

The six degree-of-freedom actions at each member end are tied to the fiber response at two cross-sections within the member length through the numerical integration procedure and interpolating functions. No assumption of lumped plasticity at the ends is made, therefore, and the full extent of cracking and plasticity can be obtained by the choice of discretization of the member. Such discretization is generally required in any event because changes in either cross-section geometry, e. g. tapering, or in reinforcement detailing often occur along the member length.

Fragility Modeling: Three measures of damage and corresponding limit states are identified for the fragility analysis:

1. The maximum compressive concrete strain in the outer fibers at the column base.
2. The ductility with respect to relative lateral displacements of the column ends.
3. The maximum-softening damage index at the free-edge of the pier cap.

These damage measures represent the spectrum of local to global indicators, going from point to member to system, respectively. Correlations between the various measures at a given intensity level provide a broader basis for retrofit assessment.

Two measures of input ground motion intensity are identified for the fragility analysis, also to provide a broader picture:

1. The peak ground acceleration (PGA).
2. The root-mean-square (RMS) acceleration.

The PGA reflects the influence of the peak inertial force and the single-most damaging cycle while the RMS reflects the influence of strong motion duration and multiple damaging cycles. Correlations between PGA and Modified Mercalli Intensity are used to place the results on the same basis as the ATC-13 DPM's.

A semi-deterministic approach is taken in the assessment of fragility here. Time history analysis is performed for three sets of basic information on each of the basic structure models.

A mean damage-intensity relationship is first established by increasing the intensity measure by a simple scaling of a reference time history record. In the case of the MRO, measured motions are available at the site itself. For the SR14/I5 and Gavin Canyon UC, records at nearby sites represent the best available information. For this series of simulations, the best estimate of the model parameters is used for each bridge.

The first estimate of fragility is obtained by introducing uncertainty associated with the bridge model itself. The principle sources of such uncertainty are taken to be the stress-strain parameters used to model confined and unconfined concrete behavior. Secondary uncertainties are taken to be the SSI parameters such as the embankment lumped properties and the deck elastic moduli. Only variabilities consistent with the estimates of natural frequencies for the structures are considered, however.

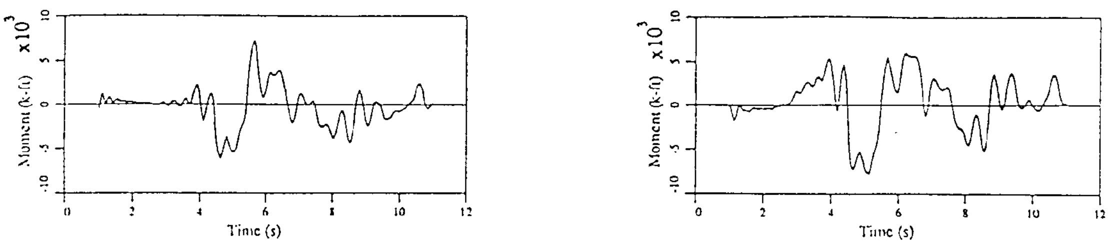
The second estimate of fragility is obtained by introducing uncertainty associated with the input ground motion characteristics for a given intensity level.

Expected Products

Fragility curves and corresponding DPM's for conventional highway bridge structures are constructed to compare against the DPM developed by ATC-13 on the basis of expert opinion. The simulated response time histories, damage state predictions, and damage-intensity relationships of each structure analyzed will provide new knowledge regarding seismic damage and dynamic response behavior of the structure classes considered.

Preliminary Results

The analysis of the MRO is nearing completion. Figure 2 shows transverse bending moment time histories predicted for the MRO using the Imperial Valley transverse and vertical acceleration records. Figure 2a shows response when measured records for each abutment and the column base are input directly such that no SSI is required. Figure 2b shows response for the SSI model where the measured records for the column footing level are input to the abutment/embankment grounding points.



a. Fixed abutment- no SSI.

b. Embankment model- SSI.

Figure 2. Column base transverse moment time histories for the MRO.

The current state of development for the SR14/I5 model is shown in Figure 3. As-built drawings are being reviewed for development of the Gavin Canyon UC model.

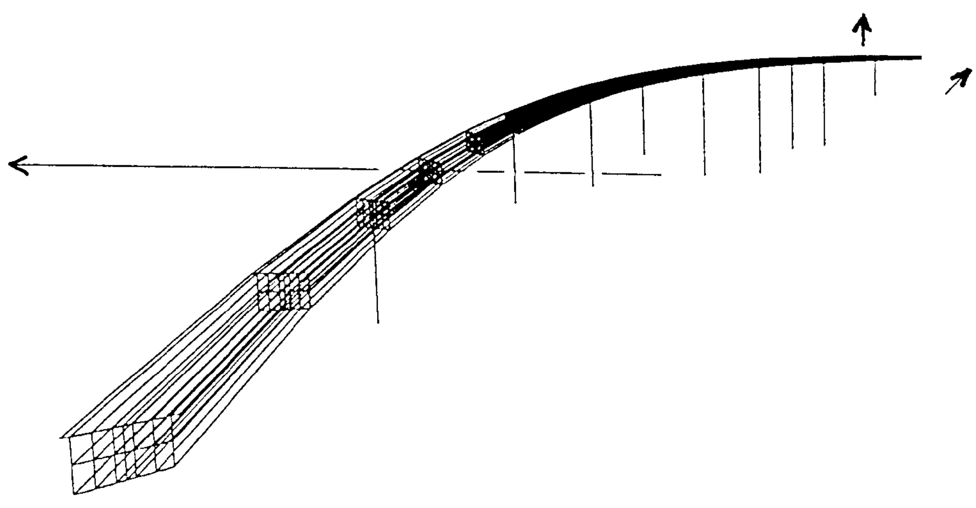
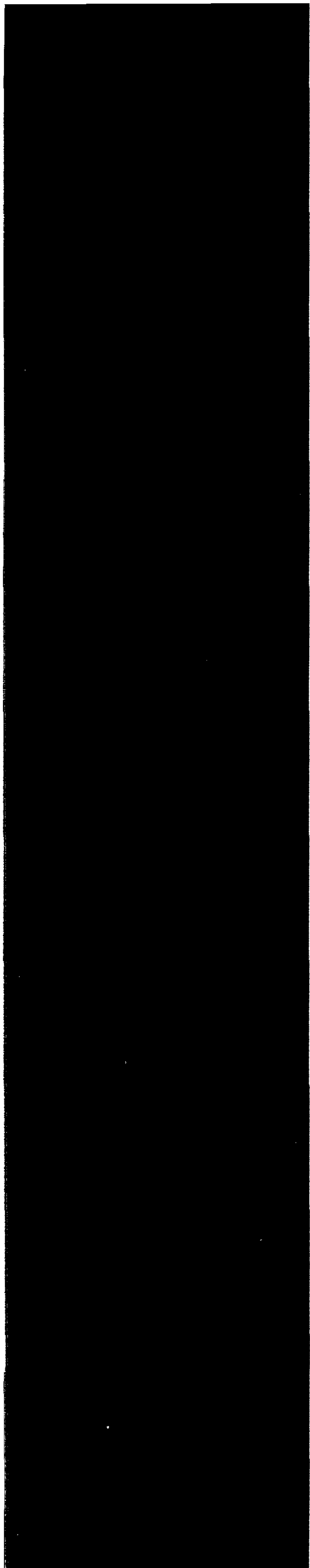


Figure 3. Preliminary model for the SR14/I5 Separator and Overhead.

Concrete, Masonry, and Composite Construction

SESSION III



FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS RESEARCH ABSTRACT

Title: Release Methodology of Prestressing Strands

Author(s) and Affiliation(s):

Jeffrey Kannel, Catherine French, and Henryk Stolarski
Department of Civil Engineering
University of Minnesota, Minneapolis, MN

Principal Investigator: Catherine French

Sponsor(s): Minnesota Department of Transportation

Research Start Date: November 1994
Expected Completion Date: May 1996

Research Objectives:

Cracks have been observed to form in the ends of pretensioned bridge girders when using the flame-cutting technique to release the girders from the precasting bed. Because some of the cracks were observed to remain open following the full release of the girders. the effectiveness of the bond between the strands and the concrete in the end regions of the girder was questioned. The objective of this study was to investigate methods to eliminate these cracks which included changes in the release methodology (strand cutting pattern) and debonding some of the strands in the end regions. An FEM model using a mesh of three-dimensional continuum elements was used to model the end regions of the girders to investigate methods to eliminate the end cracking. The most promising techniques were implemented in the field. The girder ends were instrumented to compare the field observations with the analytical results.

Expected Products or Deliverables:

As a result of the investigation, a fabricator has already implemented some of the recommended changes to the strand release order, which have proven to dramatically reduce the end cracking observed at the precast plant. A final report is being submitted to the Minnesota Department of Transportation.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Jeffrey J. Kannel is a graduate research assistant at the University of Minnesota. He received his BSCE in 1994 from the University of Wisconsin - Madison.

Catherine E. Wolfgram French, Associate Professor at the University of Minnesota, received her BSCE in 1979 from the University of Minnesota, and her MSCE (1980) and Ph.D. (1984) from the University of Illinois at Urbana-Champaign.

Henryk K. Stolarski, Associate Professor at the University of Minnesota, received his MSCE in 1968 from the Technical University of Warsaw, Poland, an MS in Applied Mathematics in 1975 from the University of Warsaw, Poland, and his Ph.D. in Engineering Mechanics in 1974 from the Institute of Fundamental Technological Research, Warsaw, Poland.

Principal Investigator's Telephone Number:	(612) 625-3877
Facsimile Number:	(612) 626-7750
E-mail Address:	fqd6450@maroon.tc.umn.edu

RELEASE METHODOLOGY OF PRESTRESSING STRANDS

by

J. Kannel, Graduate Research Assistant
C. French and H. Stolarski, Associate Professors
Department of Civil Engineering
University of Minnesota, Minneapolis, MN

Research Objectives

Cracks have been observed to form in the ends of pretensioned bridge girders when using the flame-cutting technique to release the girders from the precasting bed. Because some of the cracks were observed to remain open following the full release of the girders, the effectiveness of the bond between the strands and the concrete in the end regions of the girder was questioned. The objective of this study was to investigate the cause of the cracks through analytical studies and field observations, and to investigate methods to eliminate the cracks which included changes in the release methodology (cutting pattern) and debonding some of the strands in the end regions.

Research Approach

The investigators observed approximately 30 girder end regions during the detensioning process. The girders comprised 45, 54, 72, and 81 in. deep I-sections. The percentage of free strand in the beds varied from 6.5 to 12.7 percent. The original strand cutting pattern used by the precasting plant is shown in Figure 1. Three of the crack types observed to develop during the release process are diagramatically illustrated in Figure 2. Most of the cracks (in particular the vertical cracks) partially closed by the end of detensioning. In addition to the observations documented by the investigators, survey forms were filled out by the plant inspector to document the formation of end cracks in other girders produced at the plant.

An earlier study conducted by Mirza and Tawfik (1978) suggested the end cracks form due to the restraining effect of the unreleased strands as the girder shortens from the partially transferred prestress. Their determination was based on a one-dimensional elastic-spring model of the casting bed which only accounted for stresses along the longitudinal axis of the girders. Because some cracks investigated in the present study were observed to develop at an angle to the girder axis, a three-dimensional finite element model incorporating all of the stress components was chosen for the investigation. In addition, the Mirza and Tawfik model used stress calculations based on beam theory, which does not accurately reflect the stress distribution in the end regions of the girders where the cracks form.

The model employed in the present study is shown in Figure 3. A mesh of three-dimensional continuum elements was used to model the ends of the girders through the transfer length region. To reduce computational demands, the remainder of the girder was modelled with beam elements. Truss elements were used to model the strands. Two methods were investigated to model the transfer of prestress from the strands to the concrete through the transfer length region. In one case, the area of the cut strand was varied linearly from zero at the face of the girder to the full area at the end of the transfer length region. This accounted for the gradual transfer of compression forces from the strands to the concrete. It should be noted, however, that there is a "reverse" transfer length developed by the remaining uncut strands which gradually transfer the restraining tensile forces from the strands into the concrete over a "tensile transfer length" region. This latter transfer length was ignored in a majority of the models. The use of spring elements to connect the continuum concrete elements to the strand truss elements was later investigated which accounted for both the tensile and compressive stress transfer. Both models appeared to give adequate results. Because the goal of the research was to propose methods to prevent end cracks from developing, an elastic material model was assumed for the concrete.

Girder symmetry was used to cut the model in half both longitudinally and transversely. Some full bed analyses were later conducted to investigate nonsymmetrical longitudinal effects created by friction and nonsimultaneous release at the girder ends. For the majority of the study, the strand release was modelled statically as strands were removed in groups to reduce the computational demands. The dynamic effects created by flame-cutting the strands were later investigated, but were not determined to be significant in the development of the end cracks.

The FEM models were run using ABAQUS to investigate the causes for the observed cracks and to investigate methods of mitigating the cracks. Methods which appeared to provide promising results were implemented in the field. In particular, three sets of three simultaneously cast 54 in. deep girders were monitored to investigate the effect of three different release methodologies. For the first set of three girders, the original cutting order used by the precasting plant was implemented which resulted in extensive end cracking during the release process. Two additional cutting patterns were used on the second and third pours. The use of debonding some of the strands in the end regions was also investigated with the third set of girders. Concrete surface gages and strand gages were attached to the girders to correlate the observed results with those predicted from the analytical models.

Expected Products or Deliverables

As a result of the investigation, the precasting plant has already implemented some of the recommended changes to the strand release order, which have proven to dramatically reduce the end cracking observed at the plant. A final report is being submitted to the Minnesota Department of Transportation.

Preliminary Results

Despite its limitations, the Mirza and Tawfik model provided some useful information. The model showed that the shorter the free length of cables in the bed, the greater the restraint from the unreleased strands. It also indicated that the inability to cut a strand at all locations in the bed simultaneously causes higher stresses adjacent to where the strand was cut first. These results were confirmed by the FEM model used in this study.

The FEM analytical model was run to investigate the original strand cutting pattern used by the precasting plant, which was shown in Figure 1. During release of the draped strands (pairs 1-6), the principal stress was oriented along the axis of the girder resulting from the tensile stresses imposed by the restraint of the uncut straight strands. The vertical cracks typically formed near the end of this stage of release. As the bottom straight strands were cut, the principal tensile stress realigned at approximately 45 degrees to the axis of the girder in the horizontal plane. The opposing forces applied by the released straight strands on the face of the bottom flange and the restraining uncut strands toward the center of the girder cross section were effectively "shearing off" the flange, causing the angled cracks. This can be seen in Figure 4, which shows contours of the axial stress and the shear stress in the horizontal plane at 6 in. from the end following release of strand pair #12. The cracks at the interface of the bottom flange and the web were caused by a stress concentration seen in the model at the end of release due to the transfer of prestress from the flange into the girder.

Three methods to control the development of cracks were investigated analytically and the most promising methods were implemented in the field. The first method comprised changing the strand cutting pattern. To reduce the vertical cracks observed, precutting some of the straight bottom strands was employed prior to cutting the draped strands. When implemented in the field, this was observed to move the cracks closer to the end of the beam where the compression from the precut strands was low due to the transfer length. To reduce the angled cracking, the strands in the bottom flange were cut in alternating columns to better distribute and balance the opposing forces discussed above. When implemented, this nearly eliminated the angled cracking.

The second method to control the cracking was the use of debonding some of the bottom straight strands in the end regions of the girder. This had two positive effects. First, the debonded length of strand worked to increase the free length of strand in the bed. The restraint forces between the girders were thereby reduced. Second, the restraint force in the debonded strand was anchored further into the girder. Because compression from precut strands is greater towards the end of the transfer length, precutting some bonded strands can be used to counteract the restraint from the debonded strands. Hence, the debonded strands should be cut last. When debonding was used in combination with precutting strands and alternating columns of cut strands, all cracks were virtually eliminated. The drawback of debonding is that bond is forcibly eliminated along the few debonded strands, which is contrary to the project goal of protecting the bond in the end regions. The benefit of this technique is that the debonding is a controlled process and the effect can be checked and accounted for in the design of the girders, whereas cracking has a less predictable effect on bond.

Increasing the slope of the top surface of the bottom flange was investigated as the third alternative. This was analytically determined to increase the stiffness of the bottom flange and also reduce the concentration of stresses at the base of the web. This method was not employed in the field investigation because it would have required modifying the girder forms.

Acknowledgements

This research has been sponsored by the Minnesota Department of Transportion. The authors also wish to acknowledge Elk River Concrete Products, a division of the Cretex Companies, for their cooperative efforts provided throughout the study. The views expressed herein are those of the authors and do not necessarily reflect the views of the sponsors.

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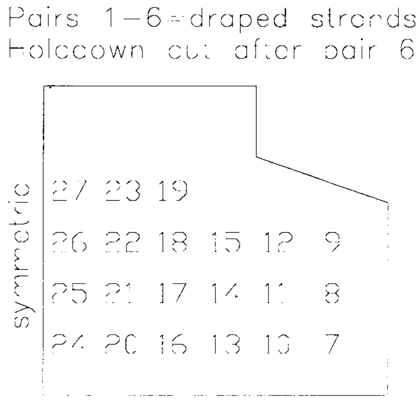

Figure 1 Cutting Pattern

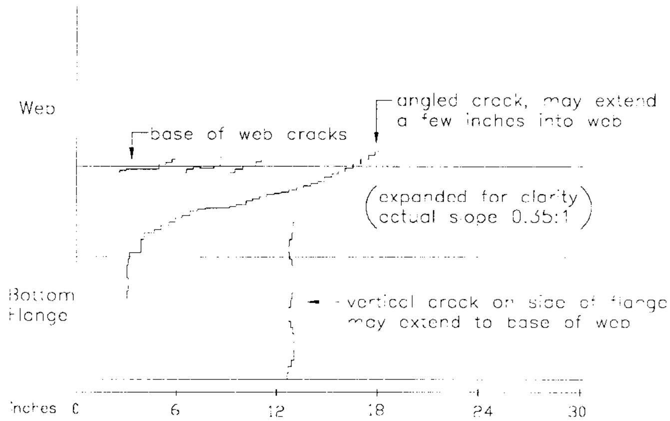

Figure 2 Types of Observed Cracks

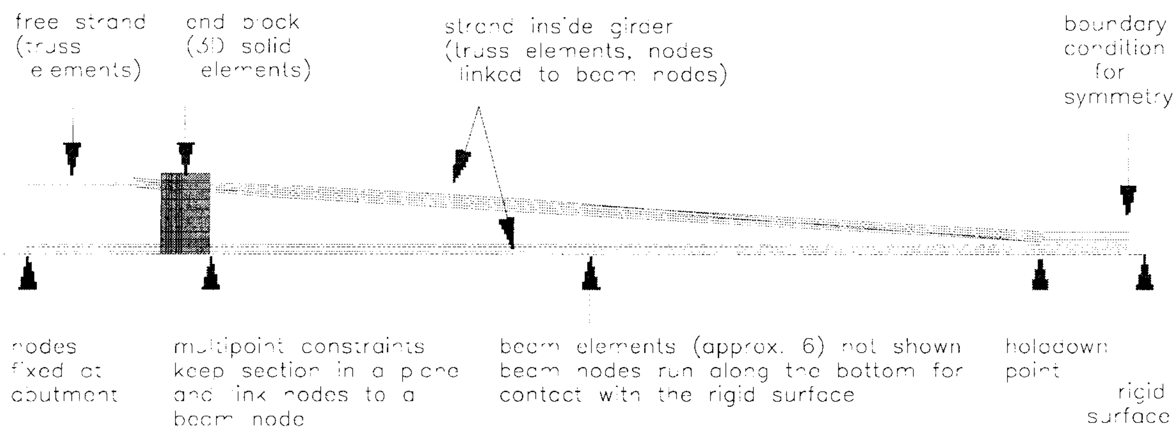

Figure 3 Finite Element Model

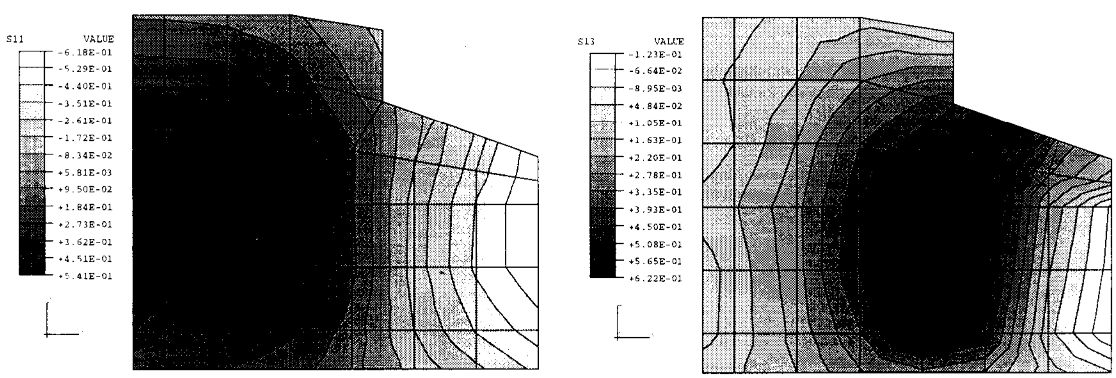

Figure 4 Contours of Axial Stress (left) and Shear Stress (right) (values in ksi)

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Durability of Bridges with Full Span Prestressed Concrete Panels

Author(s) and Affiliation(s):

Robert J. Peterman and Julio A. Ramierez
Department Civil Engineering, Purdue University, West Lafayette, IN
Randall W. Poston
Whitlock Dalrymple Poston and Associates, Manassas, VA

Principal Investigator: Julio A. Ramirez

Sponsor(s): Indiana Department of Transportation and the Federal Highway Administration

Research Start Date: September 1, 1993
Expected Completion Date: August 31, 1996

Research Objectives:

Full-span prestressed concrete form panels can be used in short-span bridges of 40 ft or less to reduce on-site formwork and labor required by conventional cast-in-place (CIP) concrete systems. These panels, which serve as permanent formwork for a CIP topping, also provide the main positive moment reinforcement for the composite system. However, bridges constructed using this procedure in Florida and Louisiana often experienced regular longitudinal and transverse cracking soon after construction. The research findings on long-term durability of this type of structure with respect to the penetration of chlorides are reported.

Expected Products or Deliverables:

This research will determine the potential durability of this type of structure. Specifically, the wide-scale use of these bridges in Indiana depends on the favorable performance of the specimens in this study. Also, recommendations for the modification of specific connection and reinforcement details for of this type of structure will be made at the conclusion of the study. Modifications to details will be based on both durability and constructibility factors.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Robert J. Peterman received his BSCE from Lafayette College in 1987 and his MS in structural engineering from Purdue University in 1989. He has three years of design experience in the prestressed concrete industry and is a registered Professional Engineer in the States of Indiana and Wisconsin. Mr. Peterman is currently a doctoral candidate in the School of Civil Engineering at Purdue University. His current research focuses on the durability and behavior of bridges with full span prestressed concrete form panels.

Julio A. Ramirez is an Associate Professor of Structural Engineering at Purdue University. He received his BSCE degree from the Universidad Autonoma de Mexico (I.T.E.S.O.) in 1977, his MS from the University of Texas at El Paso, and his Ph.D. from the University of Texas at Austin. Dr. Ramirez is Chairman of joint ACI-ASCE Committee 445, Shear and Torsion; a member of joint ACI-ASCE Committee 423, Prestressed Concrete; and a member of ACI Committee 408, Bond and Development of Reinforcement.

Randall W. Poston is currently a Principal at Whitlock Dalrymple Poston & Associates. He received his BSCE in 1978, his MS in structural engineering in 1980, and his Ph.D. in structural engineering in 1984 from the University of Texas at Austin. Dr. Poston is a member of ACI Committee 224, Cracking; a member of ACI Committee 222, Corrosion; a member of the PCI Board Committee on Research; and a member of the PCI Advisory Committee on Durability and Corrosion Protection.

Principal Investigator's Telephone Number:	(317) 494-1795
Facsimile Number:	(317) 496-1105
E-mail Address:	peterman@ecn.purdue.edu

DURABILITY OF BRIDGES WITH FULL SPAN PRESTRESSED CONCRETE PANELS

Robert J. Peterman, P.E.[1], Julio A. Ramirez, Ph.D., P.E.[1], and Randall W. Poston, Ph.D., P.E.[2]

[1]Purdue University Department Civil Engineering, West Lafayette, IN 47907
[2]Whitlock Dalrymple Poston & Associates., 8832 Rixlew Lane, Manassas, VA 22110

Research Objectives

Full-span prestressed concrete form panels can be used in short-span bridges of 40 ft. or less to reduce on-site formwork and labor required by conventional cast-in-place (CIP) concrete systems. These panels, which serve as permanent formwork for a CIP topping, also provide the main positive moment reinforcement for the composite system. However, bridges constructed using this procedure in Florida and Louisiana often experienced regular longitudinal and transverse cracking soon after construction. In this paper, the research findings on long-term durability of this type of structure with respect to the penetration of chlorides are reported. This study is being sponsored by the Indiana Department of Transportation and the Federal Highway Administration.

Research Approach

To evaluate the potential durability of this type of structure, a two-span bridge was fabricated and tested in the Karl H. Kettelhut Structural Laboratory at Purdue University, West Lafayette, IN. Each span of the bridge consisted of two prestressed concrete form panels measuring 21 ft. long, 4 ft. wide, and 6 in. thick., topped with a 6 in. thick composite CIP slab. Longitudinal steel was placed over the center pier in the CIP slab to provide for negative moment. Continuity between form panels in adjacent spans was achieved by extending the strand out of the panel ends and anchoring it in a diaphragm cast at the time of the CIP slab. The prestressed concrete panels utilized uncoated strand and rebar, while the CIP slab contained epoxy-coated reinforcement consistent with Indiana DOT practices. Composite action between the precast concrete form panels and CIP slab was attained by applying a rake finish on the top surface of the form panels.

After the CIP slab was cast and allowed to cure, the bridge was subjected to 5 million cycles of service loading by two hydraulic actuators (one per span). The loading produced a maximum stress of 44 ksi in the reinforcement over the center pier with a stress range of 18 ksi. These stresses are slightly larger than the AASHTO maximum design service load stresses and are believed to represent the extreme case of repeated service loads. In addition to the cyclic loading, the bridge was also subjected to 48 weeks of exposure cycling. The exposure cycling began after 2 million cycles of service loading were applied to ensure that crack patterns had been fully established. This weekly cycle consisted of 4 days of exposure to a 15 percent sodium chloride solution followed by 3 days of drying at a minimum of 100° F. The exposed area of the bridge was the critical area over the center pier which had transverse cracking. The durability of these bridges was assessed using the following criteria:

1. The potential for delamination between the precast panels and cast-in-place topping.

2. The resistance to chloride-induced reinforcement corrosion.

Potential For Delamination: Bridges constructed with form panels often experience regular cracking near panel joints and at pier locations. These cracks usually extend through the entire CIP slab but not into the form panels which are precompressed. The Indiana Department of Transportation expressed concern that chlorides from road de-icing salt would penetrate into the deck through these cracks and accumulate at the interface between the CIP slab and the prestressed form panels. Since the prestressed panels would typically have a higher strength and lower permeability than the CIP slab, it was believed that these chlorides would then spread out horizontally along the interface instead of permeating into the prestressed panels. Over time, this could create planes of weakness at the interface and possibly cause delamination.

The potential for delamination was assessed, in part, by taking chloride powder samples from the bridge deck at 12-week intervals. This consisted of drilling into the bridge deck near crack locations and collecting the powder at 1/2" increments. The powder was then tested for total chloride ion content using the AASHTO Standard Test T 260 procedure. The holes were drilled at least 7" deep, extending past the interface and into the prestressed panels. Powder samples were also taken at a distance of at least 12" from all cracks and compared with data from bridges in service to help estimate the actual time simulated by the accelerated testing cycles.

Impact-echo tests were also performed on the test bridge following the 5 million cycles of load and 48 weeks of exposure cycling. Impact-echo is a non-destructive testing method which relies on interpreting resonating reflections from transient stress waves introduced into the concrete by impact. A broad-band displacement transducer placed near the point of impact monitors the resonating compression waves (P-waves) of interest. For the case of the model bridge tested in the laboratory, reflections were from either the through thickness of the bridge ($\approx$ 12 in.) or, if disbondment occurred, from the interface of the CIP slab and precast panels ($\approx$ 6 in.). Impact-echo testing was conducted prior to final load testing through the point near failure to assess whether there was any tendency for disbondment as the load increased.

Resistance to Reinforcement Corrosion: Composite action was ensured by raking the top surface of the prestressed form panels in lieu of using steel reinforcement to resist the horizontal shear forces. Therefore, electrical interconnection between the steel in the form panels and the epoxy-coated steel in the CIP slab was eliminated. This reduced the potential for macrocell corrosion to occur between the two mats of steel, presumably resulting in a structure that should be more corrosion-resistant than conventional CIP concrete bridges.

To evaluate the resistance to reinforcement corrosion, both half-cell potentials and corrosion current were monitored weekly throughout the exposure cycling. Copper-copper sulfate half-cell potentials were measured by connecting a wire to four epoxy-coated rebars in the CIP slab. Potentials were then measured at the surface of the bridge deck throughout the ponded region at regular intervals. Corrosion currents were measured by placing a 100Ω resistor between wires connected to the four epoxy-coated bars in the CIP slab and wires connected to the mat of uncoated strand and mild reinforcement in the form panels. Voltage drop across the resistor was recorded and the corresponding corrosion currents were determined by Ohm's Law.

Expected Products

This research will determine the potential durability of this type of structure. Specifically, the wide-scale use of these bridges in Indiana depends on the favorable performance of the specimens in this study. Also, recommendations for the modification of specific connection and reinforcement details for of this type of structure will be made at the conclusion of the study. Modifications to details will be based on both durability and constructibility factors.

Preliminary Results

The laboratory phase of this project has recently concluded and the data from the test bridge is currently being analyzed. Some preliminary results from this study are listed below.

Chloride Powder Samples: Chloride powder samples taken after 48 weeks of exposure showed that the cycling had produced levels of chlorides at the depth of the reinforcement which greatly exceeded values from some of the oldest bridges in Indiana. Figure 1 shows the average total chloride ion contents for the laboratory test bridge after 48 weeks of exposure as well as those from a 37-year-old bridge on State Rd. 144 near Mooresville, Indiana. The bridge on S.R. 144 is the oldest bridge in the central part of the state that has not been previously overlaid or resurfaced. Figure 1 shows the chloride concentrations of the test bridge to be at least 2 times greater that those of the 37-year-old structure at all depths sampled. Figure 1 also illustrates the importance of reinforcement cover in regard to durability. For the S.R. 144 bridge, which had a cover of 1 1/4 inches, an extra inch of cover (i.e. from 11/4" to 2 1/4") will result in only 36% of the chloride concentration at the rebar depth (from 8.5 lb/yd^3 to 3.1 lb/yd^3). After 48 weeks of exposure cycling a significant accumulation of chlorides at the interface between the form panels and CIP slab of the test bridge was not observed. Numerous powder samples taken from the test bridge near cracks revealed that an accumulation of chlorides, if existing, remained isolated at the crack locations. Therefore, delamination caused by a buildup of chlorides at the interface should not be a problem during the anticipated life of these structures.

Impact-Echo Tests: The impact-echo testing conducted prior to loading the bridge to failure clearly indicated that the CIP slab was bonded to the precast panels. This showed that there had been no disbondment due to any chloride buildup at the interface. The impact echo tests taken at the various load levels up to the point near failure also clearly indicated that there was no significant loss of bond between the CIP slab and precast panels.

Half-Cell Potentials: Weekly half-cell potential readings indicated that corrosion of bridge deck reinforcement probably began during the first week of exposure and continued throughout the remainder of the test. Isopotential contour maps plotted from the half-cell data showed that the corrosion was most severe near the diaphragm region at the center pier. This area of the deck had several transverse cracks induced by shrinkage of the concrete and from the initial loading.

Corrosion Current: Measured corrosion currents indicated that corrosion of the epoxy-coated reinforcement began during the first week of exposure and then increased significantly after 27 weeks of exposure. From chloride powder samples taken at 24 weeks, it is known that this was soon after chlorides reached the entire mat of epoxy-coated steel. Therefore it seems likely that corrosion of the epoxy-coated steel initiated during the first week of exposure at crack locations and then became more widespread after chlorides permeated to other areas of the bars.

Visual Inspection of Bars: At the conclusion of the test, a portion of the ponded area of the bridge was cut up and the reinforcement was extracted, revealing corrosion of the epoxy-coated bars. This was typical of all bars and appeared most severe near the diaphragm connection. Corrosion of the epoxy coated bars was accompanied by staining of the surrounding concrete and often by longitudinal splitting of the epoxy coating. This splitting was likely due to the formation of expansive corrosion products beneath the surface of the epoxy coating. Also, the ends of the strands protruding out of the panels which were anchored in the diaphragm at the pier experienced severe corrosion with significant area loss due to pitting. It is therefore recommended that this detail be modified to ensure long-term performance. One alternative would be to use an epoxy-coated mild steel connection between panels in adjacent spans instead of extending the uncoated strand into the diaphragm for anchorage.

The results of this study indicate that the increased use of full-span prestressed form panels in Indiana is promising. Concerns of possible delamination due to accumulation of chlorides have been dismissed. While corrosion of the epoxy-coated steel was observed at chloride concentrations that exceeded levels of some of the oldest bridges in the state, it did not affect the structural performance of the test bridge and was much less than the corrosion of the uncoated strands. It is believed that adequate long-term performance can be achieved using prestressed concrete panels with epoxy coated bars in the CIP slab provided that a high-quality concrete with adequate cover is ensured.

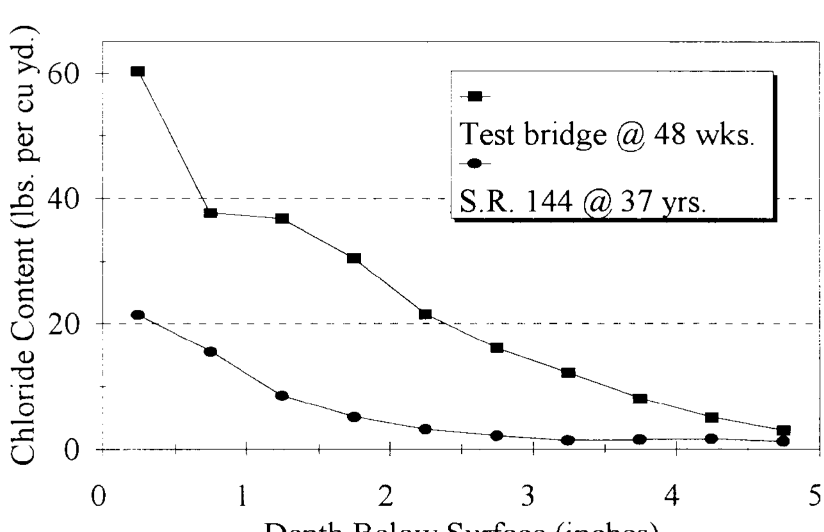

Figure 1. Total Chloride Ion Concentrations

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: High Performance Concrete in Texas Bridges

Author(s) and Affiliation(s):

Ned H. Burns, Shawn P. Gross and Michael O. Braun
Department of Civil Engineering
University of Texas at Austin, Austin, TX

Principal Investigator: Ned H. Burns

Sponsor(s): Texas Department of Transportation and the Federal Highway Administration

Research Start Date: July 1993
Expected Completion Date: September 1999

Research Objectives:
The two bridges in this research have quite different cross sections (U-shape and I-shape) and are in two different parts of Texas (370 miles apart). Both research projects have the overall objective of monitoring actual performance using high performance concrete (HPC) with maximum concrete strength in the 13-15 ksi (41 to 103 MPa) range. Development of the mix designs for the actual concrete mixes used in the construction of both the prestressed concrete girders and the decks for these bridges was an important objective. Documentation of the actual strength achieved in the construction of the bridges, as well as the shrinkage and creep characteristics of HPC, is being obtained. Instrumentation installed in the bridges will monitor performance of HPC over the period of the project.

Both projects use 0.6-in diameter strands on a 2 x 2-in (50 mm) grid layout. Data on the transfer length of these strands in pretensioned prestressed concrete members is an important objective. Tests also being conducted on development length, for 0.6-in strand.

Expected Products or Deliverables:

Two HPC bridges in Texas are being built in this project -- one in Houston and the other in San Angelo. The actual performance of these bridges is being carefully monitored, and the actual long-term observed behavior of HPC will support improved design practice with this material.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Ned H. Burns is the director of Ferguson Structural Engineering Laboratory and Zarrow Centennial Professor in Engineering at The University of Texas at Austin. He has been a research scientist and a faculty member of the Department of Civil Engineering at the University for over 38 years. He received his BSCE and his MSCE from the University of Texas at Austin in 1954 and 1958, respectively. He received his Ph.D. in civil engineering from the University of Illinois - Urbana in 1962. Over the years he has received many honors including the following: the Texas Society of Professional Engineers recognized him as the Outstanding Young Engineer of the Year in 1962 and the Engineer of the Year in 1987; the Joe W. Kelly Award from ACI in 1990; the Martin P. Korn Award from the Precast/Prestressed Concrete Institute in 1993; and the T.Y. Lin Award from ASCE in 1994. He is and has been an active committee member of ACI, PCI and PTI. He also is a member of ASCE, the American Society for Engineering Education, the Texas Association of College Teachers, NSPE, and the Texas Society of Professional Engineers. He is co-author with T.Y. Lin of the textbook Design of Prestressed Concrete Structures, 3rd Ed., John Wiley and Sons, Publishers, New York, 1981. As a principal research investigator, Dr. Burns has spoken at conferences around the world and co-authored numerous published papers and reports on precast, pretensioned concrete and post-tensioned concrete for buildings and bridges.

Shawn P. Gross is a Ph.D. candidate at The University of Texas. He received his BSCE from Tulane University in 1993. Mr. Gross has worked on laboratory testing, beam instrumentation and field data collection for the Louetta Road Overpass research project since September 1993 and has performed the same duties for the San Angelo Bridge research project since June 1995. He has co-authored a report and a paper on high performance concrete, and was a presenter at the SHRP High Performance Concrete Bridge Showcase in March 1996, which was sponsored by FHWA and TxDOT.

Michael O. Braun earned his BSCE degree from the University of Wisconsin at Platteville in 1975. He worked for private consulting engineering firms in San Antonio, Texas for 13 years before joining TxDOT in 1991. Currently he is an MS candidate at The University of Texas as part of TxDOT's Masters of Science Graduate Engineering program. He worked on laboratory tests for the San Angelo Bridge research project. Mr. Braun is a registered Professional Engineer.

Principal Investigator's Telephone Number: (512) 471-4509/(512) 471-1619
Facsimile Number: (512) 471-1944
E-mail Address:

HIGH PERFORMANCE CONCRETE IN TEXAS BRIDGES

Ned H. Burns, Ph.D., P.E.[1], Shawn P. Gross[2], and Michael O. Braun, P.E.[2]

[1]Zarrow Centennial Professor in Engineering
Department of Civil Engineering
The University of Texas at Austin, Austin, TX
[2]Research Assistant, The University of Texas at Austin, Austin, TX

Research Objectives

Two bridge construction projects in Texas are among a small number of current projects in the United States which have been designated by the Federal Highway Administration (FHWA) as demonstration projects for the use of high performance concrete (HPC). HPC may be defined in simple terms as concrete with beneficial strength and durability properties. The first Texas project, the Louetta Road Overpass on S.H. 249 in Houston, has three spans of up to 137 ft. in length. The recently developed U54 Beam, a 54-inch deep open-top U-shaped section was used in the Louetta bridge. A partial bridge cross-section containing a U54 beam and prestressed concrete deck panels is shown in Figure 1. The second Texas project, the North Concho River/U.S. 87/S.O.R.R. Overpass on U.S. 67 in San Angelo, utilizes AASHTO Type IV beams spanning up to 157 ft. Design concrete compressive strengths for both bridges are listed in Table 1. Several beams in each project use 0.6-in. prestressing strand, which is currently disallowed by FHWA in pretensioned applications. The larger strand allows for higher prestressing forces to be developed, thus taking advantage of the higher concrete strengths. In addition, the increased durability aspects of HPC are utilized in the bridge decks of each project. Research on both projects involves the development of mix designs and measurement of material properties for HPC bridge components, laboratory testing to investigate the transfer and development of 0.6-in strand in HPC, and the monitoring of bridge components for long-term performance. The overall objective of the research program is to develop design and construction practices for HPC.

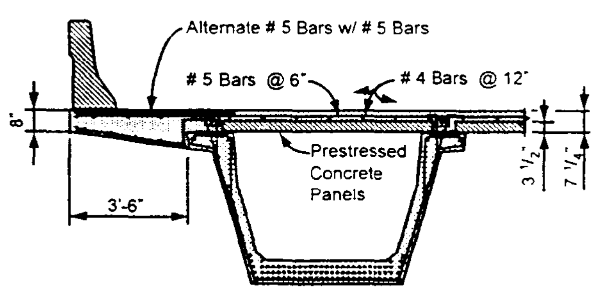

Figure 1: Partial Span Cross Section with U54 Beam

Table 1: Design Concrete Strengths (psi)

Bridge Component	Louetta	San Angelo
Beams	9,800 - 13,100	10,900 - 14,700
Substructure	10,000	6,000 - 8,000
Deck	4,000 or 8,000	6,000

Research Approach

HPC Mix Design and Material Properties: Mix designs have been developed for the HPC components of both bridges. Material properties have been investigated on laboratory trial batches for each mix, and an extensive quality control/quality assurance (QC/QA) program has been implemented to measure material properties on actual field batches. Measured fresh concrete properties include slump, air content, and unit weight. Measured hardened concrete properties include compressive strength, modulus of elasticity, split-tensile strength, and chloride-ion permeability.

In addition, the long-term creep and shrinkage properties of the HPC beam mixes were measured. These measured properties affect prestress losses and will be utilized in prediction models and data reduction for the long-term performance monitoring.

Transfer and Development Length: Transfer and development lengths tests were required for special FHWA approval of the HPC beam designs which used 0.6-inch strands at 2 in. spacing on-center. As previously noted, the use of pretensioned 0.6-inch strand has been disallowed since a 1988 FHWA memorandum. Transfer and development lengths of fully-bonded 0.6-in. strands were measured on specimens with two different cross-sections. The first specimen type, named the Hoblitzell-Buckner beams in honor of the designers, was a 14 in. wide by 42 in. deep rectangular beam with six 0.6-in. strands spaced at 2 in. in one row 2 in. from the bottom of the cross-section. The second specimen type was an I-shaped Texas Type C cross-section (similar to an AASHTO Type III) with a normal-strength composite deck. The Type C beams contained 16 0.6-in. strands in the bottom flange and 4 0.6-in. strands in the top flange. In addition, transfer lengths were measured on an actual 54-foot U-beam containing 67 0.6-in. strands.

Transfer length was determined at release of prestress by measuring the change in concrete strain at the center of gravity of the prestressing strands. Mechanical strain gauge points were epoxied to the concrete surface at 2-in. spacing and a reading was recorded immediately before and after release of prestress. All transfer length measurements were performed at the prestressing plant.

Development length tests were performed in the laboratory by testing to failure each end of the two rectangular beams (4 tests) and the four composite Texas Type C beams (8 tests). The embedment length, which is the distance from the end of the beam to the point of maximum moment, was decreased for each successive test, until the failure mode switched from flexural to bond or shear, or until the depth-to-shear span ratio was about 1.5. Load, deflection, concrete strain, and end slip were measured and cracking patterns and mode of failure were noted for each test.

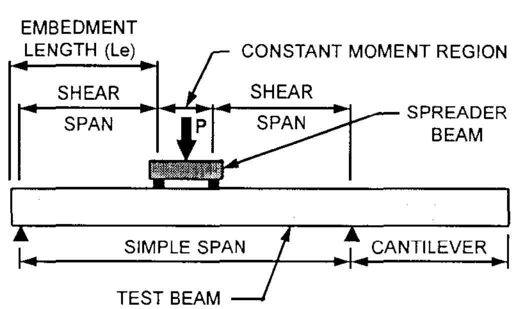

Figure 2: Development Length Test Setup

Field Instrumentation for Monitoring of Long-Term Performance: Selected components of each bridge are being instrumented to monitor long-term performance. Twelve HPC U54 beams, segments of the cast-in-place deck and precast deck panels, and a few precast pier segments will be instrumented in the Louetta bridge. Ten HPC AASHTO Type IV beams, four normal strength AASHTO Type IV beams, and segments of the cast-in-place deck and precast deck panels will be instrumented in the San Angelo bridge. Measurements will be taken at several stages to observe both immediate and time-dependent effects at different stages of the construction process.

Types of measurements include deflections (cambers), strains, and temperatures. Deflections are measured using a tensioned-wire system, in which a scale is fixed to the beam at midspan and moves relative to a tensioned wire fixed to the beam ends. Strains are measured using embedded gages (vibrating wire gages and bonded electrical resistance strain gages) and surface gages (mechanical strain gages). Strain measurements are used to determine the effects of prestress forces and applied loads, and to determine prestress losses. Temperatures are measured using embedded thermocouples, and are used to monitor hydration temperatures and thermal gradients. Temperature measurements are also used to correct deflection and strain measurements for thermal effects.

Expected Products

Two HPC bridges are being constructed in Texas as part of these projects. It is expected that the successful use of HPC in bridges will therefore be demonstrated. Ultimately, the cost effectiveness of using HPC for bridges is expected to be shown, as both construction and life-cycle costs are considered.

HPC mix designs using local materials will be produced, and the material properties of those mixes documented. Guidelines for mix proportioning will be developed. Current expressions for estimating material properties (including creep, shrinkage, modulus of elasticity, and split-tensile strength) will be examined and changes will be suggested as necessary.

Results of the long-term performance monitoring will be used to evaluate current design procedures. This includes estimations of prestress loss, camber, and thermal gradients. Where necessary, modified procedures and/or expressions will be suggested for design with HPC.

Preliminary Results

The HPC mix used for the Louetta beams has consistently produced concrete with compressive strengths well in excess of 8000 psi at 1 day and 13000 psi at 56 days. The modulus of elasticity has been measured at approximately 6 million psi at 1 day and 7 million psi at 56 days. HPC developed using local Texas aggregates has shown less ultimate creep and shrinkage than typical normal strength concrete and less than predicted by ACI 209. Creep was 20% lower when specimens were loaded at 28 days than when loaded at 1 or 2 days.

Measured transfer lengths of 0.6-in. diameter strand in HPC ranged from 30 to 40 strand diameters, which is less than the 50 strand diameters specified by the AASHTO and ACI 318 codes. The development lengths were observed to be less than 78 inches, which is significantly less than the calculated values based on the AASHTO and ACI codes.

Preliminary results of field instrumentation indicate that both prestress losses and camber are significantly lower than standard predictions indicate. Very high hydration temperatures have also been observed. Since beams are still being fabricated and jobsite construction ongoing, only a limited amount of field data is available at this time.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS RESEARCH ABSTRACT

Title: Field Measurement and Evaluation of Time-Dependent Losses in Prestressed Concrete Bridges

Author(s) and Affiliation(s):

Okey Onyemelukwe and Chris Mills, University of Central Florida, Orlando, Florida
Moussa Issa, Florida Department of Transportation
Structures Laboratory, Tallahassee, Florida

Principal Investigator: Okey Onyemelukwe

Sponsor(s): Florida Department of Transportation

Research Start Date: August 1995
Expected Completion Date: August 1998

Research Objectives:

The first phase of the project, involving bridge instrumentation and data collection, was initiated in August 1995. The primary objective is to determine the time-dependent losses in prestressed concrete bridges. The Westbound Gandy Bridge in Tampa, Florida, currently under construction, is instrumented to measure prestress losses. Typical approach span units of the bridge were originally designed as 4-span continuous post-tensioned girders of 144 ft. However, the contractor's value engineering study called for a modified design consisting primarily of 3 simply supported spans with pretensioned girders of 144 ft. The girder used is a modified AASHTO Type VI girder, which is 6 inches deeper than the Florida Bulb-T providing 18% additional area. The girder has a 44% greater moment of inertia and 64% greater ultimate capacity, thus allowing the change from 4-span continuous to 3 simply supported pretensioned spans.

Expected Products or Deliverables:

Design charts and tables accounting for the effect of time-dependent losses in design.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Okey Onyemelukwe received the B.Eng. in Civil Engineering from the University of Nigeria in 1985. He worked for one year for the Shell Petroleum Development Company. In 1988 he obtained the his MS degree from the University of Pittsburgh, and a Ph.D. in structural engineering from the same institution in 1993. Since 1993, he has worked as Assistant Professor of Structural Engineering at the University of Central Florida. His research interests include bridge engineering, computational wind engineering, structural dynamics, bridge monitoring, computational methods in civil engineering, and artificial intelligence.

Moussa Issa received his BSCE, MS and Ph.D. from the University of Texas at Arlington. He is a Research Structural Engineer with the Florida Department of Transportation structures laboratory in Tallahassee, Florida, a position he has occupied since 1989. He is also an adjunct faculty at the Florida State University, College of Engineering since 1989. His research interests include experimental mechanics, bridge load rating and testing, prestressed concrete design, and finite-element modeling of structures.

Chris Mills is currently a graduate student in structures and foundation engineering at the University of Central Florida. He received his BS degree from the University of Central Florida, in civil engineering.

Principal Investigator's Telephone Number:	(407) 823-6154
Facsimile Number:	(407) 823-3315
E-mail Address:	uoo@genesis.engr.ucf.edu

FIELD MEASUREMENT AND EVALUATION OF TIME-DEPENDENT LOSSES IN PRESTRESSED CONCRETE BRIDGES

Okey Onyemelukwe[1], Moussa Issa[2], Chris Mills[3]
[1]CEE Department, University of Central Florida, Orlando Florida 32816
[2]FDOT Struct. Research Center, 2007 East Paul Dirac Drive, Tallahassee, Florida 32310
[3]CEE Department, University of Central Florida, Orlando Florida 32816

Research Objectives

The primary objective in this project is to determine the time-dependent losses in prestressed concrete bridges. The Westbound Gandy Bridge in Tampa, Florida, currently under construction, is instrumented with vibrating wire strain gauges to measure strain and temperature in the concrete. Typical approach span units of the bridge were originally designed as 4-span continuous post-tensioned girders of 144 ft. However, the contractor's Value Engineering study called for a modified design consisting primarily of 3-simply supported spans with pre-tensioned girders of 144 ft. In this modified bridge configuration, while the beams are simply supported and determinate, the composite deck slab is a 3-span continuous system, which is indeterminate and behaves differently. This hybrid structural configuration, based on comparative design calculation, has been shown to have the same structural capability as the continuous post-tensioned alternative. However, it is lacking in supporting performance field data. Especially, with respect to time-dependent effects on camber and deflection. Therefore, the immediate and long-term effects of creep, shrinkage, and steel relaxation are being investigated in this study and their inter-dependence examined

Research Approach

Background: This paper discusses the on going research project sponsored by the Florida Department of Transportation (FDOT). The first phase of the project, involving bridge instrumentation and data collection was initiated in August 1995. Continuous data collection will continue for at least 3 years. As for the bridge configuration, typical approach span units of the bridge were originally designed as 4-span continuous post-tensioned girders of 144 ft. However, the contractor's Value Engineering study called for a modified design consisting primarily of 3-simply supported spans with pre-tensioned girders of 144 ft. The girder used is a Modified AASHTO Type VI Girder , which is 6 inches deeper than the Florida Bulb-T providing 18% additional area. The girder has a 44% greater moment of inertia and 64% greater ultimate capacity, thus, allowing the change from 4-span continuous to 3-simply supported pre-tensioned spans.

Bridge Instrumentation: The final bridge configuration consists of units with simply supported pre-tensioned girders, supporting a 3-span continuous cast-in-place concrete bridge deck slab. To enable the investigation into the effects of the time-dependent pre-stress losses, four of the bridge girders are strategically chosen for instrumentation with embedded vibrating wire strain gauges. The strain gauges are installed during casting, and monitored through out the construction stages, and into part of the bridge service life. The bridge girder instrumentation which consisted of installing vibrating wire strain gauges, multiplexers, and a state-of-the-art data acquisition system complete with cellular phone and solar panel hook-up, was completed in December 1995. Also connected to the data acquisition system is a Campbell Scientific HMP35C temperature and relative humidity probe.

Three locations along the girder span were selected for placement of strain gauges across the girder cross-section from top to bottom. The three locations are the girder mid-point, quarter-point from the support, and in the end location, about five feet from the support. The mid-point was chosen because this is were the maximum moment will occur. In choosing the five feet from the support , care was taken to ensure it was beyond the development length of the prestressing steel. With data from these gauges, information about the time-varying prestress losses near the supports can be determined. And the quarter-point was chosen to account for the effects in the locations between these two points.

In each instrumented girder cross-section, strain gauges were placed at both the non-composite beam and composite beam centroids of concrete area. It was also necessary to locate strain gauges at the centroid of the steel areas. Other intermediate points were chosen to determine if there is any trend or important effect in the regions between the centroidal locations as well. More gauges are contained within the midspan than at the quarter and five feet points. This is because the midpoint is where the maximum moment, deflection and camber occur. A total of four beams are being investigated. The four chosen, were representative of the twelve beams used in a typical unit of the bridge. They comprise of two exterior and two interior girders of the bridge located in adjacent spans.

Also instrumented with strain gauges are three 12 inch concrete cylinders to be used for shrinkage determination. The cylinders are cured same as the girders, and are placed along side the girders during its service life. Several other concrete cylinders were cast, and are used to establish the time dependent Modulus of Elasticity of the concrete and its actual compressive strength, at important events during the construction stages and into the girders service life.

At the time the bridge deck will be poured, vibrating wire strain gauges will also be installed at the maximum negative moment and positive moment regions, and other locations to be identified later, in order to capture the effect of the deck continuity on the time-dependent losses if any.

At present, data is being collected as the girders await placement on the bridge. The data collected is expected to help FDOT better understand the behavior of this hybrid structural bridge configuration, especially in the complex area of time-dependent prestress losses. Detailed comparison of the data collected against those from existing methods for computing time-dependent prestress losses will be investigated, and the results discussed as part of the project deliverable.

Expected Products

In addition to analyzing the strain measurements from the bridge girders to determine the lump sum prestress losses, the breakdown of the contributions from elastic shortening, creep, shrinkage and steel relaxation, and temperature will be deduced from data. These time-dependent prestress losses from actual measurement will be compared with the prestress loss estimates determined in accordance with several design codes. Inclusive in the design codes to be compared with are the ACI and PCI approach, AASHTO and New AASHTO-LRFD approach, CEB-FIP approach, and any other procedures as used in Canada, various European countries and Australia. The comparison will also encompass prestress losses computed using time-dependent analytical methods. Extensive use of spreadsheet computations and available computer software will be employed for these studies. The effects of the prestress losses on the camber and deflection of the girder will be investigated vis-à-vis design estimated time of the various construction stages, and the actual construction and member placement times.

The effects of these time-dependent prestress losses on the design procedures currently being used for prestressed concrete bridges will be investigated as well. As stated in the background to this research, a hybrid structural configuration is in place for the instrumented bridge. In addition to establishing its performance in this study, comparisons will be made to response of a statically determinate and statically indeterminate form of the bridge structure.

If any significant effects are determined in this study, recommendations as to its consequences on current practice will be made to FDOT and the bridge engineering research world in general.

Preliminary Results

The measured data obtained from the day the concrete was cast up to 14 days after, is currently being processed and some of the results have been reported in Figures 1 and 2. The plots show the distribution of total strain (Micro-strain) in one of the instrumented girders, taken at mid span. The bridge design called for initial concrete strength at prestress transfer of 5000 psi and 28th day compressive strength of 6500 psi. According to the design, the estimated time for strand release was about 48 hours, however, these girders were casted in the winter time (December 1995: which this year was uncharacteristically cold in Florida, with ambient temperatures of 50 - 60 degrees Fahrenheit and relative humidity 50-80), hence a release time of 4 days later.

Figure 1 shows the variation of strain with time at the top of the girder with 3 inches cover (i.e. 75 inches from the bottom), at the centroid of the girder concrete, 39 inches from the bottom, an arbitrary chosen location 27 inches from the bottom, and at 3 inches from the bottom. The strain distribution shown in the plot are normalized with respect to the strain immediately after the concrete is casted. As expected, due to high temperatures (up to 98 degrees Fahrenheit) within the concrete from heat of hydration, the concrete strains indicate expansion initially and compression after the strands are released. Higher compressive strains exist in the bottom of the beam when compared with the top, and the compressive strain decreases along the girder cross-section from top to bottom.

Figure 2 on the other hand shows the variation of the actual strain along the cross-section or depth of the girder at mid span, for Day 0 and 11 hours after beam was casted, Day 4 just before the strands were released, Day 4 after the strands were cut, and Day 14 since the girder was casted. In this plot the amount of strain at each time of interest is clearly depicted. Again, it can be seen that more compressive strain exist in the bottom of the girder than at the top.

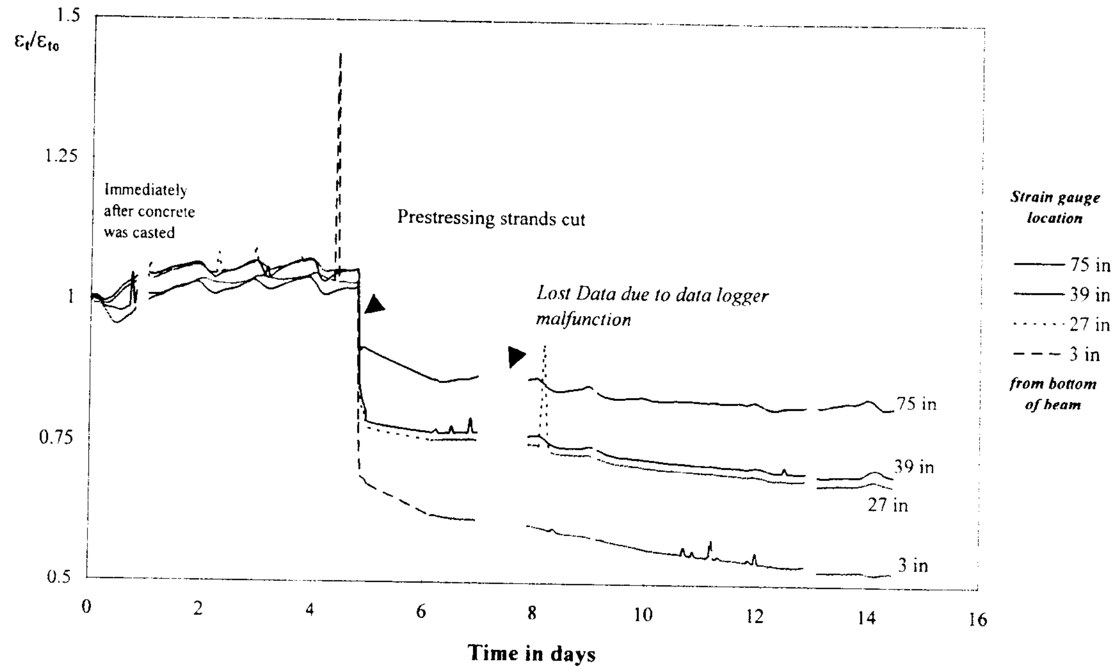

Figure 1. Normalized Strain at Midspan of Girder as a function of Time

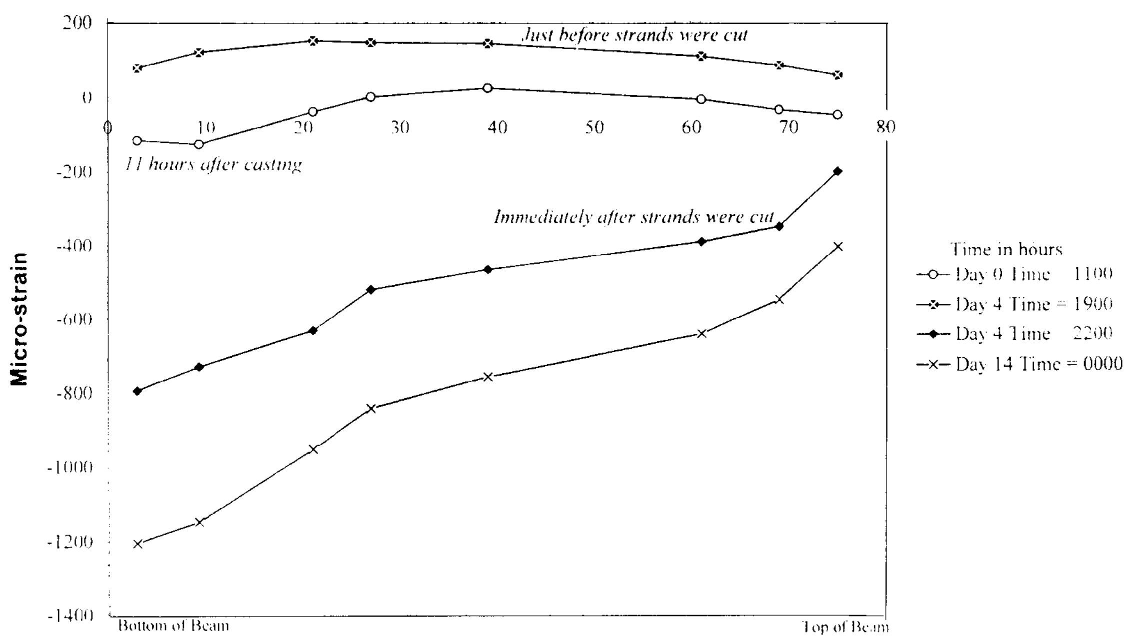

Figure 2. Cross-sectional Strain Variation at Midspan W.r.t. Time

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Field Monitoring of Prestress Forces in Four Box Girder Bridges Subjected to High Variation of Humidity

Author(s) and Affiliation(s):

M. Saiid Saiidi and Nathaniel Mangoba
Civil Engineering Department
University of Nevada, Reno, Reno, NV

Principal Investigator: M. Saddi Saiidi

Sponsor(s): Nevada Department of Transportation

Research Start Date: June 1, 1994
Expected Completion Date: August 31, 1997

Research Objectives:

The dry and desert-like climate in Northwest Nevada is typically associated with high daily variation of temperature and relative humidity (RH), with the ratio of high to low values within any 24-hour period ranging from 1.5 to 3. The concern for possible consequence of the high daily range of temperature and RH on creep and shrinkage losses led to a pilot field study completed in 1994 of a post-tensioned bridge which indicated that the actual losses may be considerably higher than those predicted by AASHTO specifications. Four new post-tensioned, box-girder bridges in Northern Nevada were instrumented during construction to collect more data on the possible adverse effects of high variation in RH and temperature on prestress losses. Hence, the objective of the current study is to determine how the actual losses compare with those predicted by AASHTO and other methods, and to determine if a modification of the specifications is warranted.

Expected Products or Deliverables:

The measured variation of prestress forces over a two-year period will indicate the total time-dependent loss in each bridge. A comparison of the measured data with those determined based on the AASHTO method will establish the applicability of the results, and will determine if any modification is necessary. The comparison of shrinkage data for specimens subjected to constant RH and those subjected to variable RH will provide information on the sensitivity of the final shrinkage strain to variable RH, which can also be factored in any modifications that may be proposed.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

M. Saiid Saiidi is Professor of Civil Engineering at the University of Nevada, Reno. Dr. Saiidi received his Ph.D. from the University of Illinois at Urbana-Champaign in 1979. He is active in research on different aspects of reinforced concrete bridge response including laboratory testing of bridge components subjected to seismic and non-seismic loads, analytical studies of the earthquake response of bridges, and field testing of bridges. He is the founding chairman of ACI Committee 341, Earthquake-Resistant Concrete Bridges, and is active in a number of other ACI and Council on Tall Buildings and Urban Habitat committees. Dr. Saiidi is a registered professional engineer in California and Nevada.

Nathaniel Mangoba is an MSCE candidate at the Civil Engineering Department, University of Nevada, Reno. Mr. Mangoba completed his BSCE degree in December 1995 at the University of Nevada, Reno. The subject of his thesis is field monitoring of prestress losses in actual bridges.

Principal Investigator's Telephone Number: (702) 784-4839
Facsimile Number: (702) 784-4466
E-mail Address: saiidi@scs.unr.edu

FIELD MONITORING OF PRESTRESS FORCES IN FOUR BOX GIRDER BRIDGES SUBJECTED TO HIGH VARIATION OF HUMIDITY

M. Saiidi[1] and N. Mangoba[2]

[1]Professor, Civil Engineering Department, University of Nevada, Reno, NV 89557
[2]Graduate Research Assistant, University of Nevada, Reno, NV 89557

Research Objectives

The dry and desert-like climate in Northwest Nevada is typically associated with high daily variation of temperature and relative humidity (RH), with the ratio of high to low values within any 24-hour period ranging from 1.5 to 3. The concern for possible consequence of the high daily range of temperature and RH on creep and shrinkage losses led to a pilot field study completed in 1994 of a post-tensioned bridge which indicated that the actual losses may be considerably higher than those predicted by the Association of State Highway and Transportation Officials (AASHTO) Specifications [1]. Four new post-tensioned, box-girder bridges in Northern Nevada were instrumented during construction to collect more data on the possible adverse effects of high variation in RH and temperature on prestress losses. Hence, the objective of the current study is to determine how the actual losses compare with those predicted by AASHTO and other methods, and to determine if a modification of the specifications is warranted for the region.

Research Approach

Introduction: The daily ambient RH in Nevada varies considerably compared to other states. This is especially true in Northern Nevada where RH can change from 30% to 70% in a single day. Because current design guidelines are based on a constant RH, a study was undertaken to determine the actual prestress losses in several bridges in Northern Nevada. Four post-tensioned, multicell, grouted, box girder bridges were included in the study and were instrumented during construction (Table 1). Each bridge will be monitored for a period of 24 months. The measured data are being compared to two prestress loss prediction models: the AASHTO specifications and Naaman's time-step method [2]. In addition to the study of the bridges, ten concrete cylinders are being used to compare the effect of variable humidity on concrete shrinkage. Five cylinders are placed in an environmentally controlled room with a nominally constant RH of 50 percent, while five other cylinders are being placed alternately in rooms with RH of 25 percent and 75 percent, each for a period of 24 hours.

Experimental data: Four types of data have been gathered in the course of the study. They include strains on the surface of the girders near mid span; deflection; and creep and shrinkage of cylindrical concrete specimens. Field measurements were taken more frequently during the first few weeks after the structures were stressed because majority of the prestress losses occur during this stage. Measurement schedule has been one day, three days, seven days, fourteen days, one month, two months, and three months after the initial prestressing date for each bridge, followed by a bimonthly measurement schedule.

The strain readings were taken from the mechanical strain gages attached on the concrete surface inside the box cell. These gages were placed at the same level of the prestressing tendons, and they all had a 30-in. gage length. For the two-span bridges, eight gages were used: four gages attached on the exterior girder and the other four on the interior girder. The gages were placed near the locations of maximum positive and negative moments under bridge dead load. In each of the single-span bridges only four gages were installed, two on each girder (interior and exterior). The gages in the single-span bridges were attached near the mid span. The collection of the strain data began prior to stressing of each bridge. A reference point was established immediately after prestressing was completed. Subsequent data indicated the amount of strain change in the bridges based on which the magnitude of prestress loss was obtained. Three types of time-dependent prestress losses occur in prestressed bridges: creep, shrinkage, and relaxation losses. Because relaxation losses are not associated with strain changes, the data collected in this study show only the combined creep and shrinkage losses.

In addition to prestress changes, creep and shrinkage strains have been measured on 6 in. by 12 in. concrete cylinders which were obtained from the concrete placed in each bridge. The cylinders are kept inside the box girders. Three cylinders are used to collect shrinkage data and two are stressed to a constant stress level representing the prestress effect in each bridge. The deflection in single-span bridges has also been measured at mid span. In the other two bridges, the deflection is measured near the point of maximum positive moment under the dead load. Surveying equipment have been used to measure the deflection.

Theoretical Analyses: Two methods were used to calculate the prestress losses: the AASHTO lump-sum estimate of separate losses [1] and the time-step method as presented by Naaman [2]. All the four bridges are designed based on AASHTO specifications. An ambient RH of 50 percent was used in all calculations.

The time-step method presented in [Naaman 1982] was used to calculate the time-dependent prestress loss of the bridges. This method takes into account the interaction among different loss components. A computer program was developed to implement the method, details of which are presented in [3]. This program calculates all the time-dependent losses (creep and shrinkage of concrete and relaxation of steel) and can take into account the variation in relative humidity.

Expected Products

The measured variation of prestress forces over a two-year period will indicate the total time-dependent loss in each bridge. A comparison of the measured data with those determined based on the AASHTO method will establish the applicability of the AASHTO results to the bridges in the region, and will determine if any modification is necessary. Because the primary sources of any potential discrepancies between the design and the actual losses are creep and shrinkage, the direct measured creep and shrinkage strains on the cylindrical specimens placed inside the bridges will allow the comparison of these data with what is implied in AASHTO and provide a possible guidance on the parameters which need to be modified. The comparison of shrinkage data for specimens subjected to constant RH and those subjected to variable RH will provide information on the sensitivity of the final shrinkage strain to variable RH, which can also be factored in any modifications that may be proposed as a result of this study.

Preliminary Results

Figures 1 to 4 show the measured and calculated prestress histories in all the bridges thus far. The data indicate the variation of prestress due to time and changes in temperature and RH. Typically significant prestress losses have been occurring during Summer months due to the high temperature and low RH. In Winter months the data indicate a gain in prestress, due to the absorption of ambient moisture by the bridges and the increase in the volume.

Table 2 shows the shrinkage and creep losses calculated for each bridge using the AASHTO and the time-step methods after 40 years. It can be seen that the time-step results are generally lower than the AASHTO losses.

The combined life-time creep and shrinkage losses for the four bridges are presented in Table 3. The measured values shown in the table are extrapolated from the measured data thus far using a regression analysis with logarithmic fit. It can be seen that there is close correlation between the AASHTO and measured values for the two multi-span bridges. The measured losses in the Old Virginia Rd. bridge, on the other hand, are considerably higher than the AASHTO and time-step values. Also, in all bridges, the time-step results are lower than the measured values. Future data may change these trends. However, the data thus far indicate that the AASHTO code underestimates the losses considerably in at least one single-span bridge. Other measured data are not shown due to space limitation for this paper.

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1. American Association of State Highway Transportation Officials (AASHTO), Standard Specifications for Highway Bridges, Washington, D.C., 1992.

2. Naaman, A. E. Prestressed Concrete Analysis and Design, McGraw-Hill Book Company, New York, 1982.

3. Saiidi, M. and Hutchens, E., "A Study of Prestress Changes in a Post-Tensioned Bridge during the first 30 months", Civil Engineering Department, Report No. CCEER-92-3, University of Nevada, Reno, April 1992.

Table 1 - General Information about Bridges

Bridge No. & descripton	No. of Span	Span Length	Width	Date of Prestressing
I-1952, S. Meadows Int.	2	161' & 192'	60'	9/28/94
I-1949, Mt. Rose Int.	2	153' & 123'	83'	11/21/94
I-1951, Zolezzi Ln. GS	1	215'	145'	4/28/95
I-2007, Old Virginia Rd. GS	1	102'	55'	5/22/95

Table 2 - Calculated Creep and Shrinkage Losses

METHOD	ZOLEZZI	OLD VIRGINIA	S. MEADOWS M(-)	S. MEADOWS M(+)	S. MEADOWS M(+) & M(-)	MT. ROSE M(-)	MT. ROSE M(+)	MT. ROSE M(-) & M(+)
AASHTO								
CREEP (KSI)	16.06	12.97	7.96	12.4	10.18	9.03	10.1	9.57
SHRINKAGE (KSI)	7.6	7.6	7.6	7.6	7.6	7.6	7.6	7.6
TIME-STEP								
CREEP (KSI)	13.09	14.35	6.41	10.1	8.28	7.21	7.74	7.48
SHRINKAGE (KSI)	4.67	6.53	5.51	5.51	5.51	3.91	3.91	3.91

Table 3 - Lifetime Prestress Loss Comparison

CREEP & SHRINKAGE LOSSES (KSI)	ZOLEZZI	OLD VIRGINIA	S. MEADOWS M(-)	S. MEADOWS M(+)	S. MEADOWS M(+) & M(-)	MT. ROSE M(-)	MT. ROSE M(+)	MT. ROSE M(-) & M(+)
AASHTO	23.66	20.57	15.6	20	17.78	16.6	17.7	17.17
TIME-STEP	17.76	20.88	11.9	15.6	13.79	11.1	11.65	11.39
MEASURED (EXTRAPOLATED)	19.21	28.85	14.2	15.3	14.84	13.2	23.65	18.68

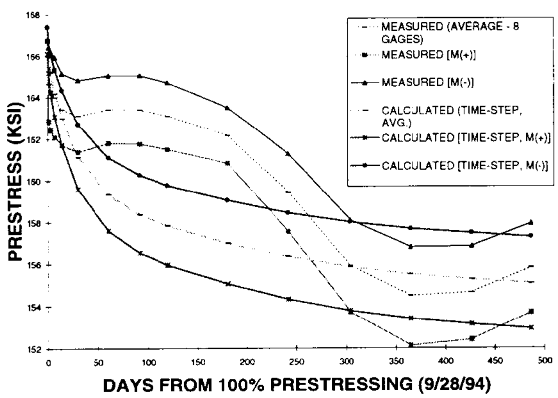

Fig. 1 - Prestress Variation in S. Meadows Bridge

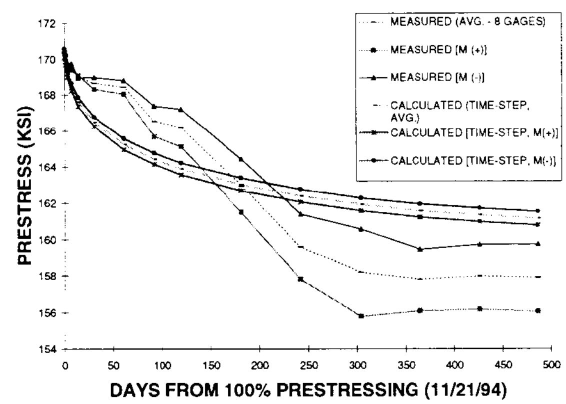

Fig. 2 - Prestress Variation in Mt. Rose Bridge

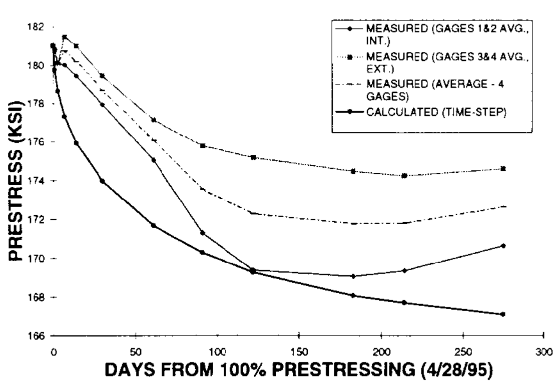

Fig. 3 - Prestress Variation in Zolezzi Ln. Bridge

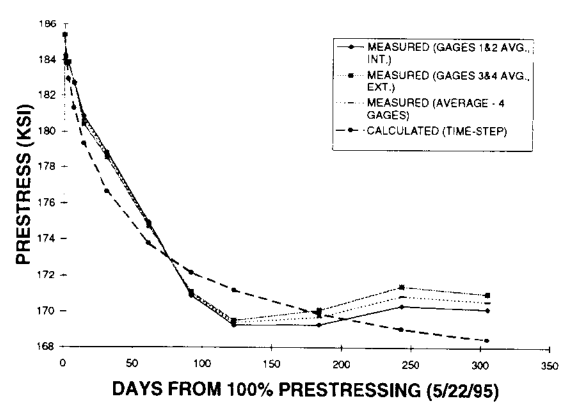

Fig. 4 - Prestress Variation in Old Virginia Bridge

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Load Rating of Masonry Arch Bridges and Culverts

Author(s) and Affiliation(s):

Thomas E. Boothby, The Pennsylvania State University, University Park, PA
Vikram K. Dalal, Ohio Department of Transportation, Columbus, OH

Principal Investigator: Thomas E. Boothby

Sponsor(s): Ohio Department of Transportation

Research Start Date: February 1, 1994
Expected Completion Date: June 1, 1996

Research Objectives:

Although the masonry arch has been used as a structural element in road bridges for centuries, today's heavier trucks and busier roads may for the last time be testing the limits of these structures. Moreover, many masonry arch bridges are listed in or eligible for listing in the National Register of Historic Places, which triggers Federal mandates for their preservation and protection. The objective of the present research is to develop an experimentally verified method for determining accurate load ratings for masonry arch bridges and culverts.

Expected Products or Deliverables:

The load rating procedure will rely on frame analysis of a unit width of the structure, with some additional checks for transverse effects. The procedure will be based on work done in the United Kingdom, which has been incorporated into a Standard and an Advice Note. The uncertainty of masonry structures, and the use of a limit states procedure, is reflected in an overload factor of greater than 3 applied to masonry arch bridges.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Thomas E. Boothby is Assistant Professor of Architectural Engineering at The Pennsylvania State University, a position he has held for the last four years. Previously, he was a post-doctoral researcher at the University of Nebraska-Lincoln, and a structural designer for various consultants in Albuquerque, NM. He has earned a Ph.D. in civil engineering from the University of Washington, an MSCE from Washington University in St. Louis and a BA in Architecture from Washington University. His research specialty is assessment, maintenance, repair, and rehabilitation of old and historic bridges and other structures.

Vikram K. Dalal is a Research Engineer with the Ohio Department of Transportation. He has been active in many significant bridge research projects over the last ten years.

Principal Investigator's Telephone Number: (814) 863-2082
Facsimile Number: (814) 863-4789
E-mail Address: tebarc@engr.psu.edu

LOAD RATING OF MASONRY ARCH BRIDGES AND CULVERTS

Thomas E. Boothby[1] and Vikram K. Dalal[2]

[1]Department of Architectural Engineering
The Pennsylvania State University, University Park, PA 16802
[2]Bureau of Research and Development
Ohio Department of Transportation, Columbus, OH 43216-0899

Research Objectives

Although the masonry arch has been used as a structural element in road bridges for centuries, today's heavier trucks and busier roads may for the first time be testing the limits of the world's masonry arch bridges. Moreover, many of these structures are listed in or eligible for listing in the National Register of Historic Places, which triggers Federal mandates for their preservation and protection. The objective of the present research is to develop an experimentally verified method for determining accurate load ratings for masonry arch bridges and culverts.

Research Approach

In this study, a simple, accurate, and portable method of collecting a comprehensive set of arch ring displacements was established. Full-scale quasi-static load tests were then conducted on eight masonry arch bridges in Ohio and Pennsylvania. With the data collected, an analytical procedure to predict arch ring displacements was created using finite element software. A parametric study assisted in the understanding of how various input quantities affected the displacement output. Displacement data from previous experiments conducted in the United Kingdom on similar masonry arch bridges were compared to the FEM model's output as well. The finite element model also determines thrusts and moments in the joints for a given loading condition, which enables predictions regarding weaknesses in the structure and potential failure modes. Such data will eventually lead to a method of determining the load-carrying capacity of a given bridge.

Bridges Under Study

Field tests were conducted on five masonry arch bridges. Table 1 lists the dimensions of each structure. The first bridge tested is located in Chester County, Pennsylvania, the next three are in Lorain County, Ohio, and the final bridge is in Mt. Vernon, Ohio. The Chester County bridge is a small masonry arch bridge built in 1916. It is roughly semi-circular in shape, the only bridge in the program with this geometry. It is also unique in that the arch ring is constructed with random stone voussoirs, varying in depth from 0.28 to 0.64 meters. The mortar joints vary in thickness from 6 to 20 mm. Spandrel walls, 15 m long, are located on each side of the arch barrel. The bridge is in overall poor condition, with numerous cracks present in the arch ring and in the spandrels. Three similar bridges, all in or near Oberlin, Ohio in Lorain County, were tested next.

Two of the structures are located in Wellington Township, on Cemetery Road and Jones Road and the third is in Oberlin itself. All three are segmental single-span bridges, and are made up of cut-stone voussoirs. The mortar joints are thin, with a maximum thickness of no more than 6 mm. The fifth bridge tested is an 1892 structure located in Mt. Vernon, Ohio. One end span of the five-span structure was instrumented for the test. It is segmental with 31 regular cut stones making up the arch ring.

Characteristic	Chester	Cemetery	Jones	Oberlin	Vernon
Span (m)	2.59	7.16	6.89	6.10	11.59
Height (m)	1.30	3.05	2.62	2.59	2.74
Span/Height	2	2.35	2.63	2.35	4.22
Thickness (cm)	28-62	45	45	40	60
Roadway Width (m)	6.55	6.10	4.88	6.96	15.24
Arch Barrel Width (m)	7.32	7.62	6.73	8.79	19.51
Fill Over Crown (cm)	37.5	30	30	30	30
Angle of Embrace	180°	162°	149°	161°	102°
No. of Voussoirs	21	23	23	23	31

Table 1: Summary of Bridges in the Testing Program

Three additional bridges have been included in the testing program: a small, rustic bridge with two 4.5 m spans, a single span structure with a 9m span and a 19m width and a monumental two span structure with a 15.2m spans.

Testing Procedure

A portable high-speed data acquisition system was used to conduct full-scale service load tests on the bridges. The system consists of a portable computer with a data acquisition card installed, a series of linear variable differential transducers (LVDT's), and signal conditioners for each of the LVDT's. The system allows for high-speed acquisition of displacement data throughout the test. At each bridge, a series of quasi-static runs were made using loaded trucks. Each particular test was performed a total of three times. For each test a loaded truck was placed so that the computer would begin collecting displacement data after the truck had moved approximately five feet, at a point where the first axle of the truck had not yet passed above the springing of the arch. As the truck began to move, the data acquisition system was triggered by the truck, and started collecting voltage data from LVDT's applied to the intrados of the bridge. For all runs, data were collected at a rate of 100 scans per second, and a total of 4000 scans were made per channel. The test data were filtered digitally after acquisition with a low pass filter with a cut-off frequency of 5 Hz and a roll-off frequency of 0.1 Hz.

Expected Products

The load rating procedure will rely on frame analysis of a unit width of the structure, with some additional checks for transverse effects. The procedure will be based on work done in the UK, which has been incorporated into a Standard and an Advice Note. (Department of Transport

1993, 1993a) The uncertainty of masonry structure, and the use of a limit states procedure is reflected in an overload factor of greater than 3 applied to masonry arch bridges.

Preliminary Results

The displacement data collected in the field tests provide considerable information on the behavior of masonry arch bridges under quasi-static loads. Table 2 gives a synopsis of the maximum displacement results for the Lorain County bridges for the fully loaded truck crossing the entire structure

Since two trucks of different axle weights were used for each bridge except the Mt. Vernon bridge, the relationship between deflections and loads can be used to determine the linearity of the response. If the load-deflection curve begins to exhibit nonlinearities, the development of damage in the arch ring is likely. While this is not necessarily an indication of impending failure, it is a sign that a significant portion, possibly one-quarter to one-third, of the bridge's capacity is being approached, as suggested by Hendry et al.(1985). To examine the linearity of responses, Table 3 relates the increase in axle loads to the average increase in maximum crown displacements for the tests at each bridge. In the Chester County bridge, the crown deflections increased at a slightly greater rate than the axle load between the half loaded and fully loaded trucks. It appears that the fully loaded truck is inducing a small nonlinear response in the bridge. The responses of the three similar bridges in Lorain County, Ohio do not indicate nonlinear behavior. The ratio of deflection increases was slightly less than the ratio of axle weight increases. This is primarily because of the different axle configurations between the half-loaded truck. All half loaded trucks had single axles, while the fully loaded trucks in Lorain County had tandem rear axles, with a larger spacing. This wider distribution of loads in the fully loaded trucks reduced the magnitudes of the crown displacements

The similar size and shape of the three Lorain County bridges allow observations to be made about the range of deflection responses. All three bridges were tested using the same two trucks. The maximum displacements given in Table 2 for these bridges show a general degree of consistency among all three structures. For comparison purposes, the maximum crown displacement for the half loaded truck, left edge of roadway, half span test at the Cemetery Road bridge will be used as a baseline. This value was 0.396 mm. The same test at the Jones Road bridge resulted in a maximum displacement of 0.318 mm, a 20 percent decrease from the Cemetery Road bridge test. At the Oberlin bridge, the equivalent test caused a maximum crown displacement of 0.470 mm, a 19 percent increase. For the fully loaded truck test at the Cemetery Road bridge, the maximum crown displacement was 0.630 mm. At Jones and Oberlin the equivalent displacements were 0.381 mm and 0.612 mm, respectively. These were 40 percent and 3 percent decreases, respectively. The relative stiffness of the Jones bridge can be attributed to the greater span/rise ratio.

From the above percentages, a general range of displacements can be seen for a given bridge type and loading pattern. For the three similar bridges in Lorain County, variations of up to 40 percent are seen. Additional tests on similar bridges could be used to further predict the expected range of responses. Also, knowledge of specific factors such as haunching, fill properties, and mortar thickness could help in predicting the variation in responses.

Bridge	Maximum Displ. (mm)			Final Displ. (mm)		
Test	Near Abut	Crown	Far Abut	Near Abut	Crown	Far Abut
Cemetery						
FLF	-76	622	-56	-3	3	-5
FRF	-46	224	-23	3	3	-3
Jones						
FF	-109	368	-84	-5	3	-3
Oberlin						
FCF	-3	612	-33	3	23	-3
FLF	-3	224	-25	0	5	-3

Table 2: Summary of results from Lorain County Bridges

	Half Loaded Truck Axle Weight (kg)	Fully Loaded Truck Axle Weight (kg)	Increase in Axle Load, percent	Avg. Increase in Max. Crown Displ., percent
Chester County	4680	11270	241	283
Cemetery Road	9590	15840	165	157
Jones Road	9590	15840	165	125
Oberlin	9590	15840	165	133

Table 3: Linearity of response of Chester County Bridge and Lorain County Bridges

Acknowledgments

The research described in this paper was sponsored by the Ohio Department of Transportation. The authors wish to particularly acknowledge Karen Young of Ohio DOT for assistance in the understanding of the history of masonry arch bridges in Ohio, and the bridge crews and truck drivers of Ohio DOT, for their enthusiastic help during the testing program described herein.

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FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Innovative Prestressed Steel Composite Short to Medium Span Bridges

Author(s) and Affiliation(s):

Dan Burroughs and James Lockwood, J. Muller International, Chicago, IL
Joseph Hartmann, American Iron and Steel Institute, Washington, D.C.

Principal Investigator: Dan Burroughs

Sponsor(s): American Iron and Steel Institute

Research Start Date: Fall 1995
Expected Completion Date: June 1996

Research Objectives:

This study reviews methods to combine both prestressing, high-performance steel and composite concrete decks to improve bridge performance, constructibility, aesthetics, and economy. Initially, a total of eight bridge concepts have been developed in the short to medium span range using innovative concepts of prestressing and high-strength steel. Three bridge concepts have been developed to the preliminary design phase including plan details and cost estimates. Single and two-span bridges with span lengths of between 100 to 180 feet have been studied. The alternates developed use prestressing both longitudinally in the main structural components and transversely in the deck.

Expected Products or Deliverables:

A final report with specific recommendations, bridge layouts, cross-sections, details, and cost estimates for three of the prestressed steel composite bridge alternates.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Dan Burroughs is Engineering Manager of the Chicago office of J. Muller International. Mr. Burroughs has extensive experience in the design, analysis, management, quality control, and construction of complex bridges. His experience includes both precast concrete segmental and cast-in-place concrete segmental bridges, prestressed concrete girder bridges, steel girder bridges, steel arch bridges, cable-stayed bridges, and bridge erection equipment. He has written numerous papers on bridge design and construction.

James Lockwood is Principal of the Chicago office of J. Muller International. Mr. Lockwood has been responsible for all aspects of the design of complex bridge structures during the past 10 years. He has comprehensive design experience in precast and cast-in-place segmental bridges, cable-stayed structures, and composite bridges including five years in Europe. He has provided technical assistance to contractors during the construction of segmental bridges. For over three years, Mr. Lockwood worked closely with Jean M. Muller, Technical Director of J. Muller International, in Paris, France in developing new concepts for composite, concrete and cable-stayed bridges. Mr. Lockwood has written numerous papers and delivered various presentations on bridge design and construction.

Joseph Hartmann is the Bridge Engineer for the American Iron and Steel Institute. Mr. Hartmann is currently responsible for implementing the bridge market development plan of the steel industry and advising the industry on all aspects of bridge design and research. He has design experience in straight and curved steel girder bridges and precast segmental concrete bridges. Mr. Hartmann also has materials research experience with bridge steels. He formerly worked for the Federal Highway Administration and T.Y. Lin International. He is currently a member of the AISI/AASHTO T-14 Steel Bridge Task Force and the ASCE Bridge Committee, and is a former member of the PCI Technical Bridge Committee. Mr. Hartmann has delivered numerous presentations on bridge design.

Principal Investigator's Telephone Number: (312) 670-2100
Facsimile Number: (312) 670-2101
E-mail Address:

INNOVATIVE PRESTRESSED STEEL COMPOSITE SHORT TO MEDIUM SPAN BRIDGES

Dan Burroughs, PE[1] and James Lockwood, PE[1] and Joseph Hartmann, PE[2]

[1]J. Muller International
400 N. Michigan Avenue, Ste. 1500, Chicago, IL 60611
[2]American Steel & Iron Institute
1101 17th Street, NW, Ste. 1300, Washington, DC 20036

Research Objectives

This research reviews methods to combine both prestressing, high-performance steel and composite concrete decks to improve bridge performance, constructibility, aesthetics and economy. For the first phase of the project, a total of eight bridge concepts were developed in the short to medium span range using innovative concepts of prestressing and high-strength steel. For the final phase, three bridge concepts have been developed to the preliminary design phase including plan details and cost estimates. Typical two-span overpass structures with span lengths from 100 to 120 feet and a single-span structure with a span of up to 180 feet have been studied. All alternates use high-performance GR 50 or 70 steel and LRFD design specifications. The alternates use prestressing both longitudinally and transversely in the deck. The prestressing is applied as pretensioning, post-tensioning, or a combination of both. The combination of steel and prestressed concrete is based on the incentive to use each respective material where it is traditionally found to be the most economical.

Research Approach

The approach to this project has several aspects. The first has been to develop innovative concepts based upon past experience with other projects or based upon an original approach. Mr. Jean Muller, Technical Director of J. Muller International, has been directly involved in this development. Industry input is a key factor in this project to ensure that all concepts are feasible and that they meet the requirements of the bridge community. Meetings with industry representatives including fabricators, prestressing suppliers, the FHWA, state DOT officials, and other designers has led to valuable input into this project. Design will meet the requirements of the AASHTO LRFD design specification. Analysis will be undertaken with FEM as required to optimize the preliminary designs. Future research may include testing of key components and possible demonstration projects with full scale testing.

Expected Products

Products from this research will include both a Phase I and Phase II report. The Phase I report developed eight innovative bridge concepts and discusses methods to improve bridge durability, maintainability, and inspectability. Also, the structural behavior of these alternates are reviewed with a discussion of the seismic behavior, fatigue performance, and redundancy of these types of structures. Bridge economy is reviewed and recommendations are made to improve economy in all aspects of fabrication, construction, materials, layout, and details. The Phase II report develops three concepts to the preliminary design stage with relevant plan details, discussion of concepts, and cost data.

Preliminary Results

Preliminary results indicate that it is possible to combine prestressing, high-strength steel and concrete into cost-effective short to medium span bridges. It is also possible to increase the long-term durability, reduce maintenance, and improve the structural performance for seismic loading, fatigue, and redundancy. At this point in the project, eight concepts have been developed as well as a Phase I report. These concepts have certain common characteristics. Thcy all use corrosion-resistant GR 50 or 70 steel, have an applied wearing surface, use high-strength concrete in the deck which is placed into biaxial compression with prestressing, reduce or eliminate deck joints and bridge bearings with the use of integral abutments and piers, and use longitudinal prestressing in the main structural components.

Three concepts are shown on the following figures. Figure 1 is a composite post-tensioned steel corrugated web box with a concrete bottom slab. The deck is cast-in-place concrete with transverse post-tensioning. Figure 2 is a post-tensioned twin warren truss with the bottom chord members connected to form a single unit. The deck is transversely post-tensioned and haunched at the top chord of the truss. Figure 3 is a twin steel post-tensioned box girder also with a haunched and post-tensioned concrete deck.

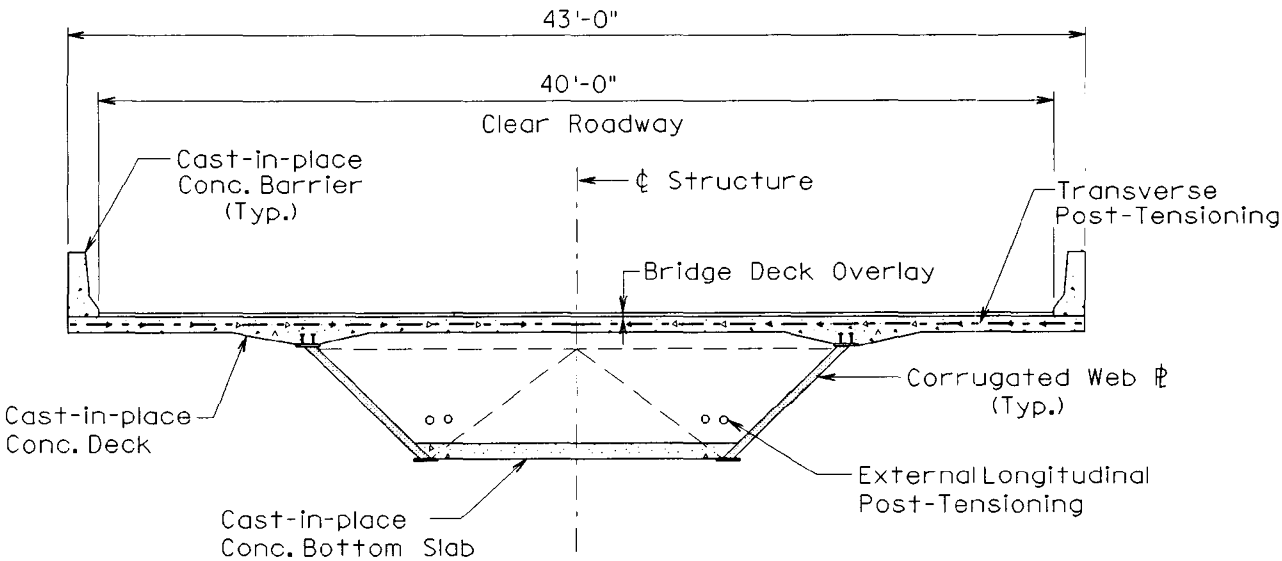

Figure 1 COMPOSITE CORRUGATED STEEL BOX

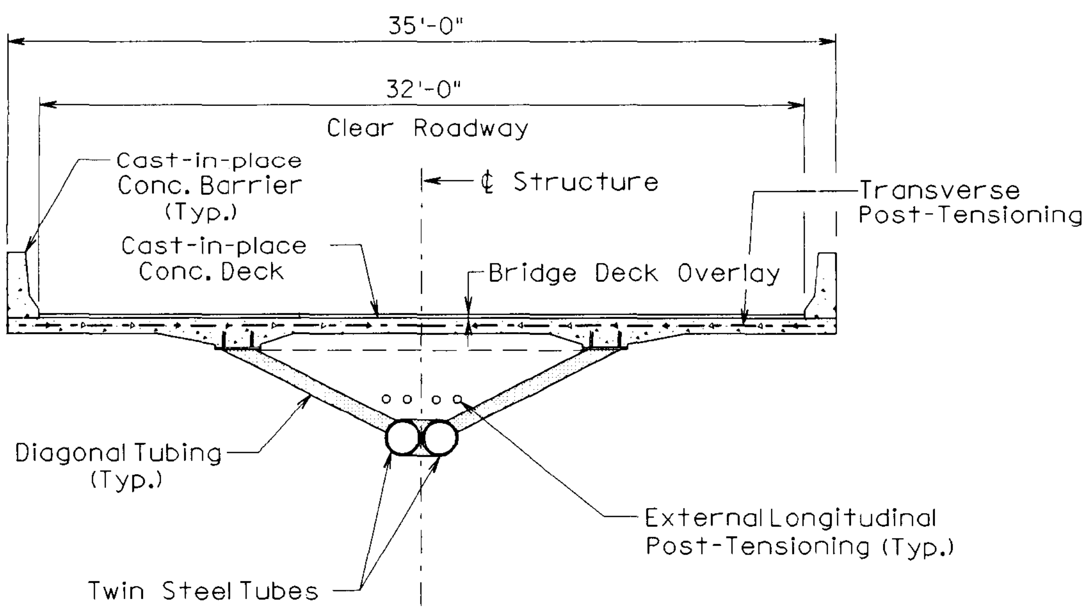

Figure 2 TWIN WARREN TRUSS

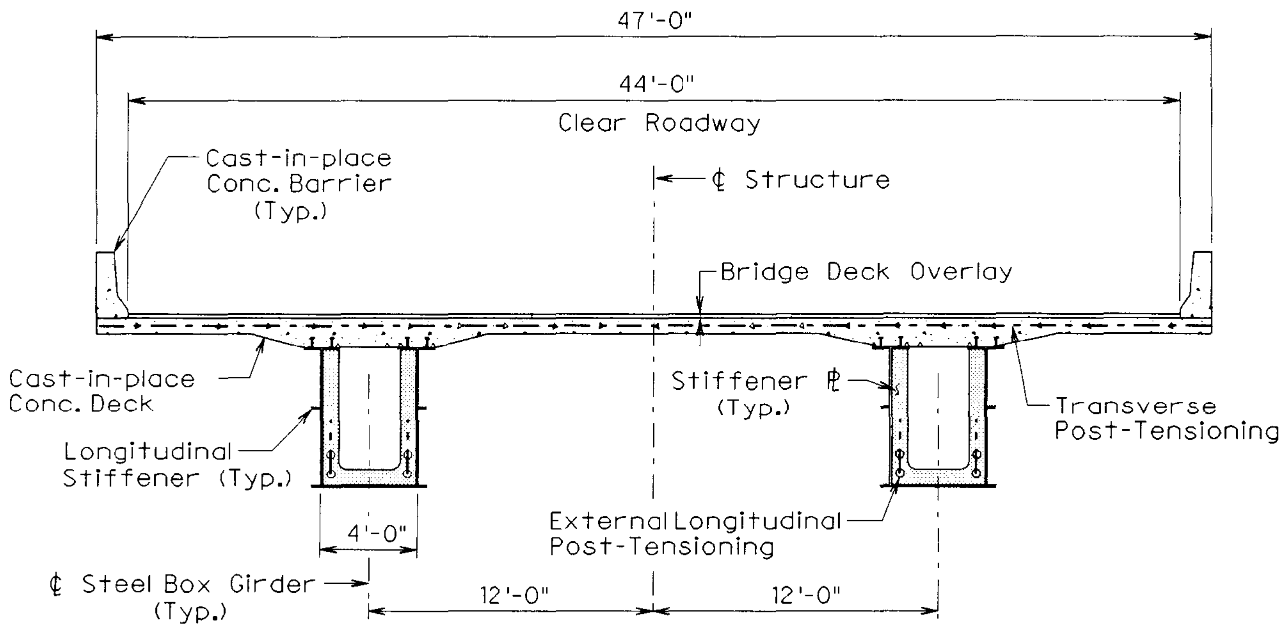

Figure 3 TWIN STEEL BOX GIRDERS

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Slab Participation for Axial Forces in Composite Cable-Stayed Bridges

Author(s) and Affiliation(s):

David D. Byers, HNTB Corporation, Kansas City, MO
Steven L. McCabe, Department of Civil Engineering
The University of Kansas, Lawrence, KS

Principal Investigator: Steven L. McCabe

Sponsor(s): The University of Kansas Graduate Research Fund, HNTB Corporation, and The National Science Foundation (pending)

Research Start Date: October 1, 1995
Expected Completion Date: October 1, 1998

Research Objectives:

The objectives of this project are to: establish a relationship for the effective width of deck that can be readily calculated in design that addresses the secondary effects caused by high axial stresses near the shear stud connections and creep effects that are a function of location in the structure; develop guidelines for a direct stiffness modeling and solution routine that can determine the immediate and long term behavior of these systems under load that will utilize an effective width, steel-concrete composite flexural member; develop recommendations for time-dependent modeling for this special class of bridge system; and gain insight into stress distribution in thin concrete deck members in composite systems subjected to axial loading that will be useful in further studies regarding stability issues of deck systems of this type.

Expected Products or Deliverables:

Guidelines for effective slab width prediction for use by designers, structural analysis algorithm using effective slab width and time-dependent analysis, and experimental calibration of analytical results.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Steven L. McCabe has been on the University of Kansas faculty since 1985. He conducts research and teaches courses in structural engineering including advanced analysis courses and reinforced concrete design. An Associate Professor of Civil Engineering at the University of Kansas in Lawrence, he received his Ph.D. from the University of Illinois in civil engineering with an emphasis on structural dynamics. During the 1995-96 academic year, Dr. McCabe is a Fulbright Scholar and Visiting Professor of Structural Engineering at the Norwegian Institute of Technology in Trondheim, Norway. He is conducting research in finite element analysis of concrete structures and the performance of headed reinforcing systems, and teaching a course in advanced concrete.

His research interests in addition to bond and development of reinforcement include earthquake engineering and structural dynamics as well as the application of computer-based analysis techniques to static and dynamic analysis problems. Other interests include the performance of bridges, specifically cable-stayed bridges, under long term sustained loading. He publishes technical papers on subjects related to these areas. He is an active member of many national and international professional societies including CRSI, ASCE and ACI. Currently, Dr. McCabe is the chair of ACI Committee 439, Steel Reinforcement, secretary of ASCE-ACI Committee 447, Finite Element Analysis of Reinforced Concrete Structures, and a member of ACI Committees 368, Earthquake Resisting Systems and 408, Bond and Development. He is also a voting member of Building Code subcommittee ACI 318-B, Bond and Reinforcement. Dr. McCabe is an invited member of the Bond Models and Size Effect task groups of the Committee Euro-International du Beton and a past associate editor for the ASCE Journal of Structural Engineering. He also serves as a consultant to engineering firms on earthquake and analysis projects. Dr. McCabe is a registered professional engineer in three states.

David D. Byers is a Ph.D. candidate in Civil Engineering at the University of Kansas. He received his BS in Architectural Engineering from the University of Kansas in 1987 and is currently completing his MS degree. Mr. Byers worked as a structural design engineer for nine years and has received an educational leave of absence from the bridge design department at HNTB Corporation, where he worked as a design engineer for the past four years. Mr. Byers' research interests focus on the design and performance of long-span cable-stayed bridges and bridge deck behavior. Mr. Byers is a registered professional engineer in the state of Missouri.

Principal Investigator's Telephone Number:	(913) 864-3826
Facsimile Number:	(913) 864-3199
E-mail Address:	slmccabe@kuhub.cc.ukans.edu

SLAB PARTICIPATION FOR AXIAL FORCES IN
COMPOSITE CABLE-STAYED BRIDGES

David D. Byers, P.E.[1] and Steven L. McCabe, Ph.D, P.E.[2]

[1] Bridge Design Engineer, HNTB Corporation, Kansas City, MO
and Ph.D. Candidate, Department of Civil Engineering, The University of Kansas
[2] Associate Professor of Civil Engineering
The University of Kansas, Lawrence, KS 66045-2225

Research Objectives

In recent years the cable-stayed bridge has become one of the most popular choices for medium and long span bridge structures throughout the world. Improved construction techniques, computer aided design capabilities and the structure's own architecturally dramatic style have all combined to increase its popularity.

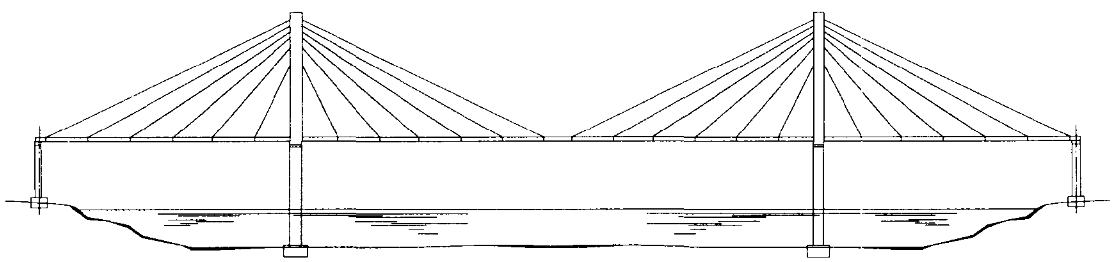

Fig. 1 Conceptual Composite Cable-Stayed Bridge

The cable-stayed bridge can be classified as a continuous structure supported by a series of elastic restraints, represented by the cables as shown in Fig. 1. The cable supports or "stays" are typically arranged in a diagonal pattern extending upward from the bridge deck or roadway to an anchorage point located along the face of a vertical column element or "pylon." The primary function of the cable is to provide intermediate vertical support of the bridge deck from above, thereby allowing the bridge to span long distances while maintaining extended vertical and horizontal clearances above the navigation channel below.

Because of the angular orientation of the cables, "the structural behavior of cable-stayed bridges requires the deck system to resist bending moments and shear forces as on ordinary bridges, but in addition axial forces. This special feature calls for an adaptation of design codes which do not explicitly consider these effects. French or British codes for composite bridges ignore axial forces and composite structures are...considered as columns rather than as girders in steel or concrete bridges" [3]. In addition to these stability issues, axial forces, when combined with bending forces created by the structure's own weight, produce deformations that will increase over the life of the structure. Also, the step-by-step, balanced erection sequence, which is generally employed for construction of cable-stayed structures, causes each portion of the structure to become loaded at varying time intervals and thus causes each member in the structure to posses a time-dependent state of stress. Time-dependent deformations also contribute to axial load sharing between the girder and the attached deck system which must be considered by the designer to accurately predict the performance of the structure.

One important issue that is not adequately addressed in the literature or design codes, but is directly related to the time-dependent axial load effects mentioned above, is the *participation of the slab* in resisting axial forces that are present in the composite deck system. The issue of slab participation constitutes a major emphasis of this research.

Research Approach

<u>Problem Description:</u> The analysis of any deck and beam system where a flanged compression region acts as a "wide" compression element requires assumptions for design purposes as to the actual width of the flange that can be assumed as acting together with the beam or girder. There is a great deal of data in the literature regarding the performance of flanged *concrete* systems under flexural load. These studies have been utilized in developing guidelines for bridge structures through AASHTO [1] as well as foreign bridge codes such as the German DIN[2]. The basic concept is one of identifying the effective portion of the deck slab that can be realistically counted on to carry compression. Conventional assumptions for flanged concrete systems in AASHTO permit the determination of the effective width based on span length, slab thickness and beam spacing.

The problem for composite concrete deck-steel girder systems is the notable difference in elastic modulus between the two materials. The steel girder absorbs tensile stress that must be reacted in compression by the relatively flexible concrete resulting in a nonlinear stress distribution. As one moves away from the girder, the compression stress distribution drops quickly due to shearing deformation of the flange elements near the web. The challenge then is to identify a simplified method for determining what the effective width should be when both flexural and axial forces are present.

Effective width or "cooperating slab width" for concentrated axial forces, similar to those introduced into the deck system through the cables, is addressed as a separate issue in the codes. The AASHTO Design Specifications, in dealing with *segmental concrete* box beams, recommends an effective width of flange generated by intersecting 30 degree lines drawn from the edge of the concrete girder stem, as shown in Fig. 2. The German DIN specification offers a similar recommendation, however only with an angle of 26 degrees. Thus, there is no consistency in the guidelines.

At this time there is limited data and guidance available and designers are forced to select either the AASHTO or DIN approach which are "borrowed" from noncomposite analysis methods or they must develop their own procedures to predict the extent to which the slab participates. In the case of composite cable-stayed structures, these assumptions regarding the width of slab that actively participates in compression with the steel girder is a critical element in accurately modeling the stiffness and time-dependent deformations of these elements.

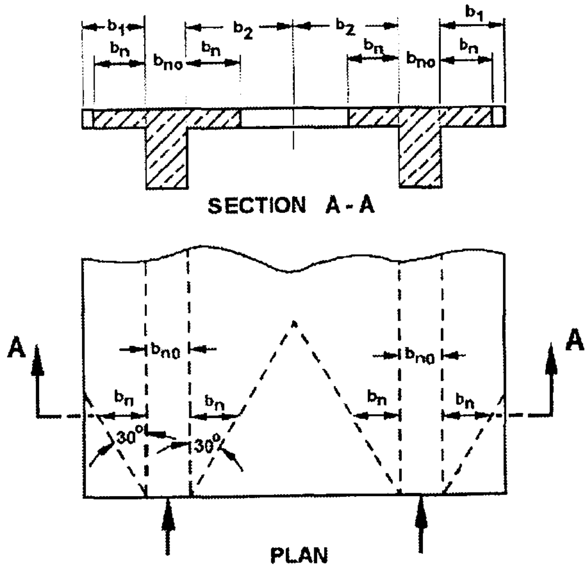

Fig. 2 "Effective Flange Widths, b_n for Normal Forces" [1]

Analytical Studies: The first phase will study these effects and their influence on the deck behavior under immediate loading. Studies using the Finite Element Method (FEM) will be conducted to establish an effective representation of the composite girder and slab system that is consistent with the behavior. This study includes three dimensional modeling of the elements and nonlinear material modeling of the deck system, especially with regard to the region where the member is made composite.

Three Dimensional Modeling: As described earlier, the state of compression in the deck changes dramatically as one moves from the girder to outer regions of the deck. Current research underway at KU gives special attention to the region of the structure at the boundary between the composite and non-composite portions. Linear assumptions in typical composite member design, using transformed sections, assumes that plane sections remain plane during bending. In this boundary region, preliminary analysis has indicated nonlinear stress-strain relations which is due to the shift in the neutral axis between the two sections and the location of the axial force being introduced.

Experimental Testing: An additional planned phase of this research will be aimed at verifying the results of the analytical study by experimental testing of two reduced scale test structures in the laboratory. A cast in place deck system and a precast panel deck system with cast in place closure strips represent the two composite test structures to be investigated. This study is aimed to address the stress distribution in the deck system during a staged erection sequence and simulated bridge closure in addition to monitoring of sustained load effects during a 90 day period prior to load testing. A direct comparison between the analytical investigation and actual behavior will then be made.

Expected Products

Bridge design has developed through the centuries in a fashion that continues to improve upon the types of materials being used, as well as to use existing materials in a more efficient manner. Thus, as new materials, design concepts and construction methods are developed, they are frequently employed in bridges because of society's need for longer, more durable spans that can be built within ever-tightening public budgets. However, with new technologies such as the cable-stayed bridge, the rush to implement the concept frequently does not permit the answering of all the important engineering questions prior to implementation. As it now stands, there is no clear guidance in the applicable codes, nor in the literature as to how to address the participation of the concrete deck system in response to axial forces in composite cable-stayed bridges.

This study will provide insight into the basic mechanics of the stress distribution process and how the response of the deck system alters the cable force and girder stresses and their distribution and the long term behavior of the overall system. Once this insight is obtained and an accurate means obtained to model the complex nature of the composite concrete deck-steel girder system, under short and long term loading, a set of guidelines will be developed to assist engineers in the design of these systems.

These guidelines will include: (1) a relationship for the effective width of deck that can be readily calculated in design that addresses the secondary effects caused by high axial stresses near the shear stud connections and creep effects that are a function of location in the structure; (2) guidelines for a Direct Stiffness modeling and solution routine that can determine the immediate and long term behavior of these systems under load that will utilize an effective width, steel-concrete composite flexural member; (3) recommendations for time-dependent modeling for this special class of bridge system; (4) insight into stress distribution in thin concrete deck members in composite systems subjected to axial loading that will be useful in further studies regarding stability issues of deck systems of this type.

Preliminary Results

Preliminary investigation has provided insight into the nonlinear stress distribution at the intersection of the composite and noncomposite region in the structure during initial loading. Incorporation of these stress effects into a step-by-step, time-dependent analysis routine is being investigated.

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FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS RESEARCH ABSTRACT

Title: Non-Linear Finite Element Analysis of Composite Bridges

Author(s) and Affiliation(s):

Kuan-Chen Fu and Feng Lu
Department of Civil Engineering
The University of Toledo, Toledo, Ohio

Principal Investigator: Kuan-Chen Fu

Sponsor(s): The University of Toledo

Research Start Date: August 1994
Expected Completion Date: February 1996

Research Objectives:

This research presents a nonlinear finite element analysis of composite bridges under working load conditions, which predicts composite bridge behavior more accurately than the current design method. The proposed analytical procedure can be readily used for the design of new bridges, for the evaluation of existing bridges, and for the study of the behavior of composite bridges under working loads.

Expected Products or Deliverables:

The objectives of this research are as follows: to establish the non-linear finite element analysis as a modern design method for composite bridges; to build test bridges in the University laboratory to collect needed data and to investigate various type of composite bridges; to devise a pre- and post-processor for the non-linear finite element software; and, to use the software as a research tool to investigate the behavior of the composite bridge under load.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Kuan-Chen Fu is Professor in the Department of Civil Engineering at the University of Toledo. He is an experienced structural engineer with interest in structural analysis, finite element methods and optimal structural systems design. He has done innovative and extensive work on structural optimization and finite element analysis. Dr. Fu was a bridge designer in Indiana for 5 years, and is a registered Professional Engineer.

Feng Lu is a graduate from the Civil Engineering Department of Tongji University. After he obtained his MS degree, he was appointed as an instructor on the teaching staff there. Currently, he is a Ph.D. candidate in the Department of Civil Engineering, The University of Toledo.

Principal Investigator's Telephone Number: (419) 530-8137
Facsimile Number: (419) 530-8116
E-mail Address:

NON-LINEAR FINITE ELEMENT ANALYSIS OF COMPOSITE BRIDGES

Kuan-Chen Fu, Ph.D., P.E. and Feng Lu, M.S.

The Department of Civil Engineering
The University of Toledo, Toledo, Ohio 43606

Research Objectives

This paper presents a nonlinear finite element analysis of composite bridges under working load conditions, which predicts composite bridge behavior more accurately than the current design method. The proposed analytical procedure can be readily used for the design of new bridges, for the evaluation of existing bridges and for the study of the behavior of composite bridges under working loads.

Research Approach

The most popular form of bridge in service is the concrete deck on steel girder bridge with shear studs providing composite actions. There are more than half a million of this kind of bridges in the US federal interstate highway system. The sheer number of the bridges demands our special attention. Nevertheless, the current method in designing the bridge is the traditional transformed section method. This method assumes that both steel girders and the composite concrete deck are made of linearly elastic materials. Under the working load conditions, the steel girder is indeed in its elastic range; but the concrete portion cannot be adequately represented by a linearly elastic model, since concrete is a non-linear material with very small tensile strength. This perhaps is one of the major reasons why the computational results produced by the method often contain large discrepancies from the experimental results.

The need for safe and economical bridges can be satisfied to the fullest only when the analysis method is closely reflecting the behavior of the bridges. Therefore, building a more realistic numerical model for the bridge is an important first step. The writers propose to use the nonlinear finite element method for the task so that the bridge structural components can work as a whole and the interactions among them can be better represented. More important is that the characteristics of the concrete under load can be simulated in the finite element model. The characteristics are:

1. The concrete is a nonlinear material in its compressive range.
2. The concrete has very small tensile strength; beyond that, the concrete cracks.
3. The concrete behaves differently under bi-axial loads than uni-axial load.

The technique to build such a numerical model is outlined as follows:

1. The flanges and the web of the steel girder are modeled by eight-node isoparametric quadrilateral shell elements. These elements are considered to be linearly elastic with a constant modulus of elasticity.

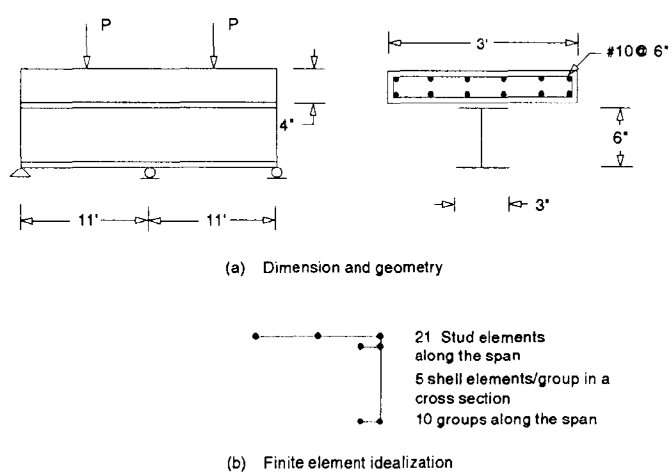
2. In the concrete deck, the reinforcing bars are idealized as a smeared two-dimensional membrane of equivalent thickness, which is modeled by an isoparametric plane stress element. Its stiffness is treated as an integral part of the concrete deck element. The shear studs are modeled by bar elements; each bar provides a dimensionless link between the concrete deck element and the neighboring top flange element of the girder.

3. The concrete deck is also modeled by isoparametric quadrilateral shell elements, but the material characteristics are represented by the Kupfer's envelope [1] which adequately describes the nonlinear behavior of the concrete under bi-axial loading conditions. To facilitate the simulation of the non-linear stress-strain relation in the compressive region, the applied load is divided into several increments and an iterative process is utilized. To reflect the non-linear variation of the modulus of elasticity, the stress-strain incremental relationship of concrete developed by Darwin and Pecknold[2] is adopted. Thus, the stiffness of the element at each load increment is computed. On the other hand, when the concrete is subjected to tensile stresses, hair cracks are likely produced. To reflect this phenomenon in the model, the extensiveness of the cracks at the Gauss points in the element are continuously examined and the stiffness of the element at each load increment is re-calculated accordingly. If crack or crushing occurs, the value of the modulus of elasticity is set to zero at that Guass point and the unbalanced forces induced are redistributed in the next iteration.

In summary, the aforementioned iterative procedure is an effective way to simulate the behavior of composite bridges.

Preliminary Results

The nonlinear analysis procedure has been coded in Fortran language. Due to time limitation, the mode of the input and output of the program is primitive. A two-span continuous composite test bridge is selected to demonstrate the applicability of the software, the results are excellent. This bridge has two equal spans with one superimposed point load of 12.2 tons at the mid point of each span as shown in Figure 1. The girder is a rolled steel I-section 152mm x 76mm x 0.0053 kN/m (6 in x 3 in x 12 lb/ft). The concrete has a strength of 47.6 MPa (6,900 psi). Shear studs are installed along the entire girder. This bridge was tested at The Imperial college[3].



Fif 1 Two span continuous beam

The applied loads are divided into 8 increments for the computer computation. The final deflections are collected along the bridge at every tenth of the span length; they are plotted on Figure 2 together with the experimental data of The Imperial College. Figure 2 shows that the numerical results are slightly larger than the experimental values on the left span, and slightly smaller than those in the right span. Basically, they agree nicely with each other.

The same bridge is analyzed by the transformed section method. The AASHTO manual specifies that: use n=8 for the live load and n = 24 for superimposed dead load such as the weight of future wearing surface; where n is the ratio of the modulus of elasticity of steel to that of concrete. Since the point loads in the experiment may be considered as either dead load or live load, both cases are investigated. However, for live load, 6.6 is used here for the value of n instead of 8, since 6.6 is the exact modulus ratio for the materials of the specimen. The computational results and the experimental data are again plotted together in Figure 3. The difference between the two is indeed very large.

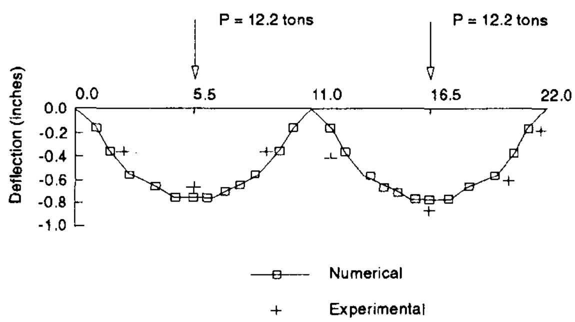
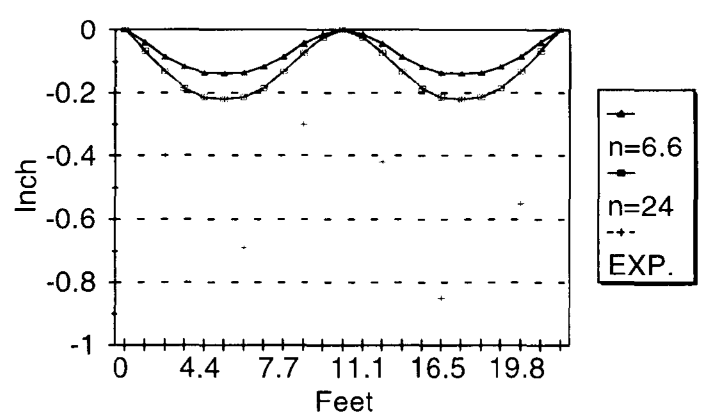


Fig. 2 Deflected shape

Fig. 3 Deflected shape



Expected Products

Observing the preliminary results presented in the previous section, the proposed nonlinear finite element technique is far better than the transformed section method. It suggests that the research is a step in the right direction. For the benefit of the bridge industry, this research must be carried on to reach its intended goals:

1. To establish the nonlinear finite element analysis as a modern design method for composite bridges by applying it to more cases of test bridges and obtain good results.
2. To use beam or plate elements to model the girder flange instead of shell elements. The purpose is to reduce the computational effort.
3. To build test bridges in the university laboratory to collect needed data and to investigate various type of composite bridges.
4. To devise a pre-processor and a post-processor for the nonlinear finite element software. The aim is to make the complicated software user friendly, so bridge engineers can use it easily for their designs.
5. To prepare a detailed documentation for the software. The purpose is to make the sophisticated program transparent, so the software may be easily modified in the future to include new research findings.
6. To modify the nonlinear analysis procedure and the software for the analyses of other type of bridges such as box girder bridges, cable-stay bridges and suspension bridges where concrete deck is used as an integral part to provide composite action.
7. To utilize the software as a research tool to investigate the validity of major design factors specified by AASHTO such as the load distribution factors for interior and exterior girders.

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[3] Yam, L.C. P. and Chapman, J.C., The Inelastic Behavior of Continuous Composite Beams of Steel and Concrete. Proc. Inst. Civil Engng 53, 487-501 (1972)

Steel Bridges

SESSION IV

[redacted]

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS RESEARCH ABSTRACT

Title: FHWA Curved Steel Bridge Research Project

Author(s) and Affiliation(s):
Michael A. Grubb and John M. Yadlosky, HDR Engineering, Inc., Pittsburgh, PA
Abdul-Hamid Zureick, Roberto Leon and Jim Burrell, Georgia Inst. of Tech., Atlanta, GA
Dann H. Hall, BSDI, Ltd., Coopersburg, PA
Chai H. Yoo, Auburn University, Auburn, AL
Sheila Rimal Duwadi, Federal Highway Administration, McLean, VA

Principal Investigator: Michael A. Grubb

Sponsor(s): Federal Highway Administration

Research Start Date: October 1, 1992
Expected Completion Date: October 30, 1999

Research Objectives:

Horizontally curved-girder bridges comprise up to a third of the total bridge market. Although their use has been on the increase, horizontally curved steel girders remain one of the least understood structural forms and one of the few unexplored frontiers of structural engineering research. Since the late 1960's, fundamental research on the behavior of horizontally curved girders has been sporadic and a concentrated effort to advance existing knowledge has been lacking. Thus, the Federal Highway Administration initiated this project with the primary objectives being to conduct fundamental analytical and physical research on the behavior of curved steel I-girder bridges, address constructibility issues, and provide data for eventual development by others of LRFD design provisions for horizontally curved steel bridges.

Expected Products or Deliverables:

Valuable information should be obtained to advance the state-of-the-art knowledge regarding the fundamental behavior of curved I-girders in bending and shear. It is also hoped to be able to correlate test results with results from refined computer analyses, increase the general awareness of some important issues that arise during construction of these bridges, and through the development and implementation of improved design and construction procedures for these structures, lead to significant cost savings.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Michael A. Grubb Senior Steel Bridge Design Specialist in HDR's Pittsburgh office, has more than 16 years of diversified experience in applied research and development, and technical marketing of structural steel and plate products. He is a graduate of Clarkson University with an MS degree from Cornell University.

John M. Yadlosky is Project Manager and Assistant Section Manager - Structures in HDR's Pittsburgh office. He has experience in the development of bridge plans, structural analysis and ratings, and cost estimates. He graduated from Bucknell University with an MS degree from Carnegie-Mellon University.

Abdul-Hamid Zureick is head of the Structural Engineering and Mechanics Group of the School of Civil Engineering at Georgia Tech. He has extensive experience in testing, design and construction of fiber-reinforced composite structures, and design of steel structures. He received his Ph.D. from the University of Illinois.

Roberto T. Leon Professor in the School of Civil Engineering at Georgia Tech, has experience in design of composite structures, experimental stress analysis, testing of full-scale and model structures in the laboratory, and field instrumentation of structures. He received his Ph.D. degree from the University of Texas at Austin.

Jim Burrell is a Ph.D. candidate in the School of Civil Engineering at Georgia Tech.

Dann H. Hall is a structural engineer with experience in bridge design and computer software development. He is known for the development of analysis, design and steel-girder optimization software. He has degrees from Clarkson University and Lehigh University.

Chai H. Yoo Professor in the Department of Civil Engineering at Auburn University, has experience in numerical analysis, finite-element analysis and the behavior of horizontally curved girders. He received his Ph.D. degree from the University of Maryland.

Sheila Duwadi is a Research Structural Engineer at the FHWA's Turner-Fairbank Highway Research Center with responsibility for research projects in steel, concrete and timber. She has degrees from Washington State University and Oregon State University.

Principal Investigator's Telephone Number: (412) 497-6079
Facsimile Number: (412) 497-6080
E-mail Address:

FHWA CURVED STEEL BRIDGE RESEARCH PROJECT

Michael A. Grubb, P.E. and John M. Yadlosky, P.E.[1]
Abdul-Hamid Zureick, P.E., Roberto T. Leon, P.E. and Jim Burrell[2]
Dann H. Hall[3]
Chai H. Yoo, P.E.[4]
and Sheila Rimal Duwadi[5]

[1]HDR Engineering, Inc.
3 Gateway Center, Pittsburgh, PA 15222-1074
[2]School of Civil Engineering
Georgia Institute of Technology, Atlanta, GA 30332
[3]BSDI Ltd.
562 Thomas Street, Coopersburg, PA 18036
[4]Department of Civil Engineering
Auburn University, Auburn, AL 36849
[5]Turner-Fairbank Highway Research Center
Federal Highway Administration, McLean, VA 22101

Research Objectives

Horizontally curved-girder bridges comprise up to a third of the total steel-bridge market. Although their use has increased steadily over the last 25 years, curved steel girders remain one of the least understood structural forms and one of the few unexplored frontiers of structural engineering research. Since the late 1960s, fundamental research on the behavior of horizontally curved girders has been sporadic; a concentrated effort to advance existing knowledge has been lacking. Thus, the Federal Highway Administration (FHWA) initiated the Curved Steel Bridge Research Project (CSBRP), with the primary objectives being to conduct fundamental analytical and physical research on the behavior of curved steel I-girder bridges, to address constructibility issues, and to provide data for eventual development by others of Load and Resistance Factor Design provisions for curved steel bridges.

Research Approach

The work on the CSBRP consists of 7 primary tasks. Each of these tasks is directly related in some fashion to an ambitious laboratory experimental plan that has been developed. Physical testing of curved-girder specimens is extremely complex. This fact is reflected in the limited amount and nature of such tests around the world to date. The tests that have been done have generally been conducted on small-scale doubly symmetrical specimens composed of very thin plate material; many of which were subject to unrealistic boundary conditions at their ends.

Most structural forms can be thought of as composed of a series of connected individual elements each possessing their own strength and stability; thus, the individual members can be tested alone as single components. In fact, the entire AASHTO specification is based on designing each individual member based on its component strength.

To properly examine the behavior of horizontally curved girders in bending and shear, it becomes highly desirable to test them in situ; that is, to conduct the tests on individual components that are part of a complete bridge structure that resists vertical loads and torsion as a system. This idea has led to the concept of testing I-girder component specimens within a full-scale three-girder bridge to ensure more realistic boundary conditions at the ends of the specimens. The use of full-scale specimens eliminates modeling and scaling concerns and the results are considered to be more directly applicable to real bridges.

The FHWA Structures Laboratory at the Turner-Fairbank Highway Research Center is singularly appropriate for the task of properly testing curved girders because a full-scale bridge can be constructed and loaded on the laboratory test floor. The use of a test frame that looks and functions like a real bridge provides two significant advantages: 1) more realistic boundary conditions are provided for the component specimen testing, and 2) for economy, parts of the test frame can be reused in later testing the bridge as a full-scale prototype. For example, in a later phase, a concrete deck can be added to the test frame to create a composite system.

The simple-span three-girder test frame to be constructed in the laboratory will have a span length of 90 feet and a radius of 200 feet measured at the longitudinal centerline. All supports will be radial. The transverse girder spacing has been established at 8.75 feet. Full-scale (48-inch web depth) singly and doubly symmetric non-composite I-girder component specimens will initially be inserted at different locations in the outermost girder of the three-girder test frame; thus, all specimens will have a radius of 208.75 feet. All steel will have a specified minimum yield strength of 50 ksi, except for the center girder, which will be fabricated from steel having a specified minimum yield strength of 70 ksi. Higher yield-strength steel is specified for this girder to help ensure that the component specimens can be loaded to their ultimate load, while the remainder of the test frame remains elastic.

Pipe sections will be used for all cross-frame members since force measurements in these members are crucial; instrumenting and deducing the axial force in tee- or angle-section members is not as reliable due to the presence of significant torsional warping stresses. Prior to testing of the frame, individual pipe-section components from the cross frames will be loaded and various methods of calibration for the measurement of member actions will be studied. The cross-frame spacing between the two outermost girders of the frame was established to provide an unbraced length for each component specimen of approximately 15.7 feet, which results in an R/L ratio (ratio of girder radius to unbraced length) of 13.33. Most practical curved-girder designs have R/L ratios of 13.33 to 20. Thus, the tests will be conducted near the lower range of the practical limits. In addition, extra lines of cross frames will be provided between the center and innermost girder in the frame to provide additional stability to these girders, reduce the lateral flange bending stresses in these girders, ensure additional load distribution to the outside girders, and ensure that these girders remain elastic throughout the entire range of test loads.

It is planned to apply loads to the test frame so that each girder in the frame is loaded approximately equally. Because of the load distribution within the curved-girder system through the cross frames, the outside girder -- supporting the component specimen -- will receive the significant majority of the total load. Loading the girders equally also helps to prevent uplift at the ends of the innermost girder and ensures a better and more realistic balance of the forces to each cross frame. Loading of the frame, however, is complicated because each girder will begin to twist and deflect immediately upon loading, which makes it difficult to maintain the load in a vertical position. A system has been developed to accommodate loading of the test frame on the top flanges of each girder at approximately the third points of the span.

In the first series of tests, a total of six non-composite curved I-girder specimens will be inserted at midspan of the exterior girder of the frame and subjected to near constant vertical moment. Later, five to seven additional I-girder component specimens will be inserted at one end of the exterior girder of the frame and will be subjected to various combinations of shear and vertical bending moment. Finally, a concrete deck will be cast onto the frame to create a full-scale I-girder bridge. The bridge will be loaded at service load and overload levels, and then an attempt will be made to load the composite system to its ultimate load. Studies are also planned during erection of the test frame to study important construction issues, such as lifting of individual curved girders and displacements that occur during erection sequencing.

The development of a rational and efficient set of component specimens that can be tested within a fixed budget is challenging because of the sheer number of variables involved; this is particularly true for curved-girder specimens. The number of combinations of key variables that can be developed is staggering and it is not possible to study each one independently in a single research program. However, the component specimens to be tested have been designed in an attempt to most efficiently study the effect of what are deemed to be some of the most important variables.

A potential difficulty with utilizing a three-girder system as a test frame is in determining the actions in a single component that has experienced significant yielding. A two-pronged strategy will be used to extract the bending moments and shears in the component specimens. In the first approach, the bending moments at midspan of the two innermost girders, which must remain elastic, will be determined from strain gages. The moments will then be subtracted from the computed statical moment at midspan (computed from the measured vertical reactions at all three girders) to obtain the moment in the component specimen in the outside girder. In the second approach, the forces in each cross-frame member between the center girder and the outermost girder will be measured. From the measured vertical end reactions and cross-frame forces and known applied loads, the bending moment in the component specimen will then be deduced from statics.

An exhaustive analytical study of the full-scale test frame with each of the six initial component specimens inserted (Figure 1) has been conducted at the Georgia Institute of Technology with the ABAQUS finite-element program; an analysis tool that makes it possible to better predict movements, reactions, and internal actions of the members so that the planned test program can be carried out safely. The analyses included both material and geometric non-linear effects. The detailed model includes all components of the test frame, including splices, connection plates, stiffeners and the load fixtures. Essentially, tests of the component specimens up to the ultimate load within the frame are being simulated on the computer. As an independent check, a similar analysis on a less detailed model is being undertaken at Auburn University using MSC/NASTRAN, V68.

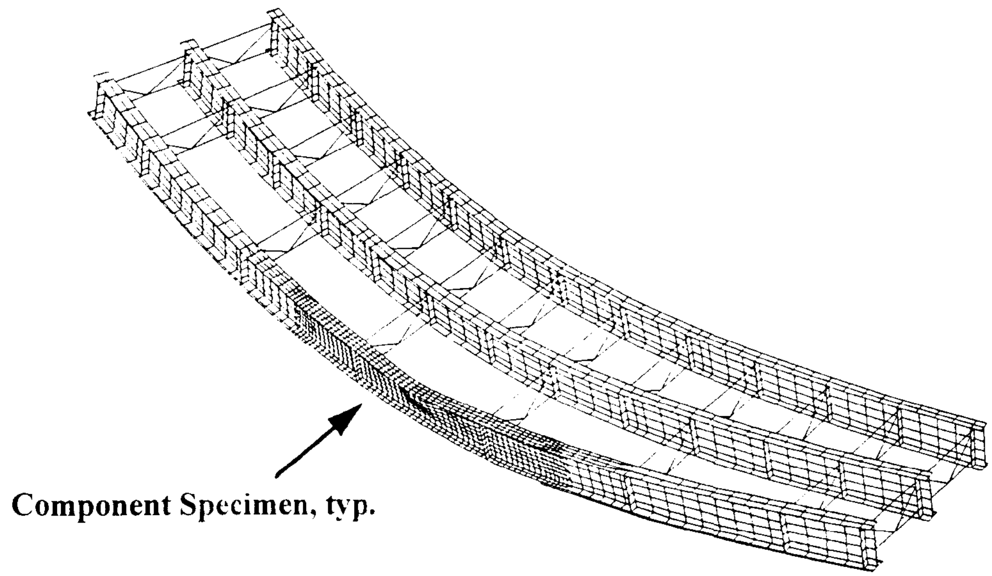

Figure 1. Detailed Analytical Model of Three-Girder Test Frame

Expected Deliverables

Since these will be the first known tests to failure of full-scale doubly and singly symmetric curved I girders within a realistic structure used as a test frame, and of a full-scale composite curved I-girder bridge, valuable information should be obtained to advance the state-of-the-art knowledge regarding the fundamental behavior of curved I girders in bending and shear. With the current exponential increases in computational power, it is also hoped to be able to correlate the test results with the results from refined computer analyses, which should lead to future analytical and experimental studies by others to confirm the results and generate additional data. Results from this project should also help to increase the general awareness of some important issues that arise during construction of these bridges. Since curved bridges represent a large percentage of the steel bridges to be constructed over the next decade, the development and implementation of improved design and construction procedures for these structures should lead to significant cost savings.

Preliminary Results

From the results of the refined analyses that have been conducted to date, the concept of using a three-girder test frame for component testing appears to be feasible. Fabrication of the test frame is scheduled to get underway this summer, with testing scheduled to commence in early 1997.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: High Performance Steels for Highway Bridges

Author(s) and Affiliation(s):

William Wright
Turner-Fairbank Highway Research Center
Federal Highway Administration, McLean, VA

Principal Investigator: Various

Sponsor(s): Federal Highway Administration, Naval Surface Warfare Center, and American Iron and Steel Institute

Research Start Date: 1992
Expected Completion Date: 1998

Research Objectives:

The objectives of these studies are (1) to develop new high performance steel grades 70W and 100W that are optimized for highway bridge construction. The primary focus is on improved weldability and toughness compared to existing steels of this strength. The approach is to greatly reduce the carbon content of the alloy and use advanced processing and micro-alloy additions to keep the required strength; (2) to produce full size plates of the new HPS grades and perform extensive laboratory testing to verify quality and performance in structural members. This will include both material property tests and full scale tests on welded plate girder members; and (3) to introduce these new steels into the market by fabricating a series of experimental demonstration bridge projects in partnership with State bridge owners.

Expected Products or Deliverables:

The development of new steel grades with appropriate specifications for adoption into the AASHTO M-270 specification.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

William Wright, P.E. has been employed for the last eight years as a research structural engineer at the Federal Highway Administration's Turner-Fairbank Highway Research Center. Responsibilities include management of the Structures Laboratory and performing research in the area of structural steel. Mr. Wright holds BSCE and MSCE degrees from the University of Maryland in College Park and is currently working on a Ph.D. degree in structural engineering at Lehigh University. Mr. Wright is a registered professional engineer in the state of Maryland.

Principal Investigator's Telephone Number:	(703) 285-2496
Facsimile Number:	(703) 285-2766
E-mail Address:	wwright@intergate.dot.gov

HIGH PERFORMANCE STEELS FOR HIGHWAY BRIDGES

William Wright, P.E.

Turner-Fairbank Highway Research Center
Federal Highway Administration
McLean, VA 22101

Research Objectives

Over the last four years, the Federal Highway Administration (FHWA) has been sponsoring development work on a new generation of high performance steel (HPS) designed to improve both the quality and economy of steel bridge construction. The first product from this research is an improved grade of 480 MPA (70 ksi) weathering steel, grade HPS-70W, that possesses superior weldability and toughness compared to the current A709 grade 70W steel. The new steel promises to streamline the fabrication process through elimination of weld cracking and preheat requirements. Greatly improved toughness should give bridge owners the confidence to allow full utilization of the steels strength in many types of steel bridge designs, including "fracture critical" applications. As a result, it is anticipated that optimized bridge designs utilizing HPS70W will be both more economical from a first cost basis as well as being more tolerant of damage throughout their life-cycle. This paper will provide a brief overview of the steel development, followed by describing the research that is currently in progress to verify the structural performance of the new steel prior to implementation.

Research Approach

Steel Development: The development of the new HPS grade has been performed under a unique partnership between the Federal Government and the steel industry. On the government side, the FHWA has entered an interagency agreement with the Carderock Division of the Naval Surface Warfare Center in Annapolis, MD. A cooperative agreement was then established with the American Iron and Steel Institute to include industry in this effort. All research work on steel development has been conducted under the direction of a steering committee consisting of representatives of FHWA, USN, and four of the major steel plate producers. The scope of research was to develop improved steels through a combination of advanced processing techniques and alloy additions to achieve the following: 1) 480 and 690 MPa yield strength; 2) weldability without preheat; 3) toughness at least suitable for use in AASHTO Zone III construction; and 4) corrosion resistance equal or greater than A588 weathering steel. The HPS-70W grade steel is the first product from this research, however, work is continuing on the HPS-grade 100W as well as a second generation of grade 70W steel that does not require quench and temper processing.

The first full scale production heat of HPS-70W (200 ton) was melted and processed into plate during March 1996. Full size plates were produced with thickness ranging from 10 to 64 mm

(0.375 to 2.5 in.). Mechanical property testing is currently underway to determine the strength and toughness properties of each plate at several locations within the plate.

Weldability testing: In January 1995, a workshop was conducted to determine how to define weldability and determine the most appropriate tests to evaluate HPS. Three tests were identified: 1) Gap-Bead-on-Plate (GBOP); 2) Tekken Test; and 3) Implant test. These tests are deemed sufficient to compare and contrast HPS with the existing steels in the A709 specification. After completion of this testing, sufficient information will exist to develop provisions for inclusion in the AWS D1.5 Bridge Welding Code. Preliminary estimates indicate that existing welding consumables will give adequate performance with HPS-70W and that no pre / post heating will be required to control weld cracking. This contrasts with the 125°F preheat requirement in the AWS specification for A709 grade 70W steel. Unless this research indicates otherwise, low hydrogen H4 practice will probably still be required when welding HPS.

Fatigue / Fracture Testing: While most people would agree that high toughness is a desirable property in a bridge steel, there has been little work to determine what effect toughness has on bridge reliability and performance. The current fracture control plan in the AASHTO specifications gives minimum toughness requirements to prevent problems with the existing grades of steel in the A709 specification. There is no consideration given for steels that have much higher toughness than the minimum value. It needs to be determined what, if any improvement can be made in bridge reliability when above-minimum toughness steels are used. Various researchers have suggested that it may be possible to eliminate design and inspection penalties for fracture critical members if the steel has sufficient toughness. Preliminary test results for the new HPS-70W steel indicate toughness as high as 200 ft-lb @ -10°F may be possible. This type of upper-shelf behavior suggests that brittle fracture may no longer be a limit state of concern for bridges fabricated from high toughness HPS.

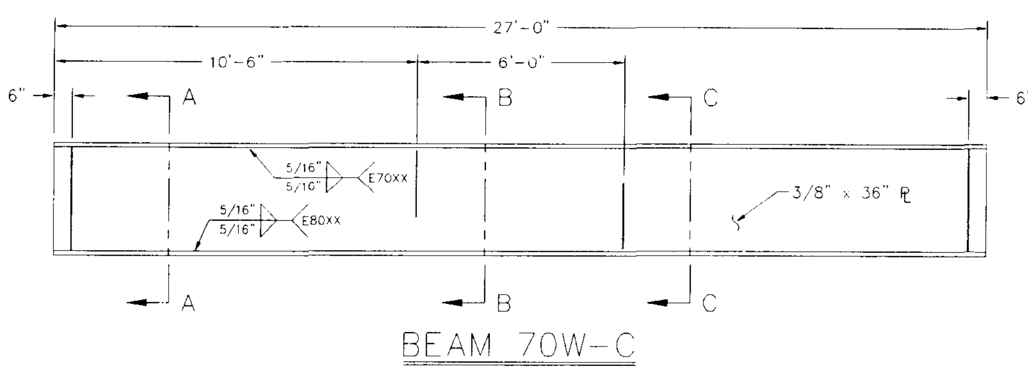

Figure 1. HPS test girder for fatigue / fracture testing

A series of plate girders is being fabricated from the HPS-70W plates for testing at the Turner-Fairbank Highway Research Center to study the fracture performance of plate girders fabricated from HPS. A typical girder, shown in figure 1, is 990 mm deep by 8.22 m long (39 in. By 27 ft.). Four-point loading will be applied using 978 kN capacity servo-hydraulic jacks. Fatigue cracks will be initiated and grown to a predetermined size at critical locations on the girder. An overload stress will then be applied to attempt to force brittle fracture to occur. If no failure occurs, additional fatigue cycles will be applied to increase the crack size and the overload repeated. Eventually, either brittle fracture will occur or there will be insufficient section remaining to resist the overload. To simulate the worst case encountered in bridge service, the girder will be cooled to AASHTO Zone III temperatures prior to each overload test.

In addition to the full scale tests, a series of small scale fatigue tests will be conducted to develop fatigue life data for AASHTO category A and C details. It is not anticipated that the new HPS will show significantly different fatigue performance from current steels, but this needs to be demonstrated.

Once the fatigue / fracture performance of HPS members is determined, a model will be developed to relate this performance with small scale test results. Further, computer modeling is planned to determine what effect HPS may have on redundancy of a typical two-girder bridge.

<u>Suitability for Plastic Design:</u> The compact section criterion for allowing plastic design in the AASHTO specifications was developed largely based on data developed from lower strength steels. Limited data exists for steels with yield strength greater than 50 ksi. The AASHTO LRFD specification and the soon-to-be-released 16th Edition of the Standard Specification limit plastic design to steels with yield strength of 345 MPa (50 ksi) or lower. This places limits on the efficiency of bridge designs when higher strength steels are used. Although there is no indication that plastic design will be a problem with the new HPS-70W, this needs to be verified by testing to before plastic design can be justified.

A series of tests is planned for both the University of Nebraska and at Lehigh University to study the moment-rotation behavior of HPS-70W. The scope of testing is currently being defined but it is expected to consist of small scale tests, full scale beam tests, and tests of beams with composite deck slabs. The results should be sufficient to determine if plastic design should be allowed for HPS-70W within the limits of the AASHTO code.

<u>Demonstration Projects:</u> Construction of actual bridges fabricated from HPS-70W will be the culmination of this research program. The first step will be to directly substitute the new HPS-70W for grade 50W in several bridges planned for construction in the near future. No design optimization will be performed and the additional strength and toughness benefits of HPS will not be utilized. This will, however, allow documentation of the fabrication process and allow a data base to be developed concerning the fabricability and quality of the product. If any unexpected problems arise, there will be a significant safety cushion available to the bridge owner with this approach.

The economic impact of designing with HPS will be determined in a second phase of demonstrations where bridge designs are fully optimized to exploit the properties of HPS. Research at Lehigh University has shown that for several typical bridge designs, it is possible to reduce the amount of steel by up to 18 percent when strength is increased from 50 to 70 ksi.[1] The other factors that affect the in-place cost of a bridge also need to be considered, including fabrication efficiency, substructure requirements, and cost of steel. It is anticipated that the total in-place cost of many bridges fabricated from HPS-70W will be lowered, however, this needs to be verified under realistic construction conditions.

The ultimate potential for improving both the cost and performance of steel bridges may come from entirely new design concepts that are optimized for HPS. New structural shapes, mixed material types, and improved design systems are some of the possibilities being considered. Although it is beyond the scope of this paper, parallel research is being sponsored by FHWA to investigate the potential of innovative design types.

Expected Product

Overall, this research has the potential to make a major impact on the economics of steel bridge design. The immediate benefit of lower first cost is possible, as well as reduced life-cycle and maintenance costs over the life of a typical structure. The specific product will be the new HPS-70W grade of steel along with sufficient research data to show it's viability for bridge construction. Additional products will continue to result in the future, including an HPS-100W grade and a second generation of the 70W grade.

References

(1) Homma, K., and Sause, R., "Potential for High Performance Steel in Bridges," Proceedings, ASCE Structures Congress XIII, American Society of Civil Engineers, New York, NY, April, 1995.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Structural Systems for High Performance Steel Bridges

Author(s) and Affiliation(s):

John M. Kulicki, Wagdy G. Wassef and Philip A. Ritchie
Modjeski and Masters, Inc.
Harrisburg, PA

Principal Investigator: John M. Kulicki

Sponsor(s): Federal Highway Administration

Research Start Date: Summer 1993
Expected Completion Date: Summer 1996

Research Objectives:

The objectives of this project are to identify the design and construction impediments facing today's steel bridges, to solicit new ideas for tomorrow's bridges, namely, ideas that will maximize the exploitation of high performance steel in bridge construction and to evaluate the potential of the suggested innovative systems.

Expected Products or Deliverables:

The expected products of this project include a report that includes compilations of the impediments to the use of steel bridges and the recommendations of all groups involved in bridge construction to eliminate or minimize these impediments. The report will also include comparisons between bridges designed using conventional systems and innovative systems.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

John M. Kulicki is currently President and Chief Engineer of Modjeski and Masters, Inc. He has been principal-in-charge of such note-worthy bridge projects as the Bay View bridge in Quincy, Illinois, the Second Bluewater Bridge in Port Huron, Michigan, and the rehabilitation of the I-24 bridge near Paducah, Kentucky. He was principal investigator during the development of the AASHTO-LRFD Bridge Design Specifications. He received his Ph.D. from Lehigh University and is a registered Professional Engineer in 14 states.

Wagdy G. Wassef is a Senior Engineer with Modjeski and Masters, Inc. He has been involved in several bridge projects including the Second Bluewater Bridge in Port Huron, Michigan, studies on the Newburgh-Beacon bridges near Newburgh, New York, and some studies related to the AASHTO-LRFD Specifications and the prioritization of seismic rehabilitation of bridges. Dr. Wassef is a registered Proessional Engineer in the state of Iowa.

Philip A. Ritchie is a Structural Designer with Modjeski and Masters, Inc. He has been involved in the Second Bluewater Bridge in Port Huron, Michigan. Dr. Ritchie's research interests include the development of innovative bridge systems and the structural applications of fiber composites.

Principal Investigator's Telephone Number: (717) 790-9565
Facsimile Number: (717) 790-9564
E-mail Address:

STRUCTURAL SYSTEMS FOR HIGH PERFORMANCE STEEL BRIDGES

John M. Kulicki, Ph.D., P.E.[1]
Wagdy G. Wassef, Ph.D., P.E.[2]
Philip A. Ritchie, Ph.D.[3]

[1]President and Chief Engineer
Modjeski and Masters Inc., Harrisburg PA 17105
[2]Senior Engineer
Modjeski and Masters Inc., Harrisburg PA 17105
[3]Structural Designer
Modjeski and Masters Inc., Harrisburg PA 17105

Research Objectives

The objectives of this project are to identify the design and construction impediments facing today's steel bridges, to solicit new ideas for tomorrow's bridges, namely, ideas that will maximize the exploitation of high performance steel (HPS) in bridge construction and to evaluate the potential of the suggested innovative systems. Innovative structural systems are expected to improve the economy of steel bridges, a vital link in the highway system.

Research Approach

To identify the design and construction impediments facing today's steel bridges and to solicit new ideas for future steel bridges, a group of engineers were invited to a two-day workshop that was held in February, 1994. Among the attendees were representatives from all groups involved in steel bridge construction; the Federal Highway Administration, academia, practicing engineers, steel fabricators and steel producers. These engineers were from Canada, Europe, Japan, and the United States. To evaluate the potential of innovative systems suggested in the workshop, preliminary designs of bridge structures were prepared using conventional systems and the innovative systems. These designs are being compared to evaluate the potential savings that may be realized from the use of the innovative systems. This paper summarizes some of the innovative ideas and their advantages.

<u>Material properties of High Performance Steel</u> The development effort of HPS concentrated on steels with yield strengths of 485 and 690 MPa (70 and 100 ksi). The goal was to employ advanced processing techniques to reduce the amounts of expensive alloys required to produce these grades of steel while improving other desirable properties of the material. This approach is expected to reduce the cost of the material and provide higher fracture toughness, improved weldability, improved fabricability and high corrosion resistance.

Some heats of the newly developed high performance steel were produced and rolled into plates during the winter of 1995. At the time this paper was written, the produced plates were being tested to confirm their properties.

Innovative Bridge Structural Systems Following is a description of some of the innovative systems being considered:

Built-up I-girders with corrugated webs: Corrugated plates can be used to replace the flat webs of conventional built-up welded plate girders. The corrugated plates can be produced by cold-forming long, flat plates. The corrugations can have a trapezoidal or sine-wave cross-section (see Figure 1) . While the trapezoidal corrugations are expected to be easier to form, the sine-wave corrugations are expected to have less residual cold-forming stresses and to be easier to weld to the flanges. In addition, the elimination of the sharp corners in the sine-wave corrugations is expected to facilitate welding the web to the flanges and to produce welds with better fatigue resistance.

The corrugated webs are relatively flexible when subjected to forces parallel to the longitudinal axis of the girder and, therefore, do not contribute to the moment resistance of the girders. The required flanges of a girder with a corrugated web are larger than those of a conventional girder with the same depth. However, corrugated webs are not susceptible to web stability problems and, thus, allow deeper girders with thinner webs and smaller flanges. The overall weight of the resulting girders is less than conventional built-up girders.

Built-up girders with tubular, concrete-filled flanges: A flat flange of an I-shaped girder can be replaced by a relatively thin structural tube (see Figure 2). Seamless and longitudinally or spirally welded tubes can be used. High strength, nonshrink grout can then be used to fill the tube.

The weldability of HPS will allow welding a torsionally stiff tubular flange to a relatively flexible web plate without creating fatigue problems at the connection. The web depth of this system is smaller than that of a conventional girder with the same total depth. The shallower web depth allows thinner, still stable, webs.

Tubular flanges can be used in conjunction with both flat and corrugated web plates. In the case of corrugated webs, a flat or near-flat surface of the tubular flange will be required to facilitate the weld between the web and the tube. This can be achieved by using a rectangular tube or by flattening a circular tube at the side where the weld is applied.

In the case of tension flanges, prestressing strands may be placed inside the tube. The construction sequence can be designed to ensure that the grout inside the tube will remain in compression for all load combinations. Due to confinement, the compressive strength of the grout inside the tubular flanges is expected to increase. Therefore, higher allowable stresses in the grout may be allowed.

Built-up I-girders with double, sheet metal (sandwich) webs: Panels consisting of two metal plates with a filling in between have been used in the aerospace industry for many years. Utilizing the same concept in constructing the webs of bridge girders may result in material savings. A typical cross-section of a bridge girder with double web plates is shown in Figure 3. The thickness of the web plates is expected to be less than what is generally viewed as minimum plate thickness and, therefore, high corrosion resistance is required to fully utilize the strength of the web plates.

Providing adequate connection between the two thin web plates is essential to force the two plates and the filling to act as one unit and to eliminate the possibility of each plate buckling independently. The connection between the filling and the web plates can be achieved using one of the following techniques:

- Using a filling that adheres to the web plates
- Using adhesives to connect rigid fillings to the plates
- Mechanical connection

The effect of the weld heat input on the filling material and the connection between the filling and web plates needs to be determined. The use of more advanced directed energy welding methods (e.g. laser welding) may reduce such effects. The resulting welds have less porosity and, therefore, are less susceptible to fatigue damage. In addition, the lower heat input produces lower residual stresses than what is assumed in the current design specifications.

Expected Products

The expected products of this project include a report that includes compilations of the impediments to steel bridges and the recommendations of all groups involved in bridge construction to eliminate or minimize these impediments. The report will also include comparisons between the bridges designed using conventional systems and those designed using the innovative systems. In addition, recommendations for further studies to develop design criteria for some of the aspects of the innovative systems will be included.

Preliminary Results

The preliminary results obtained from the designs conducted to-date indicate that significant savings can be realized by using the suggested innovative systems. The amount of savings varies based on the system and the location of the section in the bridge, i.e. positive or negative moment region. The effect of some unknown cost factors, such as the fabrication cost of some new components, is being evaluated.

Acknowledgements The Authors thank the FHWA for support of this project. Mr. William Wright is technical liaison for the project. Other members of the research team are the ATLSS Center at Lehigh University, in particular, Drs. J. W. Fisher and R. Sause and Dr. A. S. Nowak of the University of Michigan.

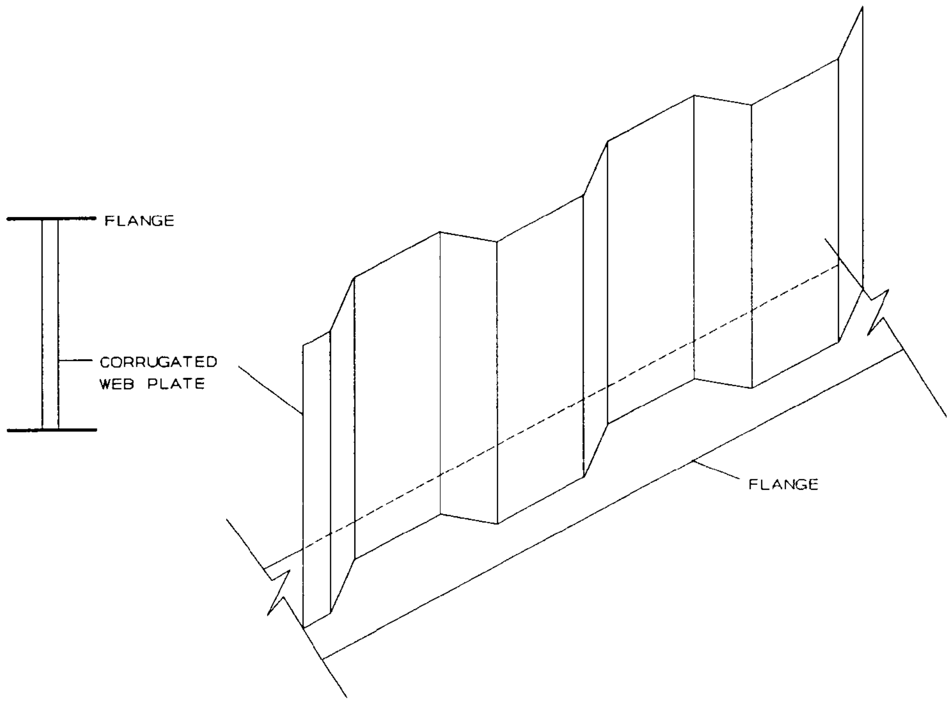

Figure 1. I-girder with Trapezoidal Corrugated Web

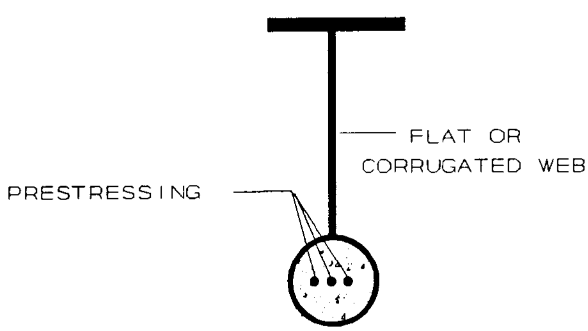

Figure 2. Girder With Concrete-Filled Tubular Flange

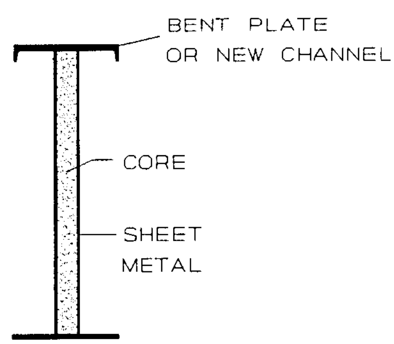

Figure 3. I-girder With Double, Sheet Metal Web

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS RESEARCH ABSTRACT

Title: Characterization of the Environment for Weathering Steel Design Considerations

Author(s) and Affiliation(s):

Christopher L. Farschon and J. Peter Ault
Ocean City Research Corporation, Ocean City, NJ
Robert A. Kogler, Turner-Fairbank Highway Research Center
Federal Highway Administration, McLean, VA

Principal Investigator: Christopher L. Farschon

Sponsor(s): Federal Highway Administration

Research Start Date: 1992
Expected Completion Date: 1998

Research Objectives:

The objective of this project is to identify the components that contribute to the corrosivity of the atmospheric environment. The performance of weathering steel and various coating materials are being studied in representative environments across the continental U.S. Meteorological data will be gathered along with material performance data, which will be used to formulate design guidelines for steel highway structures based upon the characteristics of the various macro-environments.

Expected Products or Deliverables:

The research will yield a variety of new knowledge about the corrosion of weathering steels in various environments and will attempt to correlate environmental parameters with expected performance. Novel test techniques are being developed that could be used to ascertain the corrosion suitability of weathering steel for a given environment during the design phase of a structure. This will allow bridge engineers to base material selection (steel type and coatings type) on environmental data that is either readily available or easily gathered during the design and planning phases of bridge construction.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Christopher L. Farschon is a Senior Staff Engineer with Ocean City Research Corporation. Mr. Farschon has experience as an engineer working on Federal Highway Administration contracts for four years. Mr. Farschon's previous FHWA research has focused on alternative coating materials for highway bridges and investigation of bridge maintenance painting materials and processes. Mr. Farschon holds a B.S. in Mechanical Engineering from Drexel University and is a member of NACE, SSPC, ASME, and ASTM.

J. Peter Ault is a Senior Engineer and Laboratory Manager with Ocean City Research Corporation. Mr. Ault has been working on coatings and corrosion research on Federal Highway Administration contracts for over 6 years.

Robert A. Kogler is a Materials Engineer with the Federal Highway Administration specializing in coatings and corrosion control.

Principal Investigator's Telephone Number: (609) 399-2417
Facsimile Number: (609) 399-5233
E-mail Address:

CHARACTERIZATION OF THE ENVIRONMENT FOR WEATHERING STEEL DESIGN CONSIDERATIONS

Christopher L. Farschon,[1] J. Peter Ault, P.E.,[1] and Robert A. Kogler[2]

[1]Ocean City Research Corporation
Tennessee Avenue, Ocean City, NJ 08226
[2]Turner Fairbank Federal Highway Research Center
6300 Georgetown Pike, McLean, VA 22101

Research Objectives

Under a Federal Highway Administration contract titled "Characterization of the Environment" OCRC is conducting a study designed to identify the components that contribute to the corrosivity of the atmospheric environment. Of specific interest are the various macro-environments contained within the United States. The performance of weathering steel and various coating materials are to be studied in eight environments representative of various environmental extremes in the continental United States. Meteorological data will be gathered along with the material performance data. The data gathered will be used to formulate design guidelines for steel highway structures based upon the characteristics of the various macro-environments.

Research Approach

Background: Design engineers evaluate the life cycle cost during the bridge design process. Weathering steel is an attractive construction material because it has minimal future maintenance costs for corrosion control. Guidelines[1] do exist to aide engineers in the selection of steel types for construction (steels with corrosion properties that dictate painting versus weathering type steels). OCRC is monitoring environmental conditions, steel corrosion rates, and coating material performance in various environments across the United States.

Verification: The program included a nominal 1-year effort with the goal of developing a verification test plan for the primary effort which would ultimately demonstrate relationships between performance and environmental variables. One aspect of the verification test plan was exposure testing of weathering steel at two sites to demonstrate the utility of proposed test techniques. The testing techniques performed included industry standard techniques, but also included novel methods for remote measurement of steel corrosion and trouble shooting of the remote monitoring equipment.

Site Selection: During the verification state of the research,environmental variables were monitored to distinguish macro-environments across the United States. The National

[1]FHWA Technical Advisory, *Uncoated Weathering Steel in Structures*, T5140.22, October 1989.

Atmospheric Deposition Program, National Trends Network (NADP/NTN) is a 200-station rural wet-only deposition monitoring network. Sites are located nationally from Puerto Rico to American Samoa including Alaska. The program characterizes regional patterns of deposition on a national scale by excluding monitoring site locations in close proximity to point sources or large urban centers. The majority of test sites for this research are located at, or in the vicinity of NADP/NTN stations. Rainwater chemical analysis from these stations will comprise a major source of environmental characterization data.

Environmental and Material Performance Data: Environmental variables that may influence the corrosion process of steels or the durability of protective coatings has been logged through simple monitoring and through analysis of rainwater chemistry. Each test site was equipped with a commercially available portable data logger. The data logger is capable of storing up to 55,000 samples, which can be downloaded via RS232 or via modem. The variables monitored include the following:

- Monitoring of environmental variables including; Temperature (Type J thermocouple - ambient air under the exposure rack roof, steel surface exposed facing north, steel surface exposed lying horizontal, and steel surface sheltered facing north, Humidity, and Surface Moisture (4 orientations, monitored by logging corrosion potential of a moisture dependent galvanic circuit).

- Monitoring of rainwater volume and chemistry including; pH and conductivity, SO_4^{-2}, NO_3^-, Cl^{-1}, PO_4^{-3}, Na^+, K^+, Ca^{+2}, Mg^{+2}, NH_4^+, and H^+. These variables were monitored from NADP/NTN sites.

- Monitoring of corrosion rates of ASTM A-588 weathering steel and ASTM A-36 mild steel in four different orientations (NACE Standard TM-01-69). This monitoring included weight loss samples, crevice corrosion samples, and novel corrosion "fuses" which could be remotely monitored.

- Monitoring of painted steel samples to evaluate the performance of various coating types in the test environments.

Expected Products

The research will yield a variety of new knowledge about the corrosion of weathering steels in various environments. Additionally, by carefully monitoring various environmental parameters in the test environment we may be able to correlate critical parameters and threshold values with expected performance. In addition to simply gathering data, we are developing novel test techniques that could be used to ascertain the corrosion suitability of weathering steel for a given environment during the design phase of a structure. An additional program phase (not discussed in detail in this paper) includes the evaluation of coatings in the various environments.

The information gathered will allow the highway engineer to base material selection (steel type and coatings type) on environmental data that is either readily available or could be easily gathered during the design and planning phases. Proper materials selection will provide the most cost effective structure for the given environment.

Preliminary Results

Figures 1 and 2 provide representative results from the program. Figure 1 shows corrosion rate data from three of the five exposure sites. This first year data is showing dramatic differences in the corrosion rates of weathering steel in different environments. The exposure orientations are also clearly effecting the results of the program. Figure 2 shows initial rainwater chemistry data from the same three exposure locations. For brevity we have shown corrosion rate and rainwater data for nominally one year exposure at three sites. Additional data will be presented including effects of pH and time-of-wetness.

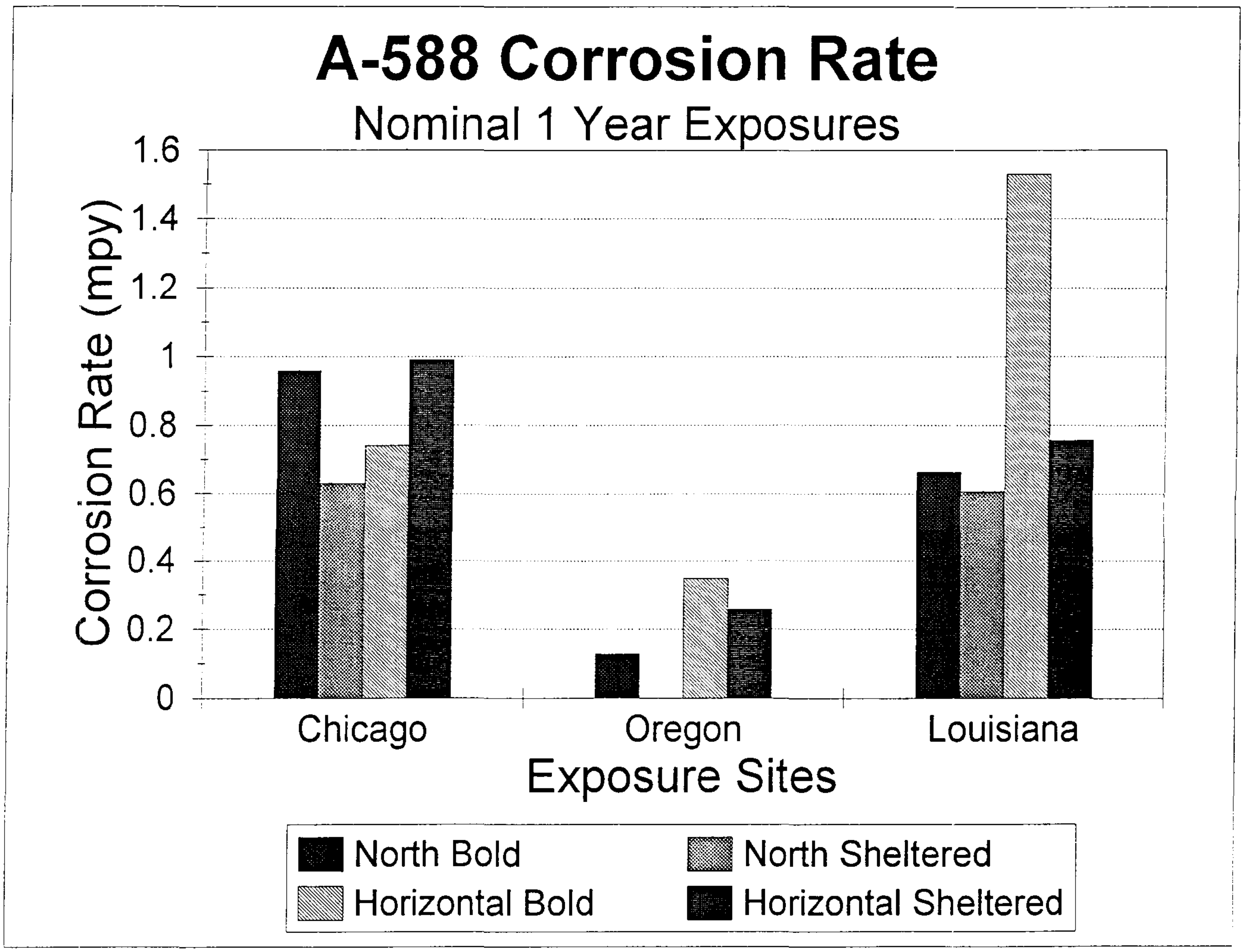

Figure 1. Corrosion rate data.

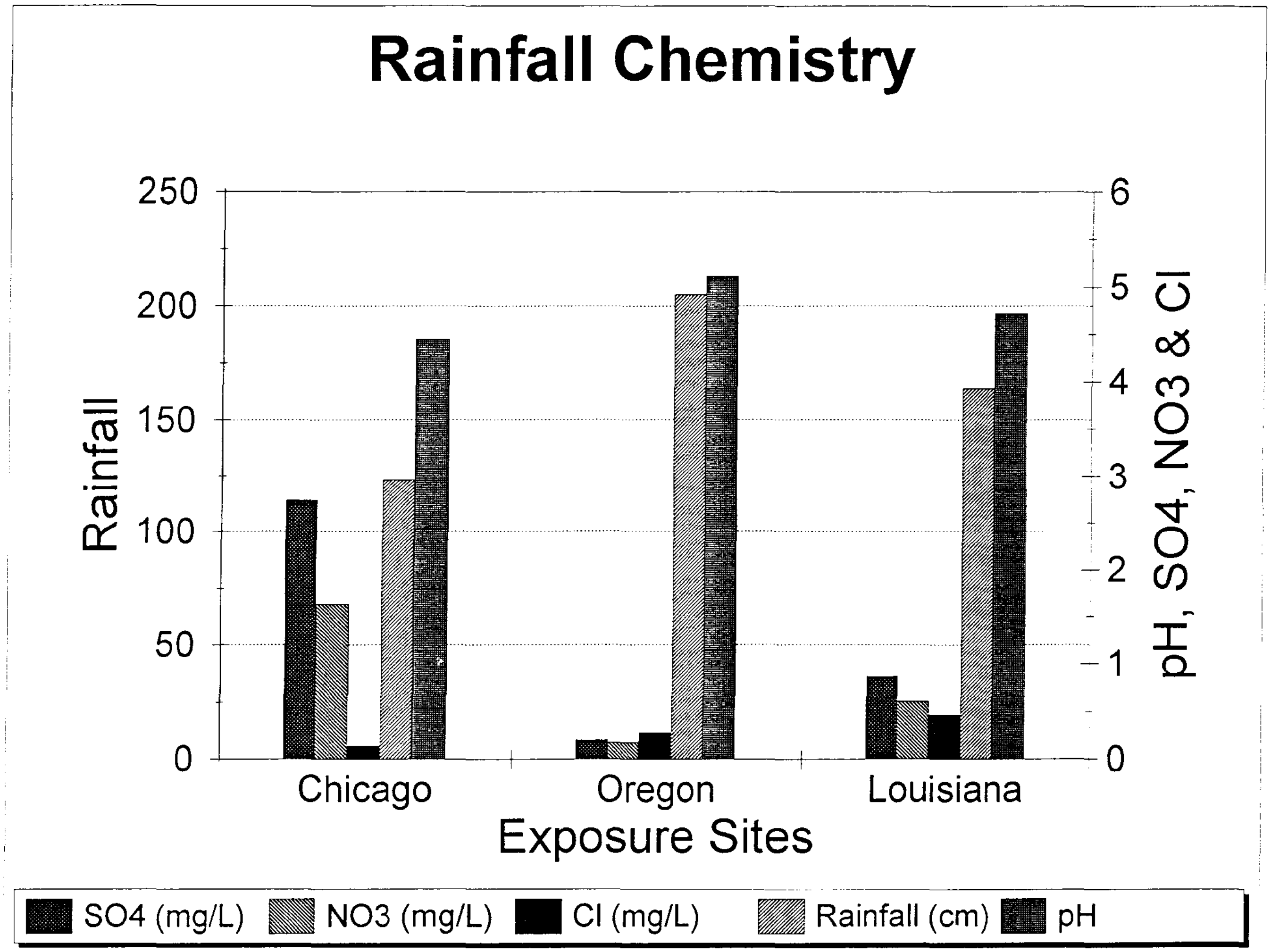

Figure 2. Rainwater data.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Expansion Joint Elimination for Steel Bridges

Author(s) and Affiliation(s):

George Tsiatas, Everett McEwen and William Boardman
Department of Civil Engineering
University of Rhode Island, Kingston, RI

Principal Investigator: George Tsiatas

Sponsor(s): Rhode Island Department of Transportation and the Federal Highway Administration

Research Start Date: August 1, 1995
Expected Completion Date: December 31, 1996

Research Objectives:

To investigate methods for expansion joint elimination of existing bridges; develop finite element based analytical models for the most promising joint elimination schemes and investigate the range of applicability; study four typical Rhode Island highway bridges and evaluate alternative methods for joint elimination of these bridges; and to develop a field monitoring program of rehabilitated Rhode Island bridges.

Expected Products or Deliverables:

A report detailing all aspects of the analytical study which will contain recommendations for expansion joint elimination during rehabilitation and a proposal for a field monitoring program.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

George Tsiatas received his BSCE from the Technical University of Athens (Greece) in 1979 and his MS and Ph.D. in civil structural engineering from Case Western Reserve University in 1982 and 1984, respectively. Dr. Tsiatas is Associate Professor of Civil Engineering at the University of Rhode Island. He specializes in infrastructure assessment and monitoring, structural reliability, and structural dynamics. His research in these areas has been funded by the National Science Foundation, the Rhode Island and Washington State Departments of Transportation, the Federal Emergency Management Agency and the U.S. Forest Service. He is a member of the ASCE technical committees on Safety of Bridges (Structural Division), Probabilistic Methods (Mechanics Division), and Dynamics and Controls (Aerospace Division).

Everett McEwen received his BSCE in 1954 from the University of Rhode Island, his MSCE in 1956 from the University of Illinois, Urbana and his Ph.D. in civil engineering in 1964 from Rensselaer Polytechnic Institute. Dr. McEwen is Professor of Civil Engineering at the University of Rhode Island. He has a strong interest in structural dynamics, numerical methods, and highway bridge performance, as well as in the design of steel and concrete structures. His research has been funded by the National Science Foundation, the Rhode Island Department of Transportation, the New England Transportation Consortium and several private companies. He has been involved in various projects related to bridge design and rehabilitation, historic bridge preservation, and earthquake design of bridges and other structures.

William Boardman is a Ph.D. candidate in civil engineering at the University of Rhode Island. He received his MS in mechanical engineering in 1990 and his BSCE in 1987, both at the University of Rhode Island. Mr. Boardman is also currently employed as Bridge Engineer with the Rhode Island Department of Transportation.

Principal Investigator's Telephone Number: (401) 874-5117
Facsimile Number: (401) 874-2786
E-mail Address: tsiatasg@egr.uri.edu

EXPANSION JOINT ELIMINATION FOR STEEL BRIDGES

George Tsiatas[1], Everett McEwen[2], and William G. Boardman, P.E.[3]

[1]Associate Professor, [2]Professor, [3]Graduate Student
Department Of Civil and Environmental Engineering
University of Rhode Island, Kingston, RI 02881

Research Objectives

The goal of this research project is to evaluate the rehabilitation of steel highway bridges by eliminating expansion joints. Various methods of joint elimination are investigated including deck only continuity and partial to full girder continuity. Analytical studies using finite elements are performed to evaluate the various joint elimination techniques. Four bridges which have been scheduled for rehabilitation in Rhode Island are used for the analytical studies. A field monitoring program is designed to be incorporated in the rehabilitated bridges in order to follow the actual performance. It is expected that reduction or elimination of bridge joints will lessen the amount of corrosion to beams, bearings, and substructure components. This reduction lessens the amount of construction traffic delays, cost of bridge repair and replacement, and provides a smoother riding surface.

Research Approach

Background: Expansion joints have been extensively used in highway bridges to accommodate superstructure displacements and rotations due to live loads, creep, shrinkage, support settlements and temperature induced expansion or contraction. Typically, the girders of a multi-span bridge are simply supported within each span and specially designed sealed expansion devices are inserted between them. This type of arrangement provides design simplicity and construction speed. Unfortunately, the expansion joints have been found to be more likely than other bridge components to be damaged or to cause damage. For example, water and deicing chemicals eventually go through and penetrate the substrate of the bridge causing deterioration of main girder ends, bearings, and other key components. In addition, expansion joints can cause uncomfortable riding, increased impact loads and increased maintenance.

Because of the adverse effects, there is a trend to build continuous new bridges with no expansion joints except perhaps at the abutments in the case of longer spans. Such integral bridges have been used with success up to 400 ft in the case of steel bridges and up to 800 ft in the case of concrete bridges. However, very little work has been done to rehabilitate existing bridges by eliminating the expansion joints. In addition, most of the work has concentrated in prestressed concrete type of girders in which case displacements and rotations at girder ends are small.

In order to eliminate expansion joints, the displacements and rotations at the end of the girders must somehow be absorbed or restrained. There have been various techniques proposed or implemented on a limited scale for the elimination of the joints. According to the predominant approach, no connection is made between the two adjacent beams. The joints are simply surfaced over by pavement material or the concrete deck is stripped in a section around the joint and after placement of negative reinforcement the deck is refinished. This method, which occasionally is referred to as "buried joints," has been used with success from the Tennessee Department of Transportation and most recently by the Connecticut Department of Transportation.

In Japan buried joints are classified as two types: a) displacement absorbing in which flexible pavement materials absorb movements (rubberized asphalt) and b) displacement dispersing which disperse the movements in a wider area. Buried joints are most suitable for prestressed concrete girders in which the displacements are not large. In the case of steel bridges, elongations due to temperature rise and rotations due to live loads can be significant and can lead to reflective cracking of the pavement at the location of the old joint.

For steel bridges and longer span concrete bridges some connection may be needed between the two adjacent girders. The connection can be complete by welding plates through the webs and flanges and by introducing deck continuity but this method is costly. A partial connection has been suggested to economically eliminate the joints. This is accomplished by deck continuity and a welded plate on the top flange. In this way expansion/contraction is restrained but some rotation is allowed.

Survey Results: A survey of Departments of Transportation in the United States and Canada has been made in order to evaluate the current status of bridge deck elimination. Out of the sixty-four agencies that have responded to the survey, twenty have been involved in some way with bridge joint eliminations. According to the survey, approximately 500 steel bridges have been made continuous. The exact methods of continuity used for retrofit varied from agency to agency, but most of these agencies have tried only one continuity scheme within their region. Most of the conversions have been constructed within the last five years. The most popular method of connection is deck only, ten agencies. The second most popular scheme, six agencies, was connecting deck, top, and bottom flange. Utah has completed approximately 200 conversions, limiting the maximum span length for deck only connection to 300 feet. Not as a surprise, most agencies use continuous deck joints to eliminate joints without much regard to improving the live load carrying capacity. Specific details and construction plans of the last bridge retrofitted were obtained along with any standard details that have been developed. These plans and details will be compared.

Analytical Investigation: The State of Rhode Island is in the process of rehabilitating several bridges including expansion joint elimination. Four specific bridges have been selected for investigation during the present study. These include the Garden Street Bridge #547, Pine Street Bridge #548, Hartford Avenue-West Bridge #608 and Broad Street Bridge #657. A typical case is Garden Street Bridge which is a composite steel stringer bridge with concrete deck and bituminous overlay. It consists of four simple spans, 27 ft 59 ft, 59 ft and 36 ft. In all bridges the existing deck joints are in disrepair and adjacent concrete is deteriorated. The expansion bearings are corroded and frozen. The concrete piers and abutments are deteriorated and spalling. In this particular bridge, the intermediate expansion joints (over piers 1 and 3) will be eliminated using a semi-continuous scheme made up of a steel top flange splice plate and continuous deck. A length of about five feet of concrete deck will be replaced at these two locations. In addition, existing fixed bearings at these locations will be replaced with elastomeric bearings.

As a first step of the analytical investigation, the above bridges are modeled using a simplified finite element model incorporating beam elements to model the steel beams, and spring elements to model the continuous joint scheme. In order to perform a parametric investigation various springs are used to model the continuous deck reinforcement (top and bottom mat), a top flange connection, a bottom flange connection as well as a full connection between the adjoining beams. As a second step, further analytical investigations will be made using detailed solid models of the area around the expansion joints in order to determine the exact behavior of the rehabilitated joints. The studies will include loads due to HS-25 trucks as well as temperature variations.

A parametric investigation of various bridge geometries will also be undertaken in order to establish the validity of the various joint elimination techniques and in particular the span length of applicability. In order to generate a large number of bridge geometries, the AISI Short Span Steel Bridge Plans will be used to determine realistic section properties of the steel bridge cross sections.

Field Monitoring: As part of the project an instrumentation plan is developed for two of the bridges to be rehabilitated. The plan calls for installation of strain gages to monitor the strains at the steel girders, the splice plates, the reinforcing steel in the deck, and the concrete strains. A number of tiltmeter gages will be used to measure girder rotations to assess the influence of the introduced continuity. Overall longitudinal movements will be monitored and a number of thermocouples will be used to monitor temperatures in the concrete deck and the steel girders. In addition, material testing will be performed in order to establish the as built material properties.

Expected Products:

It is expected that this research project will develop a concise procedure for deck joint elimination during major bridge rehabilitation. The analytical studies verified and calibrated using continuous field monitoring will provide information on the range of applicability, including maximum span lengths, of the various continuity schemes. With this information, bridge engineers will be better able to recommend a method of retrofit that is cost effective and provides an extended bridge service life.

Preliminary Results:

Fig. 1 includes preliminary results indicating the top mat rebar stress for various joint continuouity schemes. It is evident that the rebar stress becomes maximum when the deck and bottom beam flange are made continuous. It is interesting to notice that for this particular bridge geometries connecting the top flanges induces higher reinforcement stresses than leaving only the deck continuous. Fig. 2 shows the calculated deck crack width for the various continuity schemes. As expected, the highest potential crack size would occur in the first two cases, i.e. deck only continuity, and deck and top flange continuity. It is interesting to note that connecting the top flanges of the adjacent beams does not significantly reduce potential cracking. For this particular bridge geometry top flange continuity does not provide any benefits and as a matter of fact it increases the stresses in the deck reinforcement. Another point to be made is that with deck only continuity very little live load stresses are transferred from one span to the other. It appears that the procedure can solve the leaking joint problem but cannot provide for an increase in bridge capacity unless the joint is made fully continuous..

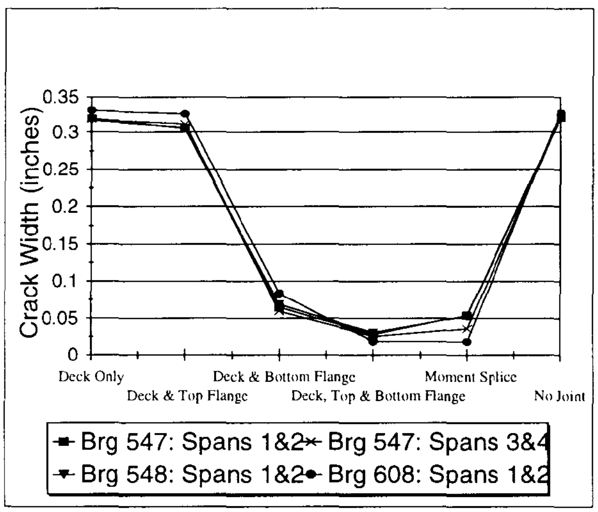

Figure 1 Top Rebar stress

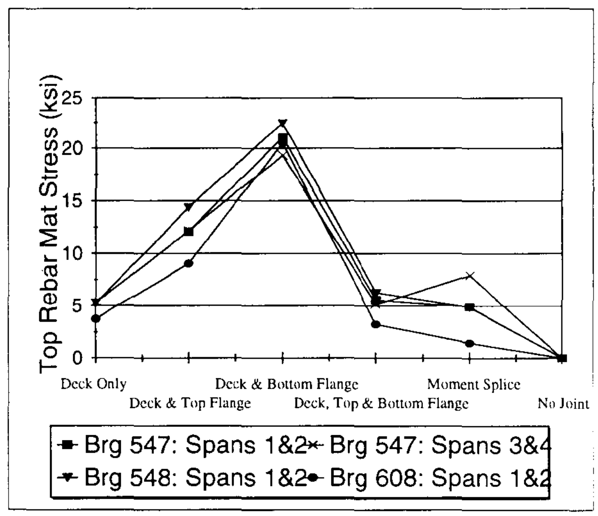

Figure 2 Deck Crack Width

Acknowledgment

Funding for this research project has been provided by the Rhode Island Department of Transportation (RIDOT) and the Federal Highway Administration. Mr. Colin Franco, and Mike Savella (RIDOT) have provided helpful information including bridge designs and blueprints. Their support is gratefully appreciated.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Bridge Girders with Corrugated Webs

Author(s) and Affiliation(s):

Mohamed Elgaaly
Civil and Architectural Engineering Department
Drexel University, Philadelphia, PA

Principal Investigator: Mohamed Elgaaly

Sponsor(s): National Science Foundation

Research Start Date: July 1991
Expected Completion Date: July 1996

Research Objectives:

The objectives of this study are to investigate the behavior of girders with corrugated webs subjected to shear and/or bending up to failure, develop simple methods to determine the ultimate capacity under these loading conditions, and determine the fatigue strength of girders with corrugated webs.

Expected Products or Deliverables:

Guide to design criteria.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Mohamed Elgaaly has 39 years of experience in teaching, research, and work in the engineering industry. He received a Ph.D. in structural engineering from the University of Michigan in the early 1960's, and has numerous publications in technical journals, mostly related to steel plate structures. He has authored chapters in books on plate and box girders. Dr. Elgaaly is a Fellow of ASCE and is a member of the Structural Stability Research Council Executive Committee and Chairman of the Task Group on Plate and Box Girders. He is currently Professor of Structural Engineering at Drexel University's Civil and Architectural Engineering Department. Dr. Elgaaly is a registered Professional Engineer in several states.

Principal Investigator's Telephone Number: (215) 895-1317
Facsimile Number: (215) 895-1363
E-mail Address: elgaalym@duvm.ocs.drexel.edu

BRIDGE GIRDERS WITH CORRUGATED WEBS

Mohamed Elgaaly, Sc.D., PE

Civil And Architectural Engineering Department
Drexel University, Philadelphia, PA, 19104

Research Objectives:

Economical design of plate and box girders normally requires thin webs. The conventional welding of stiffeners to allow the use of thin webs results in higher cost and reduced fatigue life. The use of corrugated webs is a possible way of achieving adequate out-of-plane stiffness without using stiffeners. Recently, two bridges with corrugated webs were built in France. In addition to less fabrication cost and longer fatigue life; the prestressing force in the concrete deck did not dissipate unnecessarily in the steel web due to the corrugation. The objective of this research is to examine the behavior of girders with corrugated webs subjected to shear or bending up to failure, and to develop simple methods to determine the ultimate capacity under these loading conditions; another objective is to determine their fatigue strength. A review of work by others, related to girders with corrugated webs, can be found in a paper by Elgaaly and Dagher (1990)

Research Approach:

Tests were performed on welded beams with corrugated webs loaded predominantly in shear and in bending. A Finite Element model was developed and a non-linear analysis performed; the analysis was able to depict the test results to a practical degree of accuracy. In this paper, the analytical model will be described and the results presented and compared with the experimental results. A simple approach to calculate the strength of corrugated webs subjected to shear or bending was developed. This approach makes use of local buckling of the flat folds of the corrugated web and/or the global buckling of the corrugated web panel as an orthotropic plate.

Laboratory Tests:

Shear Strength: Forty two tests on twenty one beams were conducted under pure shear; four different corrugation configurations and two thicknesses were used. The webs in the specimens tested were made of 24 and 22 gage material; average thickness of 0.0239" and 0.0299", respectively. The webs were continuously welded to the flanges from one side only, as is the practice in manufacturing these beams. All of the beams tested, failed due to buckling of the web and no rupture of the web at its connection to the flanges was observed. The failures were sudden and was due to buckling. The load carrying capacity drops at the failure load and the specimens exhibited some residual strength after failure. The test program is described in details in a report by Elgaaly and Hamilton (1993) and summarized in a paper by Elgaaly et al (1996).

Bending Strength: A total of six specimens were tested to failure under uniform bending. The test panel was made of 24 gage (0.0239") material in four specimens, using four different corrugation profiles. In the remaining two specimens the test panel was made of 22 gage (0.0299") material with two different corrugation profiles. Failure in all the specimens was sudden and due to yielding of the compression flange and its subsequent vertical buckling into and crippling of the web. From strain gage readings it was noted that the strains in the web are negligible and the strain in the flange increased linearly up to the yield strain of the flange material at failure. The test program is described in details in a report by Elgaaly and Hamilton (1993).

Finite Element Analysis And Results:

Shear Strength: Due to the cost and time associated with testing, and in order to study the effect of other corrugation configurations, the web panel aspect ratio, and the web/flange interaction on the ultimate shear capacity of corrugated webs; one has to resort to numerical analysis using Finite Elements. If Finite Element models of the test specimens can depict the test results to a reasonable degree of accuracy; then the Finite Element method of analysis can be used to conduct parametric studies in order to understand the behavior of corrugated webs with variable dimensions under shear. The eight node thin shell element (S8R5) of ABAQUS was employed to model the corrugated web; three elements across each fold of the corrugation were used. The number of elements along the depth of the panel was determined to keep the aspect ratio of the elements less than four and use a fine mesh in the vicinity of the flanges.

A study was performed on the Finite Element model of one of the test specimens to examine the effect of the magnitude of the specified minimum load increment. The mesh size effect on the results was also investigated. Based on the analysis results and in order to minimize the computational effort, a minimum load increment equals to or less than 0.015 times the anticipated failure load was used. With respect to the mesh size, the difference in the results from the models with three and four elements across the width of one fold is 1.42%. In order to cut down the computational effort without sacrificing the accuracy of the results, three elements across each fold of the corrugation were employed.

The agreement between the analytical and experimental results were satisfactory. The average ratio between the analytical and the experimental results was 1.151. The primary reason that the analytical results were higher than the experimental ones could be the presence of unavoidable out-of-plane initial imperfections in the test specimens. A Finite Element Analysis to study the effect of initial out-of-plane imperfections was performed. Imperfections in the form of a global double sine wave were considered. Better agreement between the analytical and experimental results was obtained when initial out-of-plane imperfections were assumed. Details of the Finite Element Analysis and results can be found in a report by Elgaaly and Seshadri (1996) and a paper by Elgaaly et al (1996).

Bending Strength: A Finite Element model of a corrugated panel subjected to uniform bending was developed; due to symmetry only half the panel was modeled. The vertical edges of the corrugations form nodal lines, and three elements were used across the width of each horizontal and inclined fold of the corrugations. The 8-node quadrilateral (S8R5) and the 6-node triangle (STRI65) thin shell elements, available in ABAQUS, were used to model the corrugated web, the flanges, and the stiffener. To facilitate the solution, the stress-strain relationship was assumed to be bi-linearly elastic perfectly plastic. A non-linear static analysis was performed considering both geometric and material non-linearities. The minimum load increment was kept between 1 and 1.5 percent of the anticipated failure load to insure sufficient accuracy.

In order to verify the Finite Element Model, the test specimens were analyzed. The analytical failure mode in all the specimens was identical to the experimental one, namely yielding of the flanges followed by buckling of the compression flange vertically into and crippling of the web, and the analytical ultimate moment capacities agree very well with the experimental results. The stresses in the web were found to be negligible except for very localized areas of the web adjacent to the flanges; and they are higher within the horizontal folds of the corrugation. The contribution of the web to the moment carrying capacity of the specimens is negligible. In summary, the Finite Element model was able to depict the test results to an excellent degree of accuracy; furthermore, it provided a more detailed picture of the stresses in the web.

Parametric Studies: The analytical model was utilized to perform parametric studies. The parameters considered were: the thicknesses and yield stresses of the flange and web material, the web panel aspect ratio, and different corrugation configurations were considered. The test results and the results from the Finite Element analysis, helped in establishing simple methods to calculate the ultimate bending and shear capacities of girders with corrugated webs.

Ultimate Shear Strength:

Based on Local Buckling: As stated earlier, the mode of failure is local and/or global buckling of the web. In the local buckling mode, the corrugated web acts as a series of flat plate sub-panels that mutually support each other along their vertical edges and are supported by the flanges at their horizontal edges. The local buckling stress was calculated for all the specimens, assuming simply supported or fixed boundaries along the vertical sides of the folds and fixed boundaries along the horizontal sides and the average value was used. The average ratio between the failure load obtained from the finite element analysis and the local buckling stress analysis described above was calculated to be 1.015 for all the test specimens

Based on Global Buckling: When global buckling controls, the buckling stress can be calculated for the entire corrugated web panel, using orthotropic-plate buckling theory. The global buckling stresses for the test specimens made of dense corrugation were calculated using the buckling formula for the orthotropic plate The average value of the ratio between the Finite Element analysis results and those obtained from the orthotropic plate theory is 1.067.

Local Or Global Buckling: As stated above, local buckling occurs in the case of coarse corrugations and global buckling takes place in the case of dense corrugations. The classification of what is coarse and what is dense is difficult to quantify. Hence, it is suggested that the capacity should be calculated based on local as well as global buckling, and the smaller of the two values controls.

Ultimate Bending Strength:

The contribution of the web to the ultimate moment capacity of a beam with corrugated web is negligible, and the ultimate moment should be based on the flange yield stress. The stresses in the web due to bending are equal to zero except very close to the flanges where the web is restrained.

Fatigue Strength:

Korashy and Verga (1979), tested 4 conventionally stiffened hybrid beams, 3 conventionally stiffened homogeneous beams, 6 wavely stiffened hybrid beams, and five wavely stiffened homogeneous beams under repeated loads. In the wavely stiffened beams, stiffeners were replaced by one-wave corrugation formed in the web at the location of the stiffener, so the web is not continuously corrugated. The beams depth was 270 mm and the span was 2740 or 1700 mm. The flange yield stress was 350 or 240 N/mm^2, and the web yield stress was 240 N/mm^2. The tension flange in all beams was 110 mm wide and 10 mm thick, and all the beams were loaded by two equal loads applied at about the third points of the span. The tests were conducted up to failure, with approximately 500,000 to 1,800.000 cycles at a speed of about 300 cycles / min, and a minimum-to-maximum stress ratio in the range of 0.085 to 0.227. There was no significant difference between the fatigue strength of the hybrid and homogeneous beams. The wavely stiffened beams showed approximately 50% higher fatigue strength than the stiffened beams when the stiffeners were welded to the tension flange. In the case where the stiffeners were cut off short of the tension flange the increase was approximately 25%.

Harrison (1965), did fatigue tests on two beams with corrugated webs: the depth of the web was 24" and its thickness was 0.25", and the flanges were 8" wide and 5/8" thick. The simply supported span in Beam 1 was 13' and the two test loads were applied 1.5' on each side of the span centerline. In Beam 2, the span was 12'-4.5" and the loads were applied 16.5" on each side of the span centerline. The corrugations were sinusoidal and 3" deep in both beams. The corrugation wave length was 24" in Beam 1 and 16.5" in Beam 2. The webs were welded to the flanges using semi-automatic submerged arc process. The fatigue strength of these two girders was comparable to that of the wavely stiffened beams tested by Korashy and Verga. These two beams, however, had larger flange cross sectional area, resulting in longer crack propagation period.

Currently, the author is conducting fatigue tests on six identical girders with trapezoidally corrugated webs. The web is 400 mm deep and 2 mm thick and the flange is 120 mm wide and 10 mm thick; the girders are 4100 mm long. The corrugations are 106 mm deep, the wave length is 500 mm, and the inclined folds are at 45 degrees. The tests will be conducted up to failure for different values of stress range and different loading conditions at a speed of 300 cycles/min. This work is currently in progress; it constitutes preliminary investigation and no results are available to report at this stage.

Summary And Conclusions:

Beams and girders with corrugated webs have been used all over the world in buildings and bridges. The beams manufactured and used in Germany have trapezoidal corrugations with a web thickness that varies between 2 and 5 mm. The web depth-to-thickness ratio is in the range of 150 to 260. The corrugated webs of two bridges built in France were also trapezoidal and were 8 mm thick; the web depth-to-thickness ratio was in the range of 220 to 375.

Tests conducted by the author indicate shear failure due to buckling and bending failure due to flange yielding with no appreciable contribution from the web. Finite Element analysis of the test specimens was able to depict the experimental results to a very good degree of accuracy. For corrugated webs it was found that the shear strength can be calculated, to a good degree of accuracy, using the buckling stress formulae for flat isotropic or orthotropic plates. It has to be noted that the studies conducted to date were limited to trapezoidal corrugations, where the inclined and horizontal folds are about equal in length.

Studies to determine the fatigue strength of such girders are very scarce; only two tests were conducted on two girders with continuously corrugated webs. Preliminary fatigue tests by the author on six girders with corrugated webs are currently in progress.

Acknowledgments:

The work presented in this paper is part of a study conducted under Grant No. MSS-9020559 from the National Science Foundation; Dr. K. P. Chong is the program director. The support provided by Lincoln Electric Company, under the leadership of Omer Blodgett and Duane Miller is very much appreciated. Most of the Finite Element Analysis was conducted on the Power Challenge Super Computer at the University of Illinois Computing Center, the support provided by Dr. A. Fouad of the center must be acknowledged.

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FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: A Passive Fatigue Life Indicator for Highway Bridges

Author(s) and Affiliation(s):

Douglas Thomson and Gopal Samavedam
Foster Miller, Inc., Waltham, MA
William Wright, Turner-Fairbank Highway Research Center
Federal Highway Administration, McLean, VA

Principal Investigator: Douglas Thomson

Sponsor(s): Federal Highway Administration

Research Start Date: September 21, 1995
Expected Completion Date: February 1, 1997

Research Objectives:

The primary objective of this study is to develop and demonstrate a low cost, unpowered system for tracking the fatigue loading of highway bridges. This system will serve as a useful tool for bridge owners in the assessment of fatigue and the optimal deployment of inspection and maintenance resources. The twin coupon fatigue life indicator (TCFLI) system works on the principle that stable propagation of an existing "precrack" in a coupon can be directly related to the applied load and number of cycles. The innovation of this system is in the use of two coupons of different metals which are attached directly to the structure and thus experience the same loading histogram. By simply measuring the crack lengths in the coupons at the beginning and end of any period, the average load level and number of cycles during that period is determined since the cracks will propagate at different rates in the two metals.

Expected Products or Deliverables:

The result of this project will be a prototype TCFLI system which is ready for deployment in large scale field demonstration tests. The TCFLI will be a low cost (< $200) passive device which can provide bridge loading history information. Simple to deploy and cost effective, the TCFLI can be mounted on all bridges within an owner's inventory to provide relative usage information between similar bridges and to track changes in the traffic patterns for a particular bridge.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Douglas T. Thomson is a Senior Engineer and Program Manager in the Structures and Transportation Technology Division at Foster Miller. Mr. Thomson has led numerous research programs to assess the structural integrity of aging aircraft, conventional and high speed rail systems, and military and highway bridges. He has directed laboratory and field test programs, developed instrumentation systems, and conducted numerous evaluations of transportation structures. Prior to joining Foster Miller in 1989, Mr. Thomson was a structural test engineer with United Technologies, Sikorsky Aircraft where he led evaluations of numerous aircraft components. Mr. Thomson received a B.S. in Aerospace Engineering from Syracuse University in 1986.

Gopal Samavedam is the Structures and Transportation Technology Division Manager at Foster Miller. Dr. Samavedam has conducted extensive and significant research in the theoretical and experimental analysis of fatigue and fracture mechanics. He has an extensive background in transportation structures and his fields of specialization include structural mechanics, vibrations, and optimization. He has been primarily specializing in the area of railroad track and vehicle design, analysis and testing for the last 15 years. Prior to joining Foster-Miller in 1980, Dr. Samavedam served as Principal Scientific Officer at the Research and Development Division of British Rail. Dr. Samavedam received a B.S. in Mechanical Engineering from Andhra University, Waltair and an M.E. (applied mechanics) from Calcutta University. He was awarded a Ph.D. (elasticity) by the Indian Institute of Technology.

William Wright is a staff structural engineer with the Federal Highway Administration's Turner-Fairbank Highway Research Center in McLean, Virginia.

Principal Investigator's Telephone Number:	(617) 890-3200
Facsimile Number:	(617) 890-3489
E-mail Address:	dthomson@foster-miller.com

A PASSIVE FATIGUE LIFE INDICATOR FOR HIGHWAY BRIDGES

Douglas Thomson[1], Gopal Samavedam[1], William Wright[2]

[1]Foster Miller, Inc., Waltham, MA 02154
[2]Turner-Fairbank Highway Research Center, McLean, VA 22101

Research Objectives

A twin coupon fatigue life indicator (TCFLI) system for tracking fatigue of bridge structures is being developed by Foster Miller under contract to the Federal Highway Administration. The primary objective of this program is to develop a low cost, unpowered system which will serve as a useful tool for bridge owners in the assessment of fatigue and the optimal deployment of inspection and maintenance resources. The ongoing program is conducted to define the proper system configuration for highway bridges, develop the attachment methodology and demonstrate performance through laboratory and field testing.

Research Approach

Background: The degradation of highway bridges due to vehicle loading and environmental conditions is a major concern for state and local transportation agencies. This maintenance burden is substantially increased by the fact that little or no loading history data is available for the majority of these structures. The largest number of structurally deficient bridges in the National Bridge Inventory are of steel construction. These bridges in particular are susceptible to fatigue damage and cracking due to vehicle loading.

Several options exist for monitoring the loading history of bridge structures. Superior accuracy and recording of the bridge loading histogram can be achieved with electronic strain gauge systems. However, this accuracy and functionality comes at a much higher cost and usually with a requirement for line power if long term monitoring is to be done. Vibration and acoustic emmissions monitoring systems may overcome the long term stability issues but they typically come at very high cost and require individual calibration of each bridge. Clearly, while there is an abundance of equipment available for this application, the problem of monitoring bridge fatigue is not adequately addressed because these complex, expensive systems won't likely come into widespread, continuous use. By contrast, the inexpensive TCFLI is easily installed and calibrated to provide accurate loading history information.

The TCFLI system was originally developed for use on mobile bridges for the US Army. This application was significantly different from highway bridges as mobile bridges operate at very high stress levels for a limited life. The system was developed and successfully demonstrated in field tests of tank crossings over the Armored Vehicle Launched Bridge (AVLB) at Aberdeen Proving Grounds. The current program will adapt the device for operation in a low stress, high cycle environment

TCFLI Concept: The TCFLI system works on the principle that stable propagation of an existing "precrack" in a coupon can be directly related to the applied stress and number of cycles. The basic equation used here to define crack growth is the Paris Law.

$$\frac{da}{dN} = C\Delta K^n \text{ , where, } \Delta K = \Delta\sigma\sqrt{\pi a}$$

The innovation of this system is in the use of two coupons of different metals (hence different material constants C,n) which are attached directly to the structure and thus experience the same loading histogram. Integrating the above equation, an expression for determining the number of cycles associated with a change in crack length can be determined as a function of the applied stress. This equation can be written for each coupon. Since, for coupons with the same material moduli, the applied stress and number of cycles are the same, the two coupon equations can be combined and the applied stress level directly determined from the crack extension data from the two coupons.

Thus, by simply measuring the crack lengths in the coupons at the beginning and end of any period, the average load level and number of cycles during that period is determined since the cracks will propagate at different rates in the two metals. Therefore, a weighted average stress level and number of cycles can be determined for any increment of loading history for which the initial and final crack lengths have been recorded.

Program Summary

The TCFLI development and demonstration program is being conducted to i) Optimize the system configuration and coupon materials; ii) Develop a procedure for mounting the Twin Coupon FLI system in the field; iii) Demonstrate system accuracy through laboratory tests; and iv) Conduct a long term field demonstration of the system. The TCFLI system will be demonstrated in the field beginning in the spring of 1996. Development of the optimized design and demonstration of the field attachment technique are ongoing.

The TCFLI configuration and mounting system present the most significant design challenge toward the development for application to highway bridges. This integrated approach must address several requirements including the following:

- The TCFLI system must be installed without modification of the bridge structure.
- The mounting system must not require interruption of traffic for installation.
- The estimated dead load strain in the bridge must also be applied to the TCFLI system.
- The TCFLI configuration must amplify the expected low strain levels in the bridge

Application of the dead load strains and amplification of the live load strains are required to overcome several potential error sources. Since the operation of the TCFLI is based on crack propagation, the cyclic stress on the coupons due to the live load on the bridge structure must induce a crack tip stress which is sufficiently greater than the threshold for crack propagation. While this threshold could be exceeded at low stress with a long crack, space limitations dictate that it is more practical to amplify the strain as is discussed in the preliminary results. The dead load stress is significant because crack propagation is sensitive to loading ratio (R). The dead load stress also limits crack closure effects thereby reducing crack retardation due to overloads.

Expected Products

The result of this development program will be a prototype TCFLI system which is ready for deployment in large scale field demonstration tests. The TCFLI, when fully realized as a product, will be a low cost (<$200), wholly passive device which can provide bridge loading history information. Simple to deploy and cost effective, the TCFLI can be mounted on all bridges within an owner's inventory to provide relative usage information between similar bridges and to track changes in the traffic patterns for a particular bridge. This information can, in turn, be used to assess the fatigue of various structural details and identify potential problem areas.

The TCFLI will therefore be a useful tool as part of an overall bridge management system and will assist in the optimal deployment of inspection and maintenance resources.

Preliminary Results

A system configuration has been developed to meet the program requirements. This design, which is shown in Figure 1 for the laboratory tests, utilizes a "bonded mounting block" attachment technique. For this approach, the steel blocks are first bonded to the bridge structure. This permits system installation without structural modification or interruption of traffic.

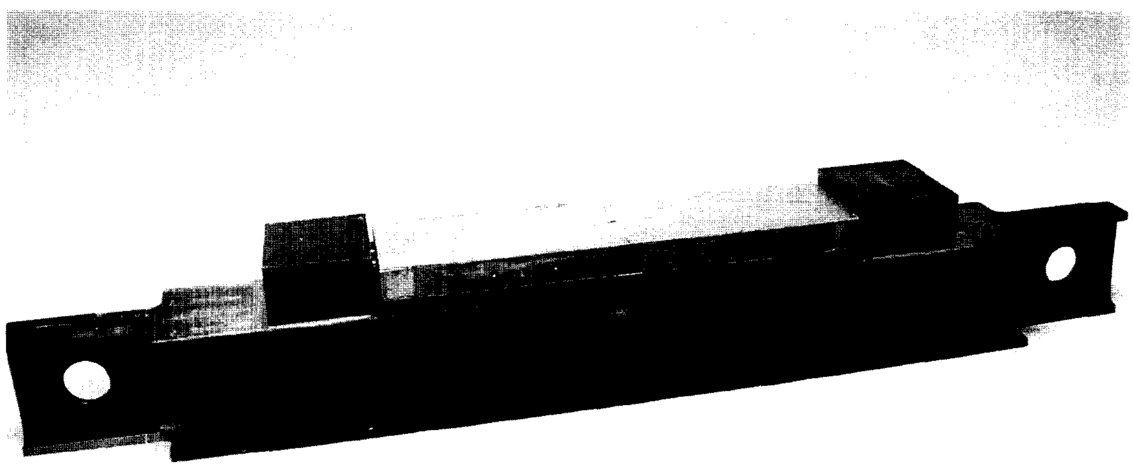

Figure 1: TCFLI Test Configuration

Following curing of the epoxy, the TCFLI system is installed with bolts which pass through the mounting blocks and into tapped holes in the steel stiffener plates. These attachment bolts are torqued to provide the proper dead load stress levels. Finally, the long steel stiffeners provide significant amplification of the live load bridge strain. Disregarding losses in the bolts and epoxy attachments the theoretical strain amplification can be easily estimated by considering the relative lengths and cross-sectional areas. Thus, by selecting materials and adjusting the relative lengths and cross section areas of the stiffener and coupons, the low stress level in the bridge can be amplified to achieve any desired applied stress level in the coupon.

Following this procedure for design of the TCFLI assembly, the laboratory test system shown previously was developed. This assembly utilizes two, precracked aluminum coupons. A steel stiffener is bonded between the coupons on either end of the assembly to leave a small coupon gauge length. Recent tests have demonstrated that the losses in the bolted connections and creep along the bond interfaces between the coupons and the stiffener and between the mounting blocks and the bridge can be minimal.

The results of the tests demonstrate sufficient strain amplification for application to a steel highway bridge which has a maximum live load stress level of at least 2.5ksi. Minor modifications of the design length will permit application for lower stress levels.

Laboratory demonstration of the TCFLI system under representative bridge loading is currently underway. The system will be demonstrated under block and random spectrum loading to verify its ability to indicate a single stress level and number of cycles which is representative of an equivalent cumulative damage.

Two demonstration sites are required for the six month field evaluation of system performance. These sites must provide a high and low live load stress environment. The Tobin Memorial Bridge on US 1 in Boston, shown in Figure2, has been selected as the high stress

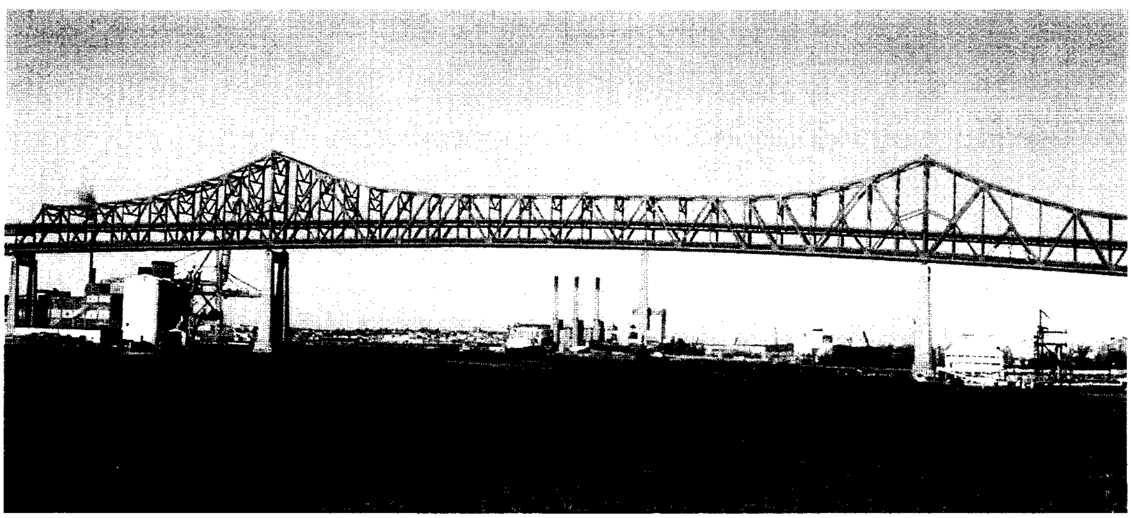

Figure 2: Tobin Memorial Bridge

application. This bridge provides the additional advantage of accurate traffic counts and classifications since it is toll bridge. The bridge is well maintained and frequently inspected by the Massachusetts Port Authority. Thus, fatigue critical areas are identified and monitored. Strain surveys are being conducted by Foster Miller to select optimal TCFLI attachment sites. The second demonstration site will be a standard, steel girder highway bridge with maximum live load stress levels below 2.5ksi. Selection of this bridge is ongoing.

The field demonstration phase of the TCFLI program is expected to begin in May 1996 with installation of the system on the Tobin Memorial Bridge. Installation on the selected highway bridge is also planned for May. Preliminary results are expected to be available for the 4th National Workshop on Bridge Research in Progress.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS RESEARCH ABSTRACT

Title: Vincent Thomas Bridge Monitoring Tests

Author(s) and Affiliation(s):

Ahmed M. Abdel-Ghaffar, Sami F. Masri and Robert L. Nigbor
Civil Engineering Department
University of Southern California, Los Angeles, CA

Principal Investigator: Robert L. Nigbor

Sponsor(s): University of Southern California and the California Strong Motion Instrument Program

Research Start Date: November 1995
Expected Completion Date: December 1996

Research Objectives:

The existing earthquake recording system on the Vincent Thomas Suspension Bridge in Los Angeles was temporarily augmented with a high dynamic range digital monitoring system in late 1995. Data collected from this temporary deployment are currently being analyzed. Objectives of this study are to demonstrate high dynamic range and real-time, remote data monitoring capabilities; collect and analyze baseline ambient vibration data prior to seismic retrofit of the bridge; and study time variation of structural vibration and extracted modal parameters for evaluation of long-term structural health monitoring methodologies. A permanent installation is being proposed to provide a test bed for long-term structural health monitoring.

Expected Products or Deliverables:
A preliminary report has already been produced ("Preliminary Report on the Vincent Thomas Bridge Monitoring Test," USC Civil Engineering Report M9510, December 1995) detailing the measurements and preliminary data analysis. Modal parameters are being provided to the seismic retrofit design team for use in finite element model verification. These data, especially the temporal variation data, are being used to test the robustness of system identification and damage detection algorithms under development for structural health monitoring applications.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Ahmed M. Abdel-Ghaffar is Professor of Civil Engineering at the University of Southern California. He specializes in structural dynamics and earthquake engineering, with special emphasis on long-span bridge dynamics. He has been involved with research on the Vincent Thomas Bridge since his graduate studies at California Institute of Technology in the 1970's.

Sami F. Masri is Professor of Civil Engineering at the University of Southern California. He specializes in structural mechanics and structural dynamics, with emphasis on system identification, structural health monitoring, and structural control.

Robert L. Nigbor is Research Assistant Professor of Civil Engineering at the University of Southern California. He specializes in experimental structural dynamics and earthquake engineering. He is also president of Agbabian Associates, Inc., where he is involved in the implementation of structural health monitoring technology.

Principal Investigator's Telephone Number:	(818) 398-2047
Facsimile Number:	(818) 398-1723
E-mail Address:	nigbor@mizar.usc.edu

VINCENT THOMAS BRIDGE MONITORING TESTS

A.M. Abdel-Ghaffar, S.F. Masri, and R.L. Nigbor
Civil Engineering Department, University of Southern California, Los Angeles, CA 90089

Research Objectives

The existing earthquake recording system on the Vincent Thomas Suspension Bridge in Los Angeles was temporarily augmented with a high dynamic range digital monitoring system in late 1995. Data collected from this temporary deployment are currently being analyzed. Objectives of this study are (1) to demonstrate high dynamic range and real-time, remote data monitoring capabilities; (2) to collect and analyze baseline ambient vibration data prior to seismic retrofit of the bridge; and (3) to study time variation of structural vibration and extracted modal parameters for evaluation of long-term structural health monitoring methodologies. A permanent installation is being proposed to provide a test bed for long-term structural health monitoring.

Research Approach

Background: The Vincent Thomas Bridge in Los Angeles has been instrumented by the California Strong Motion Instrumentation Program (CSMIP) for more than 10 years. Existing instrumentation consists of 26 accelerometers and a central analog-film recording system. The sensors are installed on the substructure and the superstructure to measure vertical, lateral, longitudinal and torsional input and response; Figure 1 shows the bridge and the sensor locations. Significant motions have been recorded for the October 1, 1987 Whittier Earthquake, the January 17, 1994 Northridge Earthquake, and several other earthquakes and aftershocks.

Analyses of the earthquake response and previous ambient vibration studies in 1975-1976 by Abdel-Ghaffar have already been used in the evaluation, improvement, and verification of computer models of the bridge structure. The dynamic response of this bridge is certainly very well documented and studied.

Caltrans is currently in the design phase of the seismic retrofit of the Vincent Thomas Bridge. There will likely be significant work done to both the foundations and superstructure of the bridge to increase its seismic capacity. This retrofit work will result in changes to the dynamic properties (mode frequencies, damping ratios, and shapes).

The combination of existing sensors, a well studied and documented structure, and upcoming known structural changes (a kind of damage in reverse) makes the Vincent Thomas Bridge an excellent test bed for structural health monitoring.

Measurement Program: On November 3, 1995 the temporary monitoring system shown in Figure 2 was installed adjacent to the existing earthquake recorders. Installation and operation of this system was a collaborative effort between the University of Southern California and the staff of

CSMIP. Signals from various sensors of the earthquake recording system were monitored by the temporary system until its removal on December 5. The data recorder had 19-bit resolution, which allowed recording of low-level ambient vibrations without additional amplification. The recorder's remote communication capability through a cellular telephone link allowed remote control of the system. This was used to record data for various times of the day and various channel combinations. More than 80 Mb of data were recorded during the one month field phase of this project, including a complete suite of ambient vibration data from all channels and data from selected channels at different times of the day and week.

The recorder had an output for a real-time digital data stream. This could be transmitted over the cellular communications system to a remote computer for real-time analysis. This capability was demonstrated, although the cellular telephone link proved to be unreliable for continuous real-time monitoring; a direct telephone line (or an IDSN link) would have allowed continuous real-time structural vibration monitoring.

Data Analysis: Data from this temporary monitoring project are being analyzed to provide:

- Modal parameters (frequencies, dampings, shapes) for the Vincent Thomas Bridge
- Temporal variation of spectra and modal parameter estimates
- Correlation of temporal variations with traffic and environmental conditions
- Comparison with past strong-motion, ambient, and computer model results

Expected Products or Deliverables

A preliminary report has already been produced ("Preliminary Report on the Vincent Thomas Bridge Monitoring Test", USC Civil Engineering Report M9510, December 1995) detailing the measurements and preliminary data analysis. Modal parameters are being provided to the seismic retrofit design team for use in finite element model verification.

These data, especially the temporal variation data, are being used to test the robustness of system identification and damage detection algorithms under development for structural health monitoring applications. Finally, a permanent installation is being proposed to provide a test bed for long-term structural health monitoring.

Preliminary Results

Figure 3 shows the measured ambient vibration spectra for three sensor locations (15, 16, and 17) on the Vincent Thomas Bridge. Refer to Figure 1 for sensor locations. These spectra are extremely noise-free because of the high dynamic range of the instrumentation. The lowest mode of vibration is at 0.16 Hz (6 seconds). Figure 4 shows spectra for sensor location 15 at three different times of the day. Although most of the frequencies are stable, there is significant difference in the troughs of the spectra (which could affect damping estimates or curve-fitting methods for system identification) and in some of the higher frequency peaks.

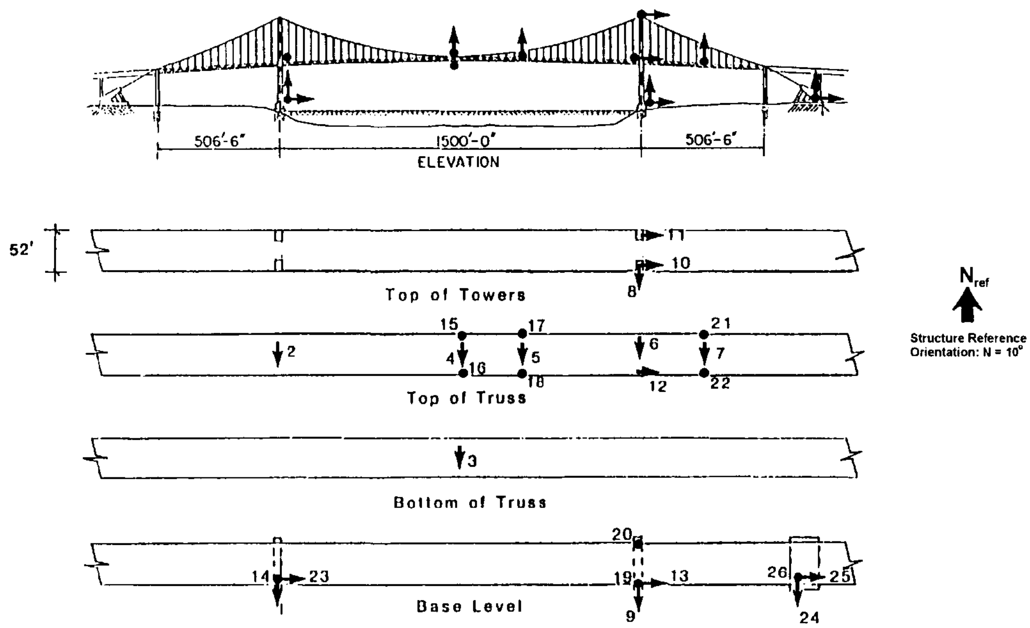

Figure 1. Accelerometer Locations

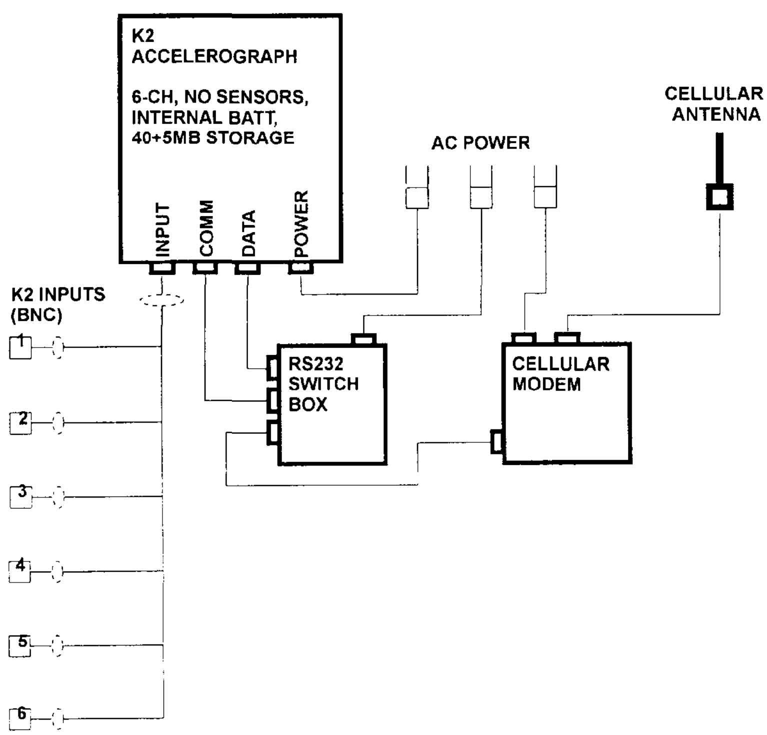

Figure 2. Temporary Digital Recording System

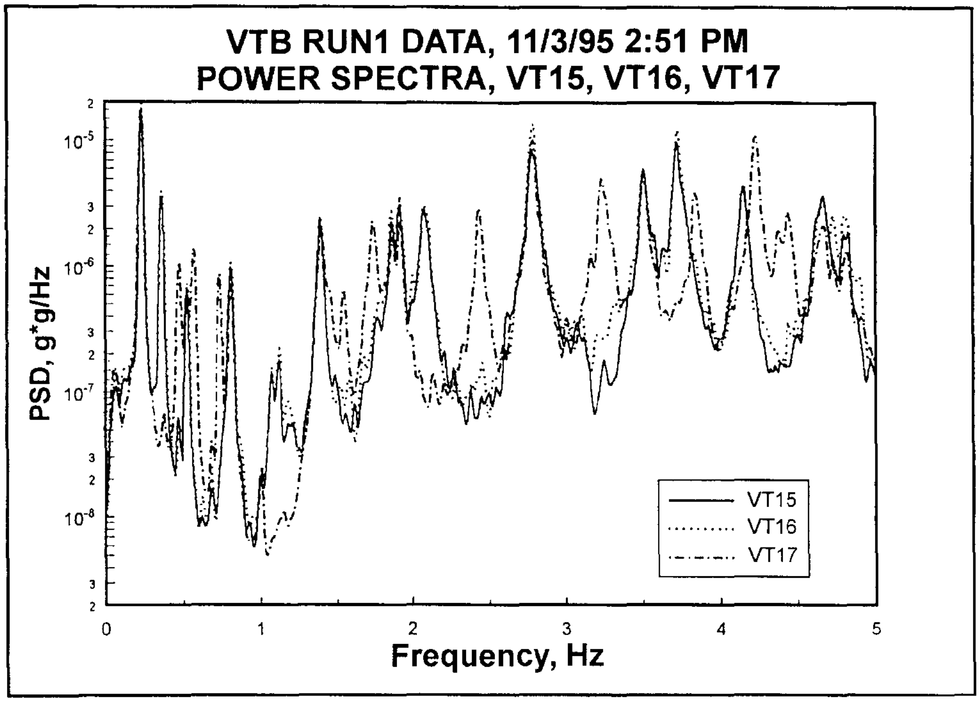

Figure 3. Measured Spectra at Different Locations

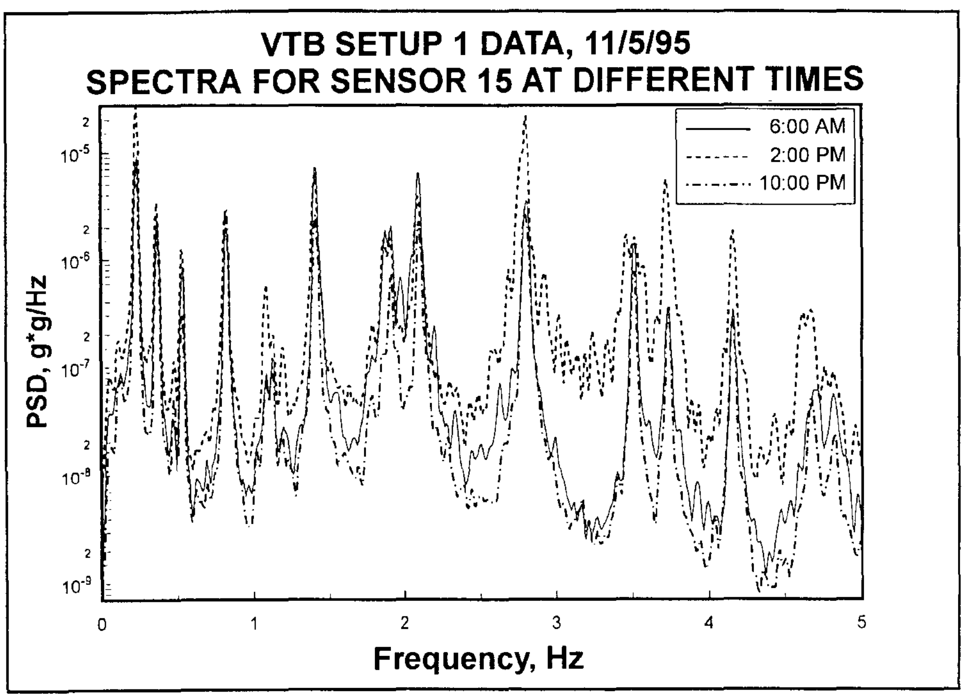

Figure 4. Measured Spectra at Different Times

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Research in Progress on Steel Bridges at University of Washington

Author(s) and Affiliation(s):

Charles W. Roeder and Gregory MacRae
Department of Civil Engineering
University of Washington, Seattle, WA

Principal Investigator: Charles W. Roeder and Gregory A. MacRae

Sponsor(s): Washington State Department of Transportation[1] and American Iron and Steel Institute[2]

Research Start Date: October 1995[1] and December 1995[2]
Expected Completion Date: December 1997[1] and Spring 1997[2]

Research Objectives:

Two major steel bridge studies are currently underway:

(1) Fatigue: This research study will establish the cause of existing fatigue problems in riveted steel bridges, develop estimates of their expected fatigue life, and determine the effect of overload vehicles and future changes in legal load limits on the fatigue life. Analytical and experimental studies will be performed.

(2) Thermal: This study will evaluate the thermal movements in steel bridges with composite concrete bridge decks. It will develop recommendations for improved design procedures for establishing design thermal movements and more rational length limits for integral construction with this bridge type.

Expected Products or Deliverables:
(1) Fatigue: Study will provide a better understanding of the fatigue and dynamic response of steel truss and tied arch bridges. Initial estimates of expected fatigue life and an indication of the load spectrum on the critical elements will be developed.
(2) Thermal: Study will produce maps which propose design temperatures for thermal movements of steel bridges and rational length limits for integral construction with these same bridge types. Improved strategies for dealing with installation temperature of bridge components, joints and bearings will be proposed.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Charles W. Roeder is a Professor of Civil Engineering at the University of Washington. He has performed extensive research on seismic behavior of steel structures, thermal movements in bridges, and bearings and expansion joints.

Gregory A. MacRae graduated from the University of Canterbury, New Zealand in 1984 and worked as a structural engineer for one year before returning to his alma mater to carry out research on the seismic performance of steel frames. As a result of his experimental testing and computer simulations, he received a doctorate in 1989. During 1990 and 1991 he carried out experimental and analytical research into the seismic performance of hollow stiffened bridge columns at the Ministry of Construction research laboratory in Tsukuba, Japan. From 1992 to 1994 he worked at the University of California, San Diego where he was involved with the bridge seismic assessment and retrofit program. He has been an Assistant Professor in the Department of Civil Engineering at the University of Washington since late 1994. His interests include earthquake engineering, design philosophy, steel structures, reinforced concrete, dynamic response and the assessment and retrofit of existing structures.

Principal Investigator's Telephone Number: (206) 543-6199
Facsimile Number: (206) 543-1543
E-mail Address: croeder@u.washington.edu

RESEARCH IN PROGRESS ON STEEL BRIDGES AT UNIVERSITY OF WASHINGTON

Charles W. Roeder[1] and Gregory MacRae[1]

[1]Department of Civil Engineering
University of Washington, Seattle, WA 98195-2700

Research Objectives

Two research projects at the University of Washington are related to the design and performance of steel bridges. The first is funded by the American Iron and Steel Institute (AISI) and considers the thermal movements in steel bridges with composite concrete bridge decks. Present AASHTO design temperatures have remained unchanged for more than 70 years, and the temperature design methods are very different for steel bridges than for concrete bridges. Further, these existing procedures may lead to uneconomical and irrational design for steel bridges. This research study is examining these design temperatures. It will develop recommendations for improved design procedures for establishing design thermal movements and more rational length limits for integral construction with this bridge type. The second research study is funded by Washington State Department of Transportation (WSDOT). Riveted steel truss and tied arch bridges were commonly used by WSDOT during the development of the interstate highway system. These bridges take a tremendous volume of truck traffic. Riveted bridges are not commonly regarded as fatigue sensitive, but there have been a substantial number of fatigue cracks observed in many of these older riveted bridges. Repairs or modifications have been made, but sometimes the repair led to other fatigue problems at other locations. This research study will establish the cause of existing fatigue problems, develop estimates of the expected fatigue life of these riveted steel bridges, and determine the effect of overload vehicles and future changes in legal load limits on the fatigue life.

Research Approach

Two research projects are described in this brief paper. The projects are quite different in content and emphasis and they will be described separately.

<u>Thermal Movements in Steel Bridges:</u> Thermal movements control the design of bearings and expansion joints of most steel bridges, and they have a significant impact on the economy of the bridge design. Bridge movements are controlled by the average bridge temperature, but theoretical calculations show that bridge temperature varies widely with time through the bridge cross section. Figure 1 shows a comparison of range of extreme average bridge temperatures obtained with theoretical calculations and the AASHTO bridge temperature ranges. The figure also shows the average bridge temperature for 3 different bridge types at very different geographic locations as a function of ambient air temperature. The ambient air temperature is defined as a weighted average of the daily high and low air temperatures. It can be seen that in any given climate, a steel bridge experiences a temperature range that is only about 20^{o}F larger than that for a concrete bridge rather than the 50 to 70^{o}F larger implied by AASHTO. Further, concrete bridges are

designed for a temperature range which does not depend on the installation temperature of the bridge superstructure, while the installation temperature for steel bridges leads to increased movements for steel bridges beyond those suggested in Fig. 1.

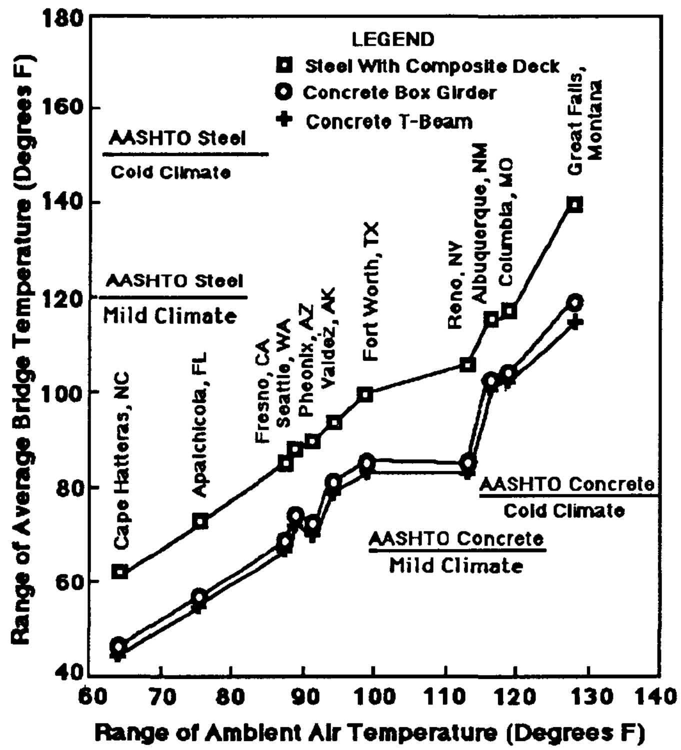

Figure 1.

Concrete bridges are designed for smaller temperature ranges than theoretically predicted, but there is little evidence that concrete bridges experience serious damage because of excess thermal movements. This suggests that the excess movements in concrete bridges are being accommodated by movements of piers, abutments and the superstructure. These combined observations lead to the conclusion that steel bridges are commonly designed for larger movements than are truly needed. At first glance these overly large thermal movement estimates may seem trivial, but they rapidly become quite significant. Bridge engineers commonly use sliding pot bearings for movements larger than ±2.0" whereas elastomeric bearings could be used if the movements were smaller. The change from pot bearings to elastomeric bearings could reduce the bridge cost by more than 2%. Bridge engineers commonly shift from compression joints to strip seals to modular expansion joints as the range of movement increases, and the cost of the joint seal increases with these changes. Further, integral construction often offers significant advantages in reduced maintenance and improved bridge performance. The overestimated thermal movements place severe limits on the use of integral construction, since it is used only when the expected thermal movement is less than 2 inches. Finally, it should be noted that bearings and expansion joints are high maintenance components, and joints and bearings which must tolerate large movements often appear more trouble prone than those designed for smaller movements. As a result, overly large design movements for steel bridges must be a serious concern.

This research is a theoretical study considering the temperatures and movements in steel bridges with composite decks. First, the researchers will develop an air temperature map and guidelines for converting air temperature into bridge temperature so that appropriate design temperature ranges can be chosen for bridges. This will be done through review of recorded temperature data throughout the US, calculation of bridge temperatures from these weather data, statistical analysis of the temperature and movement data, and translation of the data into maps and design procedures. Second, theoretical calculations of bridge movements with different installation strategies will be performed to establish consistent procedures for compensating for installation temperature and design temperature extremes. Methods of accounting formally for that portion of the thermal movement that is absorbed by deformation of the structure and foundations will be examined. Finally, these thermal movements will be examined to develop rational length limits for the use of integral construction. Experimental verification of the design maps and guidelines through limited field measurements are likely to be required. However, these are not included in this research project.

Fatigue of Riveted Steel Bridges: Over the past few years fatigue cracking has become severe on several riveted steel bridges in the state of Washington. Most of these bridges are on the interstate system. In several steel tied arch bridges, cracks have initiated just beneath the top flange of "simply supported" floor beams and some cracks as long as 10 in have been observed. Modifications have been made to improve the fatigue performance. In some cases, holes were drilled through the tip of the crack. This stabilized cracking in some cases but not in others. Elements with significant cracking have been repaired by addition of plates used to reinforce the cracked region and induce compressive stress at the tip of the crack through clamping action. Reinforcements have been added to restrain undesirable movements and deformations in some bridges, and these repairs have led to new cracks which occur in the stringers. In truss bridges, significant cracking is occurring at the copes of stringers and web angles. In one truss bridge, the bottom chord cracked half way through the section. This was discovered during routine painting and was quickly repaired.

WSDOT was concerned about the extent and evolving trends of this cracking. Very little information is available about the fatigue life of riveted steel bridges and this research project was initiated to address the issues. The first stage of the study is a 2 year project concentrating on understanding the demands on various parts of truss and tied arch bridges due to traffic loading. A second phase, concentrating on estimating the capacities is expected to be performed later. To date, analytical studies have been carried out in order to obtain an idea of the likely behavior of the bridges due to traffic loading. The tied arch at I5 crossing the Toutle River and the 3 span truss crossing the North Fork of the Lewis River have been selected as the subject bridges. In the initial analysis both bridges were analyzed using SAP90. Dead load as well as one standard HS-20 truck moving over the bridge in the lane beside the truss (or arch) was considered. Trucks were simulated to move over the bridges at speeds of 6 mph and 60 mph. Results indicated that the higher speed truck caused larger stresses. The magnitude of this difference was generally less than 7% but in some members it was as high as 21%. In Toutle River tied arch bridge, stresses as high as 6 ksi were predicted in the top and bottom chords at distance of about 1/4 of the length from each end, and deflections of about 2 in. were also predicted at about the same points. Lewis River truss bridge has a predicted stress range of approximately 3 ksi in the truss members. The stress range at the center of the stringers in both bridges are predicted to be approximately 8 ksi based on the result of a wheel group of a dual truck axle with a 10 kip wheel load. This stress is expected regardless of whether there is composite or non-composite action. It relates to an end rotation of the stringers of about 0.2% if they are simply supported. An analytical study of a beam-column joint in being carried out in order to observe the effect of strain in the member due to different fixity assumptions and cope hole detailing methods. The analysis suggests that there may be fundamental differences in the fatigue mechanism for the two bridge types, since the tied arch may be more sensitive to deformation than stress level.

Field measurements will be performed on the two bridges to obtain further information on this fatigue problem. Two types of instrumentation are proposed. The first is to measure the overall response of the bridge in order to verify the results from the dynamic analysis. It is anticipated that this will be carried out by either measuring axial strain or curvature with a full-bridge strain gage setup on some of the members indicating greater strain ranges. Due to the difficulties of measuring total displacement directly, the displacements may be computed from results of accelerometers. The second will be to measure the response of components, such as stringers and floor beams, in order to understand the actual stress and strain histories which may affect their fatigue life. Approximately 24 channels are expected to be used per bridge. Data recording is to be carried out in two ways. Firstly, data is to be recorded for several seconds when a large truck of known weight, speed and position moves over the bridge. The data recording will be initialized by a triggering system. This data will allow the time and value of peak demand at different positions to be calculated relative to the peak demand in one position. Secondly, the computer equipment will be left running to obtain a distribution of peak demand at some position in the bridge. This will then be able to be correlated back to an equivalent number of trucks of different sizes. A load spectrum which suggests the number cycles of stress demand which is likely to affect the bridge fatigue life can then be estimated. The physical problems of obtaining access to the bridges, attaching the gauges and ensuring they remain functional, reducing electronic noise from the cables and from trucks emitting strong radio wave signals, as well as keeping the computer equipment running well in a well ventilated yet weatherproof and vandalproof location are some of the challenges which still need to be faced.

Expected Products

The thermal movement study will produce maps which propose design temperatures for thermal movements of steel bridges and rational length limits for integral construction with these same bridge types. Improved strategies for dealing with the installation temperature of bridge components, joints and bearings will be proposed. The fatigue study will provide a better understanding of the fatigue and dynamic response of the steel truss and tied arch bridges. Initial estimates of the expected fatigue life of these bridges and an indication of the load spectrum on the critical elements will be developed.

Preliminary Results - None

Seismic Performance, Design, and Retrofitting (Part 2)

SESSION V

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Bridge Damage and its Consequences in the 1994 Northridge Earthquake

Author(s) and Affiliation(s):

Anne S. Kiremidjian, Nesrin Basöz and Stephanie King
John A. Blume Earthquake Engineering Center, and
Kincho Law, Department of Civil Engineering
Stanford University, Stanford, CA

Principal Investigator: Anne S. Kiremidjian

Sponsor(s): National Science Foundation

Research Start Date: September 1994
Expected Completion Date: January 1996

Research Objectives:

The objective of this project was to investigate bridge damage characteristics and the impact of bridge damage on the post-earthquake performance of the transportation system in the Los Angeles area. The bridge damage assessment concentrated on correlation of damage data with ground-motion characteristics and comparison with expected bridge damage and functionality from scenario earthquake simulations. The evaluation of the performance of the transportation system focused on emergency response planning and management activities with the objectives to identify critical links within the system that are particularly vulnerable and should be considered as high priority in retrofit decisions, and to determine the optimal alternate routes when critical routes become unavailable following an earthquake.

Expected Products or Deliverables:

Databases compiled for the greater Los Angeles area. The set of compiled databases include ground motion, a description of local and state bridges, and bridge damage.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Anne Kiremidjian is Professor of Civil Engineering and Director of The John A. Blume Earthquake Engineering Center at Stanford University. She holds a BS degree from Columbia University and MS and Ph.D. degrees from Stanford University. Dr. Kiremidjian teaches courses in earthquake engineering, structural analysis, probabilistic methods, and structural reliability at Stanford. She has conducted extensive research in the areas of seismic hazard and risk modeling, earthquake ground motion characterization, structural damage modeling and structural reliability analysis. Some of her recent work includes the development of regional damage and loss evaluation methods utilizing modern computational techniques and development of real time damage monitoring systems. Anne Kiremidjian has also worked on the formulation of reliability techniques for critical structures found at industrial facilities. Currently she supervises five doctoral students and is the principal investigator on four research grants. She is active in numerous professional organizations including ASCE, EERI, SSA, IASSAR, SEAOC, Tau Beta Pi, Sigma Xi and SWE. In many of these organizations Dr. Kiremidjian serves on the executive board or as chair of various committees.

Nesrin I. Basöz is a Ph.D. candidate in the Structures and Geomechanics Division of the Civil Engineering Department at Stanford University. Her dissertation focuses on the risk assessment for highway transportation systems which includes the prioritization of bridges for seismic retrofitting as a means of hazard mitigation, and the application of network analysis to highway transportation systems for emergency response planning and management.

Stephanie A. King is Associate Director of The John A. Blume Earthquake Engineering Center. Her research focuses on seismic hazard and risk analysis and the use of geographic information systems.

Kincho H. Law is an Associate Professor of Civil Engineering at Stanford University. Dr. Law's research focuses on the application of advanced computing technology in engineering. His research areas include computational mechanics, numerical methods, analysis and simulation of large scale systems using high performance parallel computers and distributed workstations, application of artificial intelligence and information management technologies to enhance analysis and design of engineered facilities, and to coordinate collaborative and concurrent engineering activities.

Principal Investigator's Telephone Number:	(415) 723-4164
Facsimile Number:	(415) 725-9755
E-mail Address:	ask@ce.stanford.edu

BRIDGE DAMAGE AND ITS CONSEQUENCES IN THE 1994 NORTHRIDGE EARTHQUAKE

Anne S. Kiremidjian[1], *Nesrin Basöz*[2], *Stephanie A. King*[3], *and Kincho Law*[4]

[1] Professor and Director, [2] Doctoral Candidate, [3] Associate Director
John A. Blume Earthquake Engineering Center
[4] Associate Professor
Department of Civil Engineering, Stanford University
Stanford, California 94305-4020

Research Objectives

The objective of this project is to investigate bridge damage characteristics and the impact of bridge damage on the post-earthquake performance of the transportation system in the Los Angeles area. The bridge damage assessment concentrated on the correlation of damage data with ground-motion characteristics and comparison with expected bridge damage and functionality from scenario earthquake simulations. The evaluation of the performance of the transportation system focused on emergency response planning and management activities with the objectives: (a) to identify critical links within the system that are particularly vulnerable and should be considered as high priority in retrofit decisions, and (b) to determine the optimal alternate routes when critical routes become unavailable following an earthquake.

Research Approach

The January 17, 1994 Northridge, California earthquake damaged several bridges, in some instances resulting in severe disruptions to the transportation system of the region and causing major delays. The earthquake has provided valuable information for evaluating the post-earthquake performance of the transportation system in the area. In this research, a geographic information system (GIS) based methodology was developed for risk mitigation of highway transportation systems. The developed methodology has the following components:

- evaluation of seismic hazard throughout the study region;
- evaluation of the performance of critical components, such as bridges, in the transportation system when subjected to an earthquake;
- modeling of the emergency traffic as a network flow model;
- determination of alternate routes and associated time delays due to bridge damage; and
- development of a decision support system for bridge retrofit prioritization and alternative routing of emergency vehicles from and to the disaster areas.

The first part of the project involved the collection of information for the study region, including highway transportation system inventory data, geologic data, ground motion characteristics of the earthquake, and earthquake damage data. Bridge inventory data that contain information on the physical characteristics of bridges for the greater Los Angeles area were extracted from the bridge maintenance database provided by the California Department of Transportation (Caltrans). The road inventory data were extracted from the database of Navtech™ software used for route generation developed by the Navigation Technologies Corporation. In addition, information such as a surface geology map, earthquake ground motion data, and ground motion attenuation models were also collected. The commercial GIS software Arc/INFO™ was used to store all inventory data and generate seismic hazard maps for the study region.

Bridge damage data were gathered from various sources including Caltrans *Bridge Damage Reports* (Caltrans, 1994) and reconnaissance reports prepared by Caltrans, National Center for Earthquake Engineering Research (Buckle, 1994), and Earthquake Engineering Research Institute (EERI, 1995). The observed damage data for all bridges were categorized by various parameters including, structural type, substructure material type, number of spans, design year, and ATC-13 (ATC, 1985) and NIBS (RMS, 1994) engineering classifications. The ATC-13 and NIBS bridge classes were assigned based on the data attributes contained in the Caltrans database. In addition, the bridge damage data were categorized by the type of observed damage. Judgment was required to treat inconsistencies in the interpretation of the observed damage data. Various definitions of damage have been developed in previous studies and these were reviewed in the project. New damage states for bridge components, such as columns, abutments and connections, were defined for concrete bridges based on the observed bridge damage. Bridge damage data from the Northridge earthquake were also compared to the bridge damage observed in the 1971 San Fernando earthquake.

In the second part of the project, the GIS was used to compare the recorded ground motions to those developed using various attenuation models, as well as to combine various types of spatially-referenced information. An empirical relationship between Modified Mercalli Intensity (MMI) and peak ground acceleration (PGA) was developed by overlaying recording station data on an MMI map. Empirical damage probability matrices (DPMs) and fragility curves were developed for all bridges and for bridges grouped by the parameters described in the previous paragraph by overlaying maps of bridge locations on PGA and MMI maps. In addition, the ATC-13 DPMs and NIBS fragility curves were used to estimate the bridge damage for comparison to the observed bridge damage and the developed empirical motion-damage relationships. The post-earthquake functionality levels of the bridges were also calculated using the restoration curves provided in ATC-13 and NIBS.

In the last part of the project, network connectivity analyses were conducted to: (a) identify critical bridges, i.e., bridges that would disrupt the connectivity of the system or would cause an unacceptable delay in travel time for emergency response activities upon inaccessibility; (b) to determine all possible alternate routes for given origin-destination pairs and associated travel time delays to demonstrate emergency planning; and (c) to determine available routes after the earthquake for the origin-destination pairs, assuming that bridges with functionality levels of 50 percent or more are accessible during emergency response. The network analysis was conducted using both the observed bridge damage information as well as the functionality estimated by the ATC-13 and NIBS models to simulate the emergency management decisions for available routes and detours. These results were compared to the actual decisions made after the earthquake.

Products

The results of the study provided insight for improvement of the planning, operation, and management procedures of the transportation system in order to minimize disruptions and delays due to bridge failures in future earthquakes. The GIS proved to be a very useful tool for storing, analyzing and displaying data, as well as developing and comparing relationships among various types of spatially-referenced information. Development of a GIS-based decision support system would be beneficial to develop disaster mitigation policies and enhance the efficiency and effectiveness of emergency response activities. Such a decision support system should also consider other transportation systems such as the railway or metrolink systems to assess long term economic impacts of bridge failure on the entire transportation system.

In addition, the detailed investigation of the bridge damage data demonstrated the inconsistency in bridge damage definitions among practicing engineers, and the need for a more systematic procedure for gathering of bridge damage data. Comparison of empirical fragility curves developed for damaged bridges with available ground motion-damage relationships indicated that the available ground motion-damage relationships are too rudimentary and are not representative of the actual bridge behavior under seismic loading. In a follow-up project currently sponsored by Federal Highway Administration and National Center for Earthquake Engineering Research, damage data from the Northridge earthquake is being analyzed to correlate the bridge damage to ground motion levels, bridge functionality, and repair costs.

Results

In this paper, results related to bridge damage data are presented. The data for 236 bridges reported as damaged in Los Angeles County were used to illustrate the observed ground motion-damage relationships. Figures 1 and 2 show the distribution of damage with respect to observed PGA and MMI values, respectively. In Figure 3, the distribution of damaged bridges by design year is depicted.

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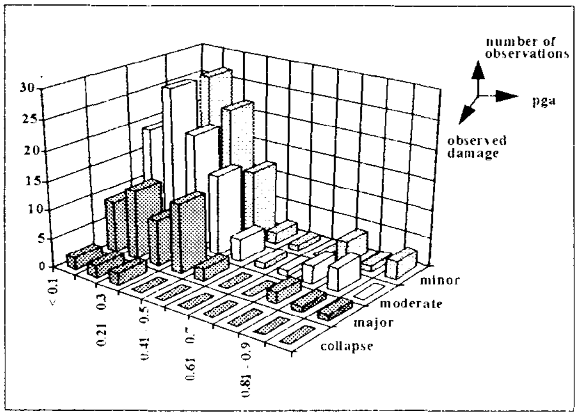
EERI, 1995, *Northridge Earthquake of January 17, 1994 Reconnaissance Report*, Earthquake Spectra, Earthquake Engineering Research Institute, Oakland, California.

Buckle, 1994, *The Northridge, California Earthquake of January 17, 1994: Performance of Highway Bridges*, Technical Report NCEER-94-0008.

RMS, 1995, *Development of a Standardized Earthquake Loss Estimation Methodology*, Prepared for the National Institute of Building Sciences by Risk Management Solutions, Inc., Menlo Park, California.

Acknowledgments

The authors would like to express their gratitude to Navigational Technologies Corporation for donating the Navtech™ software and to Environmental Systems Research Institute for donating their Arc/INFO™ software, both of which were used in this research. In addition, the authors wish to thank Caltrans for providing bridge inventory information and damage data for the Los Angeles area. This research was supported by National Science Foundation Grant CMS-9416420.



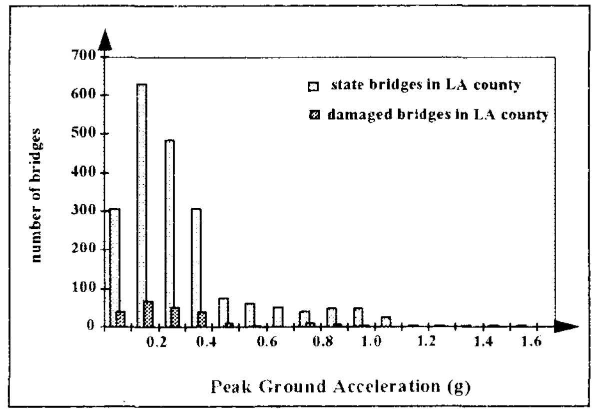
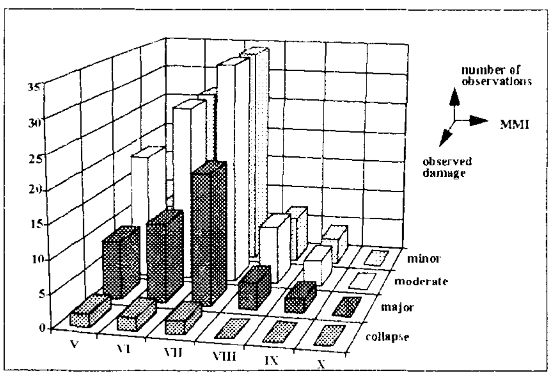


Figure 1.a Distribution of Bridge Damage by Observed PGA Levels

Figure 1.b State Bridges in the LA county by Observed PGA Levels



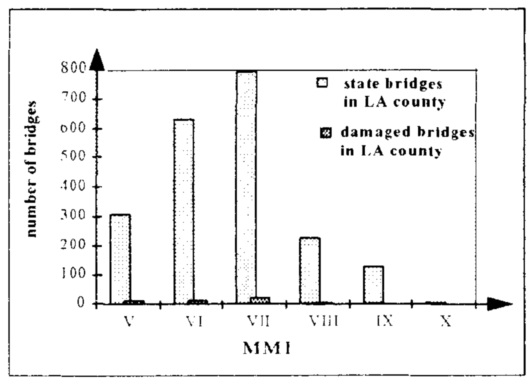
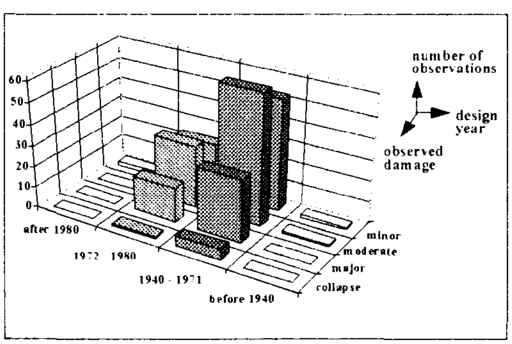


Figure 2.a Distribution of Bridge Damage by Observed MMI Levels

Figure 2.b Total and Damaged State Bridges in the LA county by Observed MMI Levels



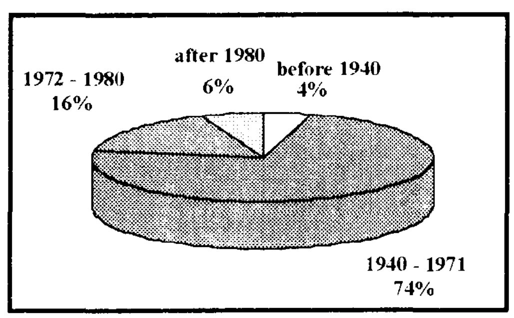


Figure 3.a Distribution of Bridge Damage by Design Year

Figure 3.b Distribution of State Bridges in the LA county by Design Year

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS RESEARCH ABSTRACT

Title: Seismic Retrofitting Experience and Experiments in Illinois

Author(s) and Affiliation(s):

William L. Gamble and Neil M. Hawkins, Department of Civil Engineering
University of Illinois at Urbana-Champaign, Urbana, IL
Iraj I. Kaspar, Illinois Department of Transportation, Springfield, IL

Principal Investigator: William L. Gamble and Neil M. Hawkins

Sponsor(s): Illinois Department of Transportation

Research Start Date: September 1993
Expected Completion Date: September 1997

Research Objectives:

The primary objective of this research is to provide information to the Illinois Department of Transportation about the strength and behavior of part of their existing stock of bridge piers which were designed without concern for seismic forces and deformations. A second objective is to provide data on the behavior of piers which were retrofitted in various ways in order to protect the lap splices which exist at the bases of most pier columns. Differential temperature effects will also be studied for the case of carbon fiber jackets, because of very large differences between the coefficients of thermal expansion of carbon fiber and the enclosed concrete, and because of the large temperature ranges which may be encountered in the Midwest.

Expected Products or Deliverables:

The study is expected to lead to improvements in existing retrofit design methods, and to greater confidence that the chosen retrofit will be adequate without being greatly in excess of the actual needs. Another expected product of the research will be guidance on the use of carbon fiber jackets in climates with very low minimum temperatures.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

William L. Gamble is Professor of Civil Engineering at the University of Illinois at Urbana-Champaign, where he has been engaged in teaching of and research on reinforced and prestressed concrete structures since 1963. He received the Ph.D. degree from the University of Illinois in 1962. His research work has concentrated on reinforced concrete slabs and prestressed concrete highway bridges, and he has co-authored one book (with Robert Park) and has published extensively. He is a registered Structural Engineer in Illinois, a Fellow of ACI, and member of ASCE, PCI, and ASTM.

Neil M. Hawkins is Professor and Head of the Department of Civil Engineering at the University of Illinois at Urbana-Champaign, a position he has held since 1991. He received his Ph.D. from the University of Illinois in 1961. He has been engaged in teaching and research continuously since that time, first at the University of Sydney, Australia, 1962-68, and then at the University of Washington, Seattle, 1968-1991. He was Chairman of Civil Engineering and Associate Dean for Research, Facilities, and External Affairs at the University of Washington. His research work has concentrated on earthquake engineering, and the design of reinforced concrete structures, and has over 200 publications. He is a former Director of ACI and EERI, and is a present Director of the Post-Tensioning Institute. He serves on two ASCE research committees, and is a member of the ACI Building Code Committee.

Iraj I. Kaspar is an Engineer of Bridge Design with the Illinois Department of Transportation, Springfield, IL. He has been with ILLDOT for more than 30 years, and has been involved with all phases of the bridge program. He received the MS degree from the University of Illinois in 1964. His current responsibilities include review and approval of bridge plans prepared by consultants in addition to supervision of design staff. He is a registered Professional Engineer in Illinois, a member of ASTM, chairman of the AASHTO Rigid Culvert Liaison Committee, a member of the TRB Committee on Culverts and Hydraulic Structures, and has been a member of various NCHRP research panels. He is a member of ISPE and NSPE.

Principal Investigator's Telephone Number:	(217) 333-6935
Facsimile Number:	(217) 333-9464
E-mail Address:	w-gamble@uiuc.edu

Seismic Retrofitting Experience and Experiments in Illinois

William L. Gamble[1], Neil M. Hawkins[1], and Iraj I. Kaspar[2]
[1] Department of Civil Engineering, University of Illinois at Urbana-Champaign,
[2] Illinois Dept. of Transportation, Springfield, IL

Research Objectives

The primary objective of this research is to provide to ILLDOT information about the strength and behavior of part of the existing stock of bridge piers which were designed without concern for seismic forces and deformations. The second objective is to provide data on the behavior of similar piers which were retrofitted in various ways in order to protect the lap splices which exist at the bases of most pier columns.

After field tests of nine columns had been completed and the bar arrangements in the as-built conditions had been documented, six additional half-scale columns were tested in the laboratory in order to investigate the effects of having non-contact lap splices. The bar separations measured in the field were sometimes more than 100 mm. Fig. 1 shows the cross section of a typical 1.37 m diameter column, with the bar locations shown to scale. The dowel bars coming up from the footing have obviously been set on a significantly smaller circle than the column bars. The smaller columns, 1.22 m diam, generally had more uniform bar spacings and cover.

Additional columns are being constructed to be tested to further investigate retrofitting procedures and designs.

Research Approach

Fully-reversed lateral load tests were conducted on nine large (1.22 m or 1.37 m diam) bridge pier columns in East St. Louis, IL. The columns, which were reinforced with 18 to 28 36-mm (#11) reinforcing bars, were available because the two mainline bridges in the interchange leading to the Poplar St. Bridge were being completely replaced. The bridges were built in the late 1960's, and the pier columns had 1090 mm long (30 d_b) lap splices of all bars at the column bases. The columns had minimal lateral reinforcement, 12.7 mm (#4) bars at 300 mm (12 in.), and this hoop reinforcement lacked positive end anchorage by hooks or other means. Thus, neither the lap splices nor the compression zone concrete had effective confinement. Horizontal forces were applied to the columns at points 6.7 m above the column bases by hydraulic rams which reacted on a steel frame which was anchored to the crash wall and pile cap by means of anchor bolts set in epoxy.

Three of the four as-built columns failed due to failure of the lap splices, and the fourth was at the point of failure when the test had to be ended. The 1.37-m columns, with 24 or 28 bars, suffered significant loss of capacity at deflections ranging from 55 to 75 mm; the 1.22-m columns, with 18 bars, deflected farther but did not develop their full yield capacities. Five columns were retrofitted by: (1) external tensioned steel bands, (2) external tensioned prestressing strands, (3) thin fiberglass jacket laid up of cloth and epoxy resin, and (4) thicker fiberglass jacket made of four pre-formed layers which were glued to the column and to each other. No retrofitted column failed, even though four of the columns were subjected to multiple load cycles with the final deflections being at least 200 mm in one direction and at least 300 mm in the other direction, at the level of the applied load.

Figs. 2 and 3 show the load-deflection curves for an as-built column and the column retrofitted with tensioned 15.24 mm diam prestressing strands spaced at 200 mm over about 1.5 times the lap splice length. The strands were stressed to about 186 kN (42 kips) force during installation. Both columns were 1.37 m in diam and contained 24 36-mm bars, and both were part of the same pier and thus should have been similar before the retrofitting operation. The two curves are to the same scale in order to emphasize the enhancement of deformation capacity which the retrofit supplied.

The next six columns will be used to explore retrofit design and behavior, with the intent that at least one of the retrofits be under-designed to the point of failure of the retrofit. These columns will be retrofitted with prestressing strands or with carbon fiber jackets. In addition to structural tests, a study of temperature effects for the carbon fiber material will be conducted, because of very large apparent differences between the coefficients of thermal expansion of concrete and carbon fiber, and the rather large temperature ranges which can be experienced in the Midwest.

Expected Products

The field tests demonstrated that each of the different retrofit methods used was able to prevent lap-splice failures, which was an important finding. However, the relative lack of instrumentation makes it impossible to know with any certainty whether all four systems were over-designed, or whether any were close to their full capacities. The next series of tests is expected to lead to improvements in existing retrofit design methods, and to greater confidence that the retrofit chosen will be adequate without being greatly in excess of the actual minimum requirements.

The results of the four as-built columns in the field and the six completed laboratory tests are being analyzed. This information will provide useful guidance to engineers who are evaluating existing bridges, and should help prioritize cases for retrofitting by helping identify the most critical cases of cover, bar spacing, and bar separation.

Preliminary Results

The most important result was that all of the retrofit methods were successful. Contrary to early expectations, there were no discernable differences between the stressed steel, or "active" systems, and the fiberglass, or "passive" systems, when judged on the basis of the measured load-deflection curves or on the basis of the limited rotation and curvature data which was obtained in the field tests.

The six columns tested to study the effects of bar separations on lap-splice strength showed that the separation was much less important than either the cover or the spacing of bars around the perimeter of the column. These tests are believed to be the first experimental study of fully reversed loadings on lap-splices in which the bars were not in contact. The conclusions are for splices in which the separation is in the radial direction of the column, and should not be extrapolated to cases where all bars are at the same cover depth, with the separation being measured in a plane parallel to the concrete surface, a case which has not been investigated.

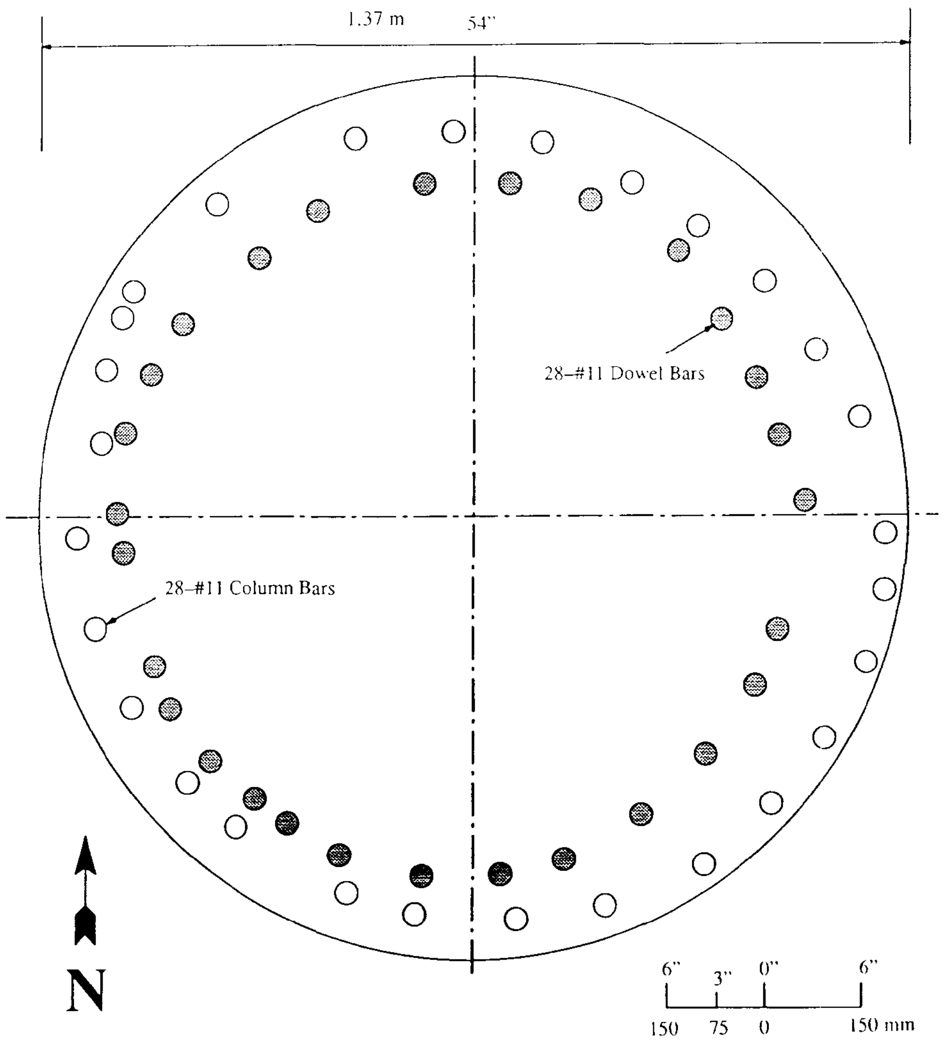


Fig. 1 Cross Section of As-Built Column B18S

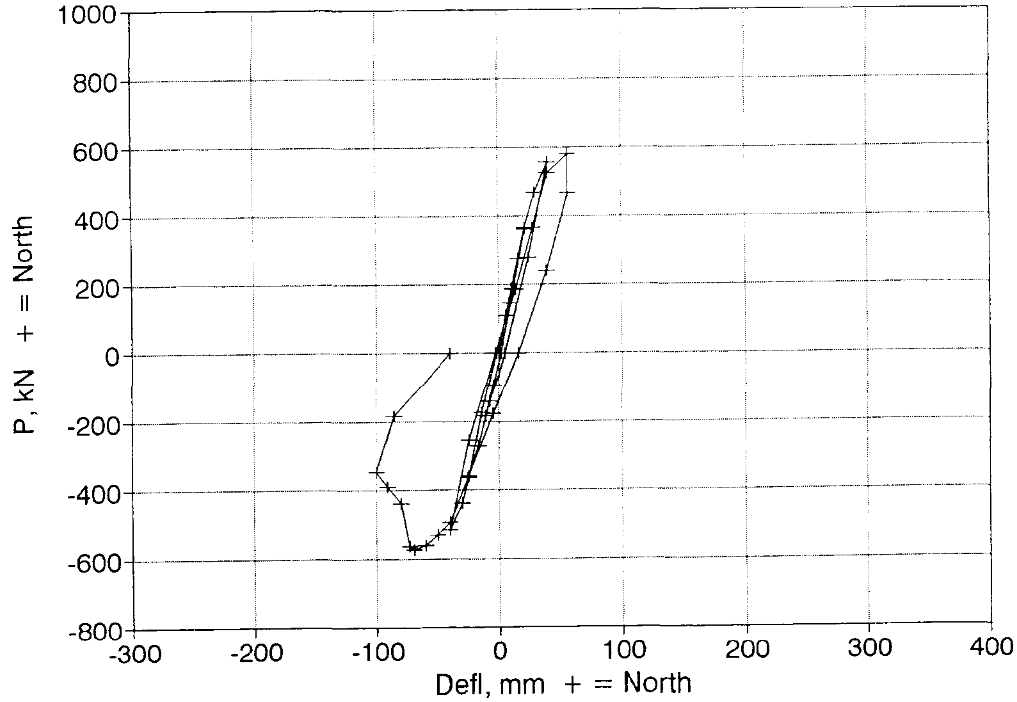


Fig. 2 Load-Deflection Curve for As-Built Column C17S

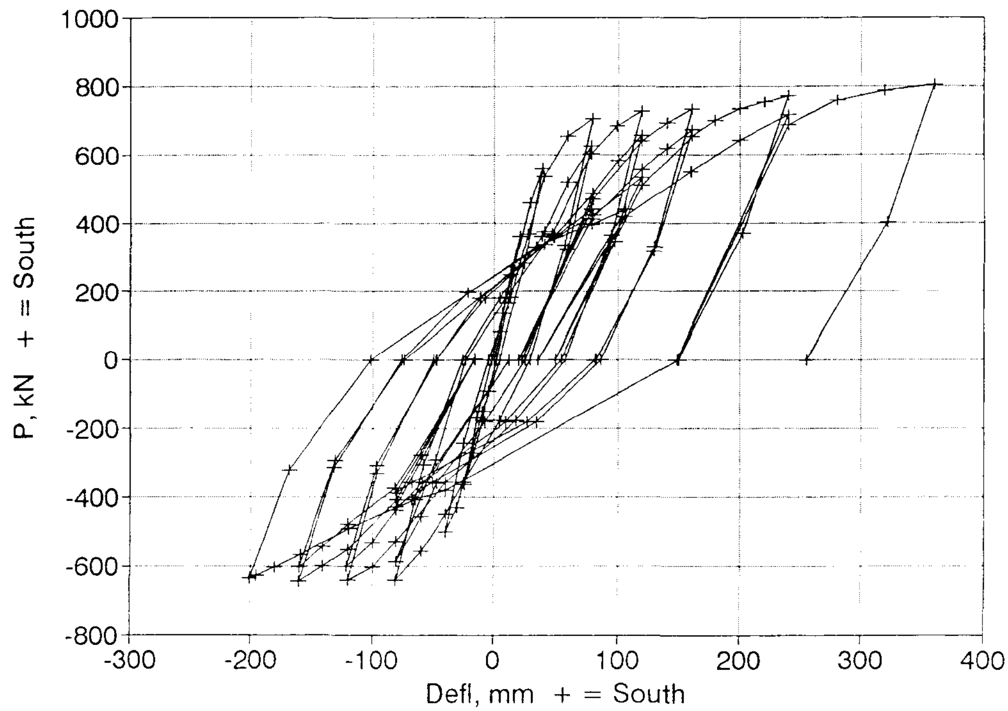


Fig. 3 Load-Deflection Curve for Retrofitted Column C17N
Retrofitted with Tensioned 15.24 mm Strands at 200 mm Spacing

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS RESEARCH ABSTRACT

Title: Strength Degradation of Existing Bridge Columns

Author(s) and Affiliation(s):

Omar A. Jaradat and David I. McLean
Department of Civil and Environmental Engineering
Washington State University, Pullman, WA
M. Lee Marsh, BERGER/ABAM Engineers, Inc., Federal Way, WA

Principal Investigator: David I. McLean

Sponsor(s): National Science Foundation

Research Start Date: September 30, 1993
Expected Completion Date: June 30, 1996

Research Objectives:

Much of the previous research on the seismic behavior of reinforced concrete columns was performed on cantilever columns that lose their flexural and shear strength at the same time. The remaining shear strength after the formation of a plastic hinge or the degradation of a spliced region is only partially understood. In this study, experimental tests were performed on column specimens with moment restraint provided at both ends, resulting in a transfer of shear through the column even after a hinging region degrades. The main objective of this study is to characterize the flexure and shear behavior of older bridge columns for purposes of seismic assessment and retrofit design, particularly with regard to assessing the shear strength present in degraded hinge regions.

Expected Products or Deliverables:

The results of this study will provide additional information to characterize the strength and degradation behavior of reinforced concrete bridge columns under seismic loading. Current and proposed flexural and shear models will be evaluated based on observed column behavior. Recommendations will be made for assessing the seismic performance of columns in existing bridges. The ability of degraded column hinge regions to continue to transfer shear and axial forces will be determined, and the viability of proposed partial retrofit strategies evaluated.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Omar A. Jaradat is a Ph.D. candidate in the Department of Civil and Environmental Engineering at Washington State University. His area of study is structural and earthquake engineering and his research interests include the assessment and retrofit of bridges under seismic forces, and the behavior and design of reinforced concrete structures.

David I. McLean is Associate Professor in the Department of Civil and Environmental Engineering at Washington State University. His research interests include the seismic response and strengthening of structures.

M. Lee Marsh is a Senior Engineer with BERGER/ABAM Engineers, Inc. His interests include the assessment, design and retrofit of reinforced concrete bridges and buildings.

Principal Investigator's Telephone Number:	(509) 335-3110
Facsimile Number:	(509) 335-7632
E-mail Address:	mclean@wsu.edu

STRENGTH DEGRADATION OF EXISTING BRIDGE COLUMNS

Omar A. Jaradat[1], David I. McLean[1] and M. Lee Marsh[2]

[1]Department of Civil and Environmental Engineering
Washington State University, Pullman, WA 99164-2910
[2]BERGER/ABAM Engineers, Inc.
33301 Ninth Ave. South, Federal Way, WA 98003-6395

Research Objectives

Much of the previous research on the seismic behavior of reinforced concrete columns was performed on cantilever columns that lose their flexural and shear strength at the same time. The remaining shear strength after the formation of a plastic hinge or the degradation of a spliced region is only partially understood. In this study, experimental tests were performed on column specimens with moment restraint provided at both ends, resulting in a transfer of shear through the column even after a hinging region degrades. The main objective of this study is to characterize the flexure and shear behavior of older bridge columns for purposes of seismic assessment and retrofit design, particularly with regard to assessing the shear strength present in degraded hinge regions.

Research Approach

Background: Before the 1971 San Fernando earthquake, bridge columns were designed with very little transverse reinforcement. Typically, No. 3 or No. 4 hoops at 12 inches on center were used in columns, regardless of cross-sectional dimensions, and the hoops had short extensions and anchorage only by lapping the ends in the cover concrete. Further, intermediate ties were rarely used. This detail results in many older columns being susceptible to shear failures, and it provides little confinement for developing full flexural capacity or preventing buckling of the longitudinal reinforcement.

Another detail commonly used in older bridges was splicing of the longitudinal bars at the bottom of the columns. Typically, starter bars were extended only 20 bar diameters from the foundations, which may not provide sufficient length to develop the yield strength of the reinforcement. Bond failure is also likely once the cover concrete spalls. These deficiencies result in a high potential for flexural strength degradation in the event of an earthquake.

As a result of the damage that has occurred to older bridges in recent earthquakes, major efforts are being directed at developing retrofit strategies to upgrade their seismic performance. An option included in current retrofit strategies for multi-column bent bridges is to retrofit only a few of the substructure elements. This partial retrofit strategy is driven by economics, particularly the costs of retrofitting footings. The approach assumes that in the substructure elements not retrofitted, column flexural strength will degrade during an earthquake, resulting in a pinned-end condition at the bases of the unretrofitted columns. The approach also assumes that the shear and axial capacities of the unretrofitted columns remain intact, with these capacities in fact being relied upon to contribute to the lateral resistance of the bridge in an earthquake.

Research Overview: An experimental investigation was made of the strength and degradation behavior of reinforced concrete bridge columns under seismic loading. Tests were conducted on eight approximately 1/3.6-scale column specimens that incorporated deficiencies present in bridges designed prior to 1971. The columns were fixed against rotation at both the top and bottom, as shown in Figure 1, resulting in a transfer of shear forces through the column even after a hinging region loses its flexural capacity. The specimens were subjected to increasing levels of cycled inelastic displacements under constant axial load. Variables in the specimen details included column height, longitudinal reinforcing ratio, lap splice length and column base retrofit detail, as shown below.

Specimen	Diameter (in.)	Height (in.)	Reinf. Ratio (%)	Splice Length	Column Base Retrofit
T1	10	70	2.0	20 d_b	none
T2	10	70	1.1	20 d_b	none
T3	10	70	2.0	35 d_b	none
S1	10	40	2.0	20 d_b	none
S2	10	40	1.1	20 d_b	none
S3	10	40	1.1	35 d_b	none
RT1	10	70	2.0	20 d_b	Caltrans Type F
RT2	10	70	2.0	20 d_b	Caltrans Type P

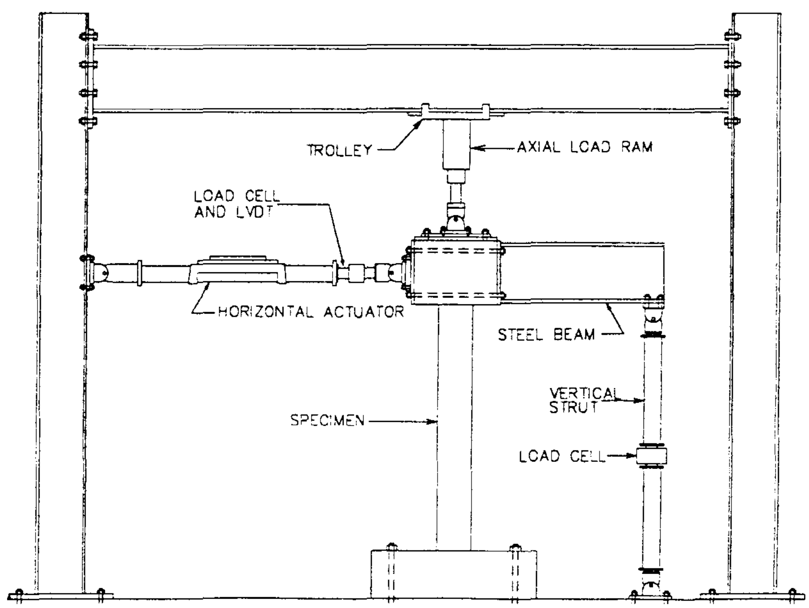


Figure 1 Testing arrangement.

Specimen performance was evaluated on the basis of flexural and shear load capacity, displacement ductility, strength degradation and hysteretic behavior. Observed behavior was compared with various current and proposed procedures for assessing the flexural and shear behavior of bridge columns under seismic loading. Results were compared with the theoretical flexural models proposed by Priestley and Seible [1] and with shear models based on ACI/AASHTO design equations, FHWA recommendations [2] and newly proposed models developed at the University of California at Berkeley [3] and University of California at San Diego [4].

Expected Products

The results of this study will provide additional information to characterize the strength and degradation behavior of reinforced concrete bridge columns under seismic loading. Current and proposed flexural and shear models will be evaluated based on observed column behavior. Recommendations will be made for assessing the seismic performance of columns in existing bridges. The ability of degraded column hinge regions to continue to transfer shear and axial forces will be determined, and the viability of proposed partial retrofit strategies evaluated.

Preliminary Results

Flexural-dominated failures occurred in five of the eight specimens. Rapid degradation in flexural strength was observed at the bottom hinging regions of the tested columns due to the presence of lap splices and poor confinement. Top hinging regions that did not have lap splices exhibited degradation in flexural capacities at higher displacement ductilities due to eventual longitudinal bar buckling. The hysteresis curves for the applied lateral load, top-section moment and bottom section moment for a specimen failing in flexure are shown in Figure 2.

Column base retrofitting based on the CALTRANS Type F retrofit detail [5] was effective in developing and maintaining the full flexural capacity at the base of the column through large ductilities. Retrofitting based on the CALTRANS Type P retrofit detail [5] resulted in lower flexural capacity than that with the Type F retrofit, and the base hinge behavior was intermediate between those columns with no retrofitting and that with the Type F retrofit.

Shear-dominated failures occurred in the three remaining specimens of this study. Two types of shear failures were observed: (1) shear failure with little or no flexural yielding, which occurred in the middle region of the columns, and (2) shear failure after extensive flexural yielding, which occurred at one of the column ends. The second type of shear failure is a result of concrete degradation within the hinging region, and it occurred at a lower shear load than that associated with the first type of shear failure. The hysteresis curves for the applied lateral load, top-section moment and bottom section moment for a specimen failing in shear are shown in Figure 3.

Work is currently underway to complete the evaluation of the observed results using current and proposed assessment models.

References

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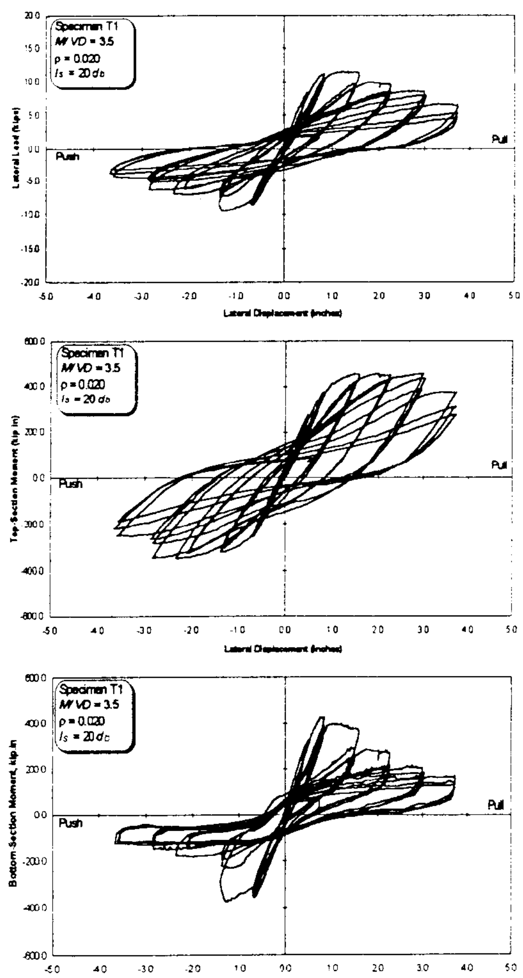


Figure 2: Hysteresis curves for applied lateral load, top-section moment, and bottom-section moment for Specimen T1.

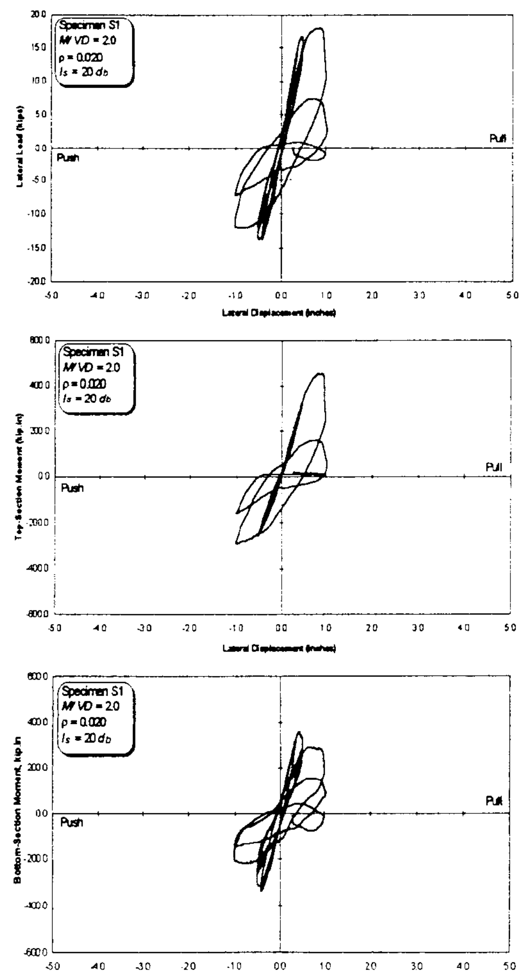


Figure 3: Hysteresis curves for applied lateral load, top-section moment, and bottom-section moment for Specimen S1.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS RESEARCH ABSTRACT

Title: Analysis and Design of Bridge Columns With Lap-Spliced Longitudinal Reinforcement

Author(s) and Affiliation(s):

Yan Xiao and Rui Ma
Department of Civil Engineering
University of Southern California, Los Angeles, CA

Principal Investigator: Yan Xiao

Sponsor(s): University of Southern California, CMI, CCCC

Research Start Date: March 1995
Expected Completion Date: June 1996

Research Objectives:

A series of experimental tests on bridge columns with lap-spliced longitudinal reinforcement have been conducted at the University of California, San Diego and the University of Southern California. Experimental results indicated that the ductility of the model columns was dominated by the bond deterioration of the longitudinal bars in the lap-spiced region and it is necessary to consider the bond deterioration in a rational assessment or retrofit design of bridge columns with lap-spiced longitudinal bars. In this research, a modified analytical model has been developed to analyze the performance of bridge columns with lap-splices. Analytical results are compared with the available test results and recommendations for retrofit design are put forward.

Expected Products or Deliverables:

The research provides a rational explanation and an accurate prediction of the behavior of columns with lap-sliced longitudinal reinforcement and suggests a practical and safe design approach allowing some degree of debonding in lap splices of longitudinal reinforcement in order to prevent the total failure of the column due to a rupture of the longitudinal bars under excessive earthquake loads.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Yan Xiao is an Assistant Professor of Civil Engineering at the University of Southern California. He received his BSCE degree from Tianjin University, China, and his MS and Ph.D. degrees from Kyushu University, Japan. His research interests include earthquake-resistant design of buildings and bridges, structural concrete, steel-concrete composite structures, and properties of structural materials.

Rui Ma is a Ph.D. candidate in the Department of Civil Engineering at the University of Southern California. He received his BSME degree from South China University of Technology, China, and his MSCE degree from USC. His research interests include earthquake-resistant design of reinforced structures and applications of composite material in structures.

Principal Investigator's Telephone Number:	(213) 740-6130
Facsimile Number:	(213) 744-1426
E-mail Address:	yanxiao@alnitak.usc.edu

ANALYSIS AND DESIGN OF BRIDGE COLUMNS WITH LAP-SPLICED LONGITUDINAL REINFORCEMENT

Yan Xiao, Ph.D., PE [1] and Rui Ma [2]

[1] Assistant Professor, Department of Civil Engineering
University of Southern California, Los Angeles, CA 90089-2531
[2] Research Assistant, Department of Civil Engineering
University of Southern California, Los Angeles, CA 90089-2531

Research Objectives

A series of experimental tests on bridge columns with lap-spliced longitudinal reinforcement have been conducted at UCSD and USC. Experimental results indicated that the ductility of the model columns was dominated by the bond deterioration of the longitudinal bars in the lap-spliced region and it is necessary to consider the bond deterioration in a rational assessment or retrofit design of bridge columns with lap-spliced longitudinal bars. In this paper a modified analytical model based on Xiao et.al. method[1] have been developed to analyze the performance of bridge columns with lap-splices. Analytical results are compared with the available test results and recommendations for retrofit design are put forward.

Research Approach

Analytical Model: A simple analytical model for analyzing the lateral force and displacement performance of reinforced concrete columns considering both flexural deformation and bond slips of longitudinal bars is developed for analyzing the performance of bridge columns with lap-splices, as shown in Fig. 1. A hinge is assumed at the bottom of the column, and the upper portion of the column is considered as an elastic beam column. Within the hinge length, the curvature is assumed uniform. While along the upper portion, the curvature is distributed linearly, as shown in Fig. 1(b). Bond links are assumed for all the starter bars above the hinge length, as shown in Fig. 1(c).

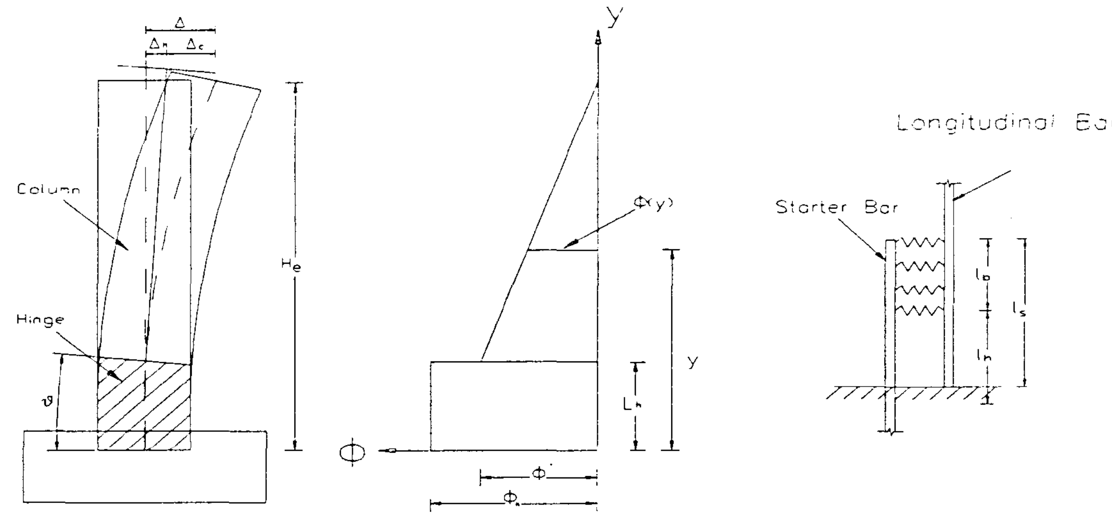
Hinge Length: Priestley et.al. have suggested the fixed length of plastic hinges for analyzing inelastic flexural behavior of reinforced concrete columns with or without jacketing [2]. In the proposed approach, the hinge length, L_h, is assumed variable corresponding to the stress of the extreme critical tensile steel, f_s. The expression takes the same format of Priestley's but uses a variable steel stress f_s to replace the yield strength f_y.

for "as built" columns: $$L_h = 0.08h + 0.15 d_{bl} f_s \qquad (1a)$$

for retrofitted columns: $$L_h = g + 0.3 d_{bl} f_s \qquad (1b)$$

where g is the gap provided at the lower bottom of the jacket.

Bond Links: It is assumed that in the lap-splice region, the stresses in the starter bars are transferred to the longitudinal column bars through a series of bond links, which are



(a) Column (b) Curvature (c) Bond links

Fig.1 Analytical Model

distributed throughout a length of L_b given by the following equation:

$$L_b = L_s - 0.15 d_{bl} f_s \tag{2}$$

where, L_s is the length of the lap-splices; d_{bl} is the diameter of a typical lap-spliced longitudinal bar; and f_s is the stress of the extreme tensile bar.

The constitutive law for the bond links is expressed by bond stress τ_b - slip S_b relationship, given by the following empirical equation.

$$\tau_b = \frac{\tau_{bc}' r (S_b / S_{bc})}{r - 1 + (S_b / S_{bc})^r} \tag{3}$$

where τ_{bc}' is the peak bond stress between the rebar and confined concrete; given as:

$$\tau_{bc}' = \tau_{b0}' + 1.4 f_l \tag{4}$$

in which $\tau_{b0}' = 9.5\sqrt{f_c'} / d_b \leq 800$ [3] (upper limit was ignored in the trial analysis); and f_l : stands for the transverse confining stress; 1.4 is shear friction coefficient. S_{bc}, is the bond slip corresponding to τ_{bc}'; and r is the parameter related to transverse confinement. The following empirical equations based on Giuriani et.al.[4] for determining S_{bc} and r are developed.

$$S_{bc} = S_{b0}\left(1 + \alpha \frac{f_l}{f'_c}\right) \tag{5}$$

where, S_{b0}=0.01in. (0.25mm); and α=75.0.

$$r = r_0 - k_r \frac{f_l}{f'_c} \tag{6}$$

where, r_0=2.0; and k_r=13.0.

Procedure: The analytical procedure involves trial and error loops for searching the correct values of neutral axis depth and the bond slip of each individual bar corresponding to each drift displacement increment at the top of the column. The equilibrium condition between bond and tensile force is used as the criteria in finding the bond slip of a lap-spliced bars. The equilibrium condition of the calculated internal axial force and the applied axial load is used as the governing criterion for determining the calculating step of correct neutral axis position in the critical section. Mander et.al.'s model is assumed as the constitutive law of the concrete throughout the analytical procedure.

Expected Products

The research conducted here will provide the bridge engineering practitioners with following helpful tools in designing new bridge or evaluating and retrofitting the existing bridges:

i. It provides a rational explanation and an accurate prediction of the behavior of columns with lap spliced longitudinal reinforcement.
ii. It suggests to establish a practical and safe design approach allowing some degree of debonding in lap splices of longitudinal reinforcement in order to prevent the total failure of the column due to a rupture of the longitudinal bars under excessive earthquake attacks.

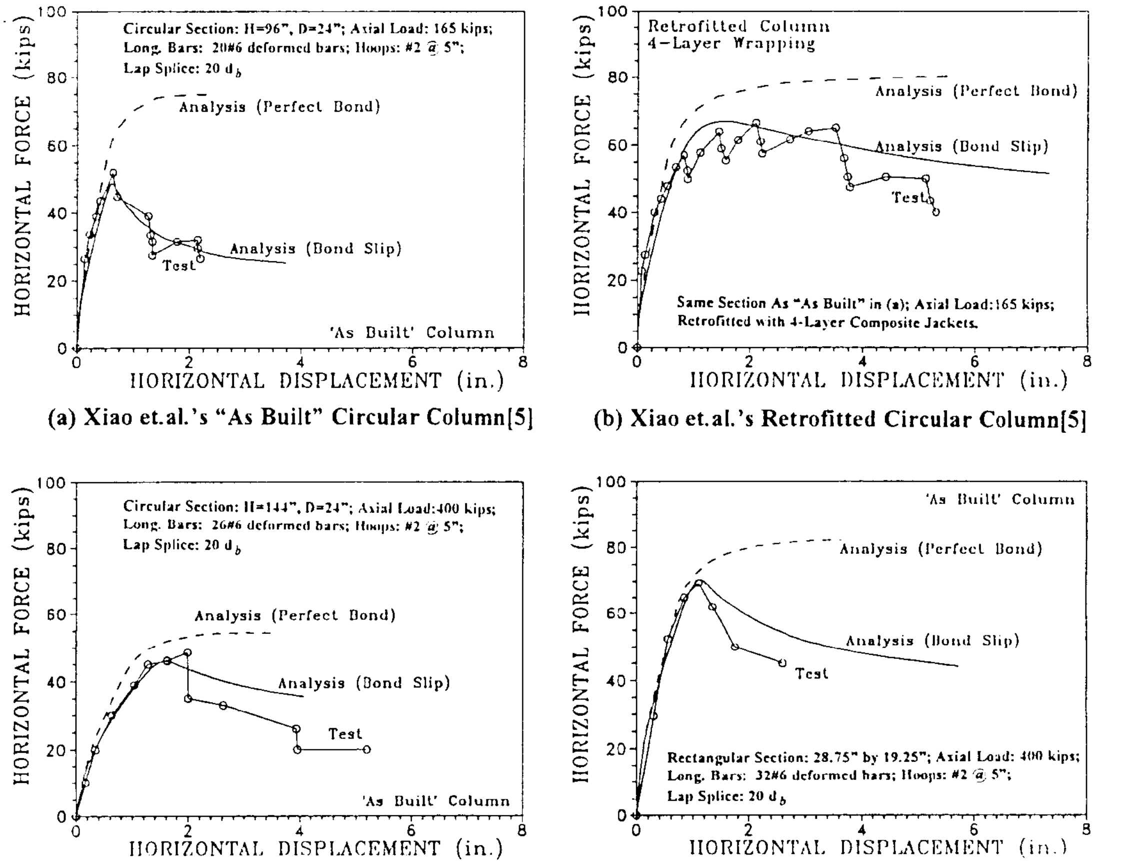
Preliminary Results

Analytical Results: Analyses have been conducted to simulate the horizontal force - displacement responses of the model columns tested at USC and UCSD. Fig.2 shows the comparisons of the analytical results and the test results of the horizontal force - displacement envelopes for the "as built" models with different section shape and reinforcement details and the retrofitted model column respectively. Solid lines in Fig.2 describe the analytical results predicted using the analytical model developed in this study, taking into consideration the bond slip of the lap-spliced longitudinal reinforcement. Dashed lines in Fig.2 show the results of the upper bound analysis in which bond slips of the lap-splices are ignored. It is clear that the analytical results using the bondslip model proposed here agree very well with the test results and are reasonably conservative. But the analytical model with perfect bond assumption produces non-conservative prediction to the test results. It is verified in this research that the proposed analytical model is a simple and satisfactory way in dealing with existing bridge columns.

Retrofit Design Recommendations: Based on the proposed analytical model discussed above, the following procedure is recommended for the retrofit design of bridge columns with lap-splices using jacketing.

i. Analyze horizontal force - displacement response and compute the ultimate ductility factor.
ii. Compare the calculated ductility factor of the existing column with the ductility demand estimated from structural analysis. If the calculated ductility exceeds the demand, then no retrofit is needed. Otherwise, the following retrofit design should be carried out.
iii. Assume a trial thickness for the retrofit jacket, and predict the ultimate ductility factor

for the retrofitted column. If the calculated ductility is less than what is required for retrofit, then jacket thickness should be increased and the corresponding ductility factor recalculated until the predicted ductility factor exceeds the required value.



(a) Xiao et.al.'s "As Built" Circular Column[5] **(b) Xiao et.al.'s Retrofitted Circular Column[5]**

(c) Priestley et.al.'s "As Built" Circular Column[2] **(d) Priestley et.al.'s "As Built" Rectangular Column[2]**

Fig.2 Comparisons of Analytical and Test Results

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FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Carbon Fiber Seismic Retrofit of Poorly Confined Square Reinforced Concrete Columns Subjected to Large Axial Forces

Author(s) and Affiliation(s):

John F. Stanton, Gregory A. MacRae and Kirk J. Nosho
Department of Civil Engineering
University of Washington, Seattle, WA

Principal Investigator: John F. Stanton

Sponsor(s): Nissho Iwai American Corporation and Tonen Corporation

Research Start Date: January 4, 1996
Expected Completion Date: June 4, 1996

Research Objectives:

The objective of this study is to provide a better understanding of the effectiveness of wrapping square reinforced concrete columns with carbon fiber in order to improve their seismic performance.

Expected Products or Deliverables:
The products expected for the study are an improved understanding of the behavior of square reinforced concrete columns retrofitted by carbon fiber wrapping; a means of estimating the quantity of carbon fiber needed to achieve a given curvature of drift level if flexural behavior controls; and preliminary data on shear strength of fiber-wrapped columns. Limitations in the scope of the project prevented study of other important variables, such as the presence of intermediate bars in the column faces, the influence of shear due to cyclic loading, etc.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

John F. Stanton is Professor of Civil Engineering at the University of Washington, where he teaches and conducts research in structural engineering. His research activities include earthquake engineering, and he is presently conducting studies on the deformation capacity of reinforced concrete columns, use of hybrid reinforcing consisting of a mixture of prestressing strand and deformed bar reinforcement for concrete seismic frames, and on the use of seismic restrainers in bridges. He has also worked in other areas, such as bridge bearings, structural stability and seismic isolation. Dr. Stanton is an active member of several committees of ACI and is serving on ACI and PCI task groups on the use of precast concrete in seismic zones. He is Director of the Structural and Geotechnical Engineering and Mechanics Program in Civil Engineering and the Structural Research Laboratory at the University of Washington.

Gregory A. MacRae graduated from the University of Canterbury, New Zealand in 1984 and worked as a structural engineer for one year before returning to his alma mater to carry out research on the seismic performance of steel frames. As a result of his experimental testing and computer simulations, he received a doctorate in 1989. During 1990 and 1991 he carried out experimental and analytical research into the seismic performance of hollow stiffened box columns at the Ministry of Construction research laboratory in Tsukuba, Japan. From 1992 to 1994 he worked at the University of California, San Diego, where he was involved in the bridge seismic retrofit program. He has been an Assistant Professor in the Department of Civil Engineering at the University of Washington since late 1994. His interests are earthquake engineering, design philosophy, steel structures, reinforced concrete, dynamic response and the assessment and retrofit of existing structures.

Kirk Nosho is a graduate research assistant at the University of Washington where he is an MSCE candidate. He has been responsible for most aspects of the experimental study described in this paper, including design and construction of the test specimens and the test apparatus and the conduct of the testing.

Principal Investigator's Telephone Number:	(206) 543-6057
Facsimile Number:	(206) 543-1543
E-mail Address:	stanton@u.washington.edu

CARBON FIBER SEISMIC RETROFIT OF POORLY CONFINED SQUARE REINFORCED CONCRETE COLUMNS SUBJECTED TO LARGE AXIAL FORCES

John F. Stanton[1], Gregory A. MacRae[1] and Kirk J. Nosho[1]

[1]Department of Civil Engineering
University of Washington, Seattle, WA 98195

Research Objectives

The research is being conducted to provide a better understanding of the effectiveness of wrapping square reinforced concrete columns with carbon fiber in order to improve their seismic performance.

Research Approach

Background: Steel jacketing has recently become popular for retrofitting both building and bridge columns. It is clearly effective, but under some circumstances, particularly if access is restricted or site welding is undesirable, application may be difficult. A considerable body of information has also been developed for fiberglass wrapping, but its long term strength is quite low compared to its short-term strength. Carbon fiber is stiffer and stronger than glass and has been used on some circular bridge columns. There the fiber has been spun around the column in a manner similar to that used for constructing some prestressed concrete tanks. Little data exists on the use of carbon fiber for wrapping square columns.

Buildings contain two classes of column that are potentially vulnerable to seismic damage. In older buildings (typically pre-1975), the seismic framing columns may contain inadequate transverse reinforcement. However these columns typically carry a relatively light axial load, on the order of $0.1f'_cA_g$. In buildings of all ages, the columns that are intended to carry only gravity loads, referred to here as "gravity columns", are often seriously under-reinforced transversely. Such columns exist in both shear wall and frame structures and this condition may exist even if the columns in the seismic frame are well designed. The 1994 Northridge earthquake demonstrated that a shortage of ties impairs the gravity columns' ability to accommodate seismic drift while carrying the axial load. High axial load significantly reduces drift capacity but the loads on gravity columns may be near the limit permitted by code of $P_u \approx 0.56f'_cA_g$. The number of under-reinforced gravity columns in service today is believed to be very large.

Transverse reinforcement is inherently less efficient in rectangular columns than in circular ones, so questions exist over the effectiveness of fiber-wrapping them. However the need for seismic retrofit of under-reinforced square or rectangular columns is extensive, because so many buildings contain them.

Experimental Program: An experimental program is in progress to investigate the cyclic drift capacity of square columns that carry high axial load.

The drift capacity of a column is affected by many parameters, such as geometry, material, reinforcement details and axial load. Not all of these could be investigated at once, so behavior in combined flexure and axial load was selected as the main focus. A decision was made subsequently to reuse parts of the specimens in monotonic shear tests.

The design of the specimens was based on the properties of the columns in an existing structure, built in 1971, which is about to be retrofitted. Those columns are either 22" square or 34" square and contain different amounts of reinforcement. The test columns were 11" square and contained

4#5 bars for longitudinal reinforcement, thereby representing the smaller prototype columns at exactly one half scale and the larger ones at approximately one third scale. Transverse reinforcement in the prototypes was #3 at 18", which was modeled in the laboratory by using smooth wire at 9" centers.

The design concrete strength in the structure was 4000 psi, but field tests suggested that the in-situ strength today lies between 6,000 and 10,000 psi. Aggregate size was 3/4". The test columns were thus made from a 3/8" aggregate mix which was designed to achieve a strength between 5500 and 6500 psi during the course of the tests. It contained no superplasticizer. The aspect ratio of the columns was selected so as to ensure failure in combined axial load and bending, and to preclude shear failure. This was done so that the drift capacities provided by various levels of fiber wrap could be compared, without the complications of allowing for different failure modes. The test columns are thus inevitably more slender than the prototypes. The test column corners were constructed with a 1/2" radius in order to avoid stress concentrations in the carbon fiber. In practice the corner of the column would have to be rounded by grinding prior to retrofitting.

The base of each column was attached to the laboratory strong floor as shown in Fig. 1 and the tip of the cantilever column was moved horizontally by a servo-controlled actuator. Displacement control was used for all horizontal motions. Axial (vertical) load was applied to the column through a steel cross head stressed down to the floor by two high-strength rods. The axial load was continuously monitored and kept constant.

The carbon fiber is supplied in sheets, in which all the structural fibers run in one direction. The fiber orientation before application is assured by the presence of a backing sheet of paper and very light diagonal fibers. The material is called Carbon Fiber Tow Sheet and it has a tensile strength of about 500 ksi and a stiffness comparable to that of steel. It is applied to the column with resin, which is spread in one layer beneath, and one over, each fiber sheet. The sheets were wrapped around the column with the fibers horizontal and were lapped by 4" on one side of the column. Two sheet strengths were used. C1-20 has a breaking strength of 2.2 kips/in width and C1-30 has 3.3 kips/ in. The two sheets are made from identical fibers, but the C1-30 sheet contain 50% more fibers per unit width.

The material comes in 20" wide rolls and was applied in 20 " bands up the column. The number of sheets in each band of each column is shown in Table 1. The half sheet of C1-20 used in Unit 4 was made by removing from the backing sheet 1/2" wide strips of fiber at 1" centers before applying the sheet to the column. The resulting 1/2" wide strips were considered to be close enough together compared to the column dimensions to provide the equivalent of a continuous sheet of half the thickness.

The most heavily loaded 22" prototype columns carry a load of 1000 kips. This results in a stress of 0.517 f'_c A_g if f'_c is taken as the original 4000 psi, but 0.344 f'_c A_g if f'_c is taken as the present 6000 psi. The axial load in the tests was chosen to be 0.333 f'_c A_g in order to represent approximately the latter condition. For each test f'_c was obtained by testing companion cylinders.

In all cases cyclic displacements were applied at increasing amplitudes until the column could no longer carry its axial load.

Expected Products

The products expected for the study are (1) an improved understanding of the behavior of square reinforced concrete columns retrofitted by carbon fiber wrapping, (2) a means of estimating the quantity of carbon fiber needed to achieve a given curvature or drift level if flexural behavior controls, and (3) preliminary data on shear strength of fiber-wrapped columns. Limitations in the scope of the project prevented study of other important variables, such as the presence of intermediate bars in the column faces, the influence of shear due to cyclic loading, etc.

Preliminary Results

The tests for flexure and axial loads are complete. The primary result, which is the horizontal deformation capacity of the column, is given in Table 1 for each test specimen. The deformation is expressed as a failure drift. It should be understood that a column that is reinforced identically to, but has a different aspect ratio than, the test columns would have a different drift capacity.

The pattern of behavior was generally that horizontal cracks in the resin occurred early in the loading history. As loading progressed, some local bulging could be seen on the compression side of the column. After a few more cycles of load the Tow Sheet fractured locally, continued to carry load for another half cycle, and then the column failed.

Specimen 2 was heavily reinforced with carbon fiber. During the initial load cycles it was accidentally displaced to 5% drift (the rig limit). However, it continued to carry the axial load without difficulty although some local bulging was visible on the compression side at the base of the column. The original loading history was then continued, and when the displacement once again reached 5%, the hysteresis loops were symmetric and displayed no sign of the previous damage. Because failure had not occurred, the axial load was then increased to $0.5f'_c A_g$, and the specimen was cycled at ± 5% drift. The Tow Sheet tore in the second cycle and, even though the column was still carrying the axial load, the test was stopped in the interests of safety. This loading combination is much more severe than that to be expected in practice, and demonstrates that carbon fiber wrapping can provide very high ductility capacities, despite the apparent unsuitability of the square shape. Specimen 3, retrofitted with one sheet of C1-20 Tow Sheet, easily achieved the target design drift of 2%.

No premature failures caused by stress concentrations at the columns corners were observed. However, not surprisingly, the failures in the Tow Sheet have generally started at the column corner. One of the advantages of the system is that the Tow Sheet possesses almost no strength perpendicular to the fiber lay direction. This means that fiber-wrapping does not increase the column flexural stiffness and strength, which in turn means that the applied shear forces at flexural failure are essentially the same as in the unretrofitted column. By contrast steel jacketing does provide vertical strength and so it increases the moment capacity of, and hence the shear demand on, the column.

Specimen no.	1	2	3	4
Retrofit	None	Heavy	Light	Very light
Band 4	none	1 @ C1-30	none	none
Band 3	none	1 @ C1-30	none	none
Band 2	none	3 @ C1-30	1 @ C1-20	1 @ C1-20
Band 1	none	4 @ C1-30	1 @ C1-20	0.5 @ C1-20
Stable cycle drift	1.5%	> 5%	2.5%	2.5%
Failure drift	1.75%	5% *	3.0%	3.0%

Table 1. Specimen Properties and Drifts Achieved.

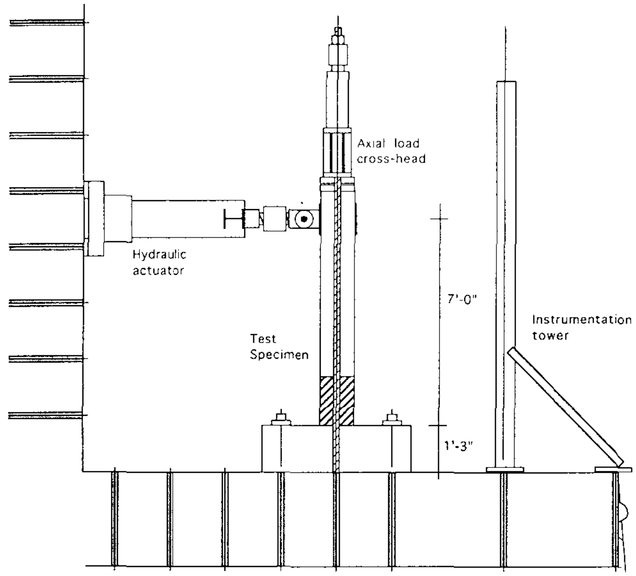

Fig. 1 Experimental Set-up.

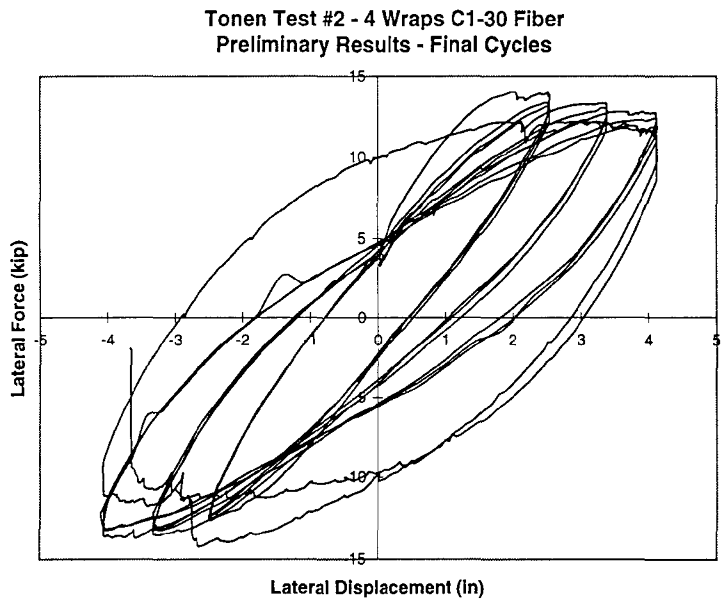

Fig. 2 Hysteresis Loop for Specimen 2.

Acknowledgments

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FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS RESEARCH ABSTRACT

Title: Pile to Pile Cap Connection Test Series

Author(s) and Affiliation(s):

Frieder Seible, M.J. Nigel Priestley and Pedro F. Silva
Department of Structural Engineering
University of California, San Diego, La Jolla, CA

Principal Investigator: Frieder Seible

Sponsor(s): California Department of Transportation

Research Start Date: June 1994
Expected Completion Date: August 1996

Research Objectives:

The main design characteristics investigated under this Caltrans research project are the development length and anchorage behavior of the longitudinal pile bars into the pile cap joint region; pull out of the steel jacket anchorage bars from the joint region; investigation of the levels of principal tensile stresses through the joint degradation; and development of joint force transfer mechanism models.

Expected Products or Deliverables:

The test results will be used as a guide to develop a consistent procedure for the seismic design of pile to pile-cap connections.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Frieder Seible is Professor of Structural Engineering at the University of California, San Diego. He received his Dipl. Ing. (Civil Engineering) from the University of Stuttgart, West Germany in 1976; his MSCE from the University of Calgary, Alberta, Canada in 1978; and his Ph.D. from the University of California, Berkeley in 1982. His research interests are in analysis and design of reinforced/prestressed concrete bridges, evaluation and rehabilitation of existing bridge structures and buildings, and development of new computer models to predict dynamic and static nonlinear response of reinforced and prestressed concrete structures under service overload and failure loads. Dr. Seible also has experience in verification of computer models by means of large or full-scale experimental testing, earthquake resistant design of reinforced concrete and concrete masonry structures, development of large-scale structural testing techniques, seismic assessment and retrofit of bridges, and the application of Polymer Matrix Composites (PMC) in civil structures. He has more than 170 papers and 90 technical reports mainly related to seismic design of bridges and buildings. Dr. Seible is a registered Professional Engineer in California.

M.J. Nigel Priestley is Professor of Structural Engineering at the University of California, San Diego. He received his B.E. in 1964 and his Ph.D. in 1967 in Civil Engineering from the University of Canterbury, New Zealand. His primary research interests are related to the design and analysis of structures to withstand seismic attack. In particular, his current research emphasis is on seismic assessment and retrofit of existing bridges; seismic design of concrete bridges; seismic design philosophy for precast concrete structures; seismic design of reinforced and unreinforced masonry structures; and the development of realistic dynamic testing methods to simulate inelastic response under seismic loading.

Pedro Franco Silva received his BSCE and MSCE from the University of California, Irvine. He is currently a Ph.D. candidate at the University of California, San Diego. His research interest is on the behavior and seismic design of reinforced concrete and masonry structures. He has had a variety of design experience since 1985.

Principal Investigator's Telephone Number: (619) 534-4640
Facsimile Number: (619) 534-6373
E-mail Address: seible@ucsd.edu

PILE TO PILE CAP CONNECTION TEST SERIES

Frieder Seible[1], M. J. N. Priestley[1], Pedro Franco Silva[2],

[1] Professor of Structural Engineering, [2] Graduate Research Assistant
Department of AMES, University of California at San Diego, La Jolla, CA 92092-0085

Research Objective

The main design characteristics investigated under this Caltrans research project are (1) the development length and anchorage behavior of the longitudinal pile bars into the pile cap joint region, (2) pull out of the steel jacket anchorage bars from the joint region, (3) investigation of the levels of principal tensile stresses through the joint as a measure of joint degradation, and (4) development of joint force transfer mechanism models. The test results will be used as a guide to draft a consistent procedure for the seismic design of pile to pile cap connections.

Research Approach

Background: Recent earthquakes around the world have caused extensive damage to concrete bridge structures. Most of the reported damage and collapse mechanisms to these types of structures were primarily on structural components above ground level [1]. However, uncertain prediction of the state of damage of those components below ground level may have left bridges with unidentified reduced capacities in the foundation system. This stems from the difficulty in post earthquake investigations of subsurface structures such as piles and pile caps. Thus, the performance and assessment of damage in bridge foundations are largely undocumented. As a result, Caltrans design engineers, in close collaboration with UCSD researchers, developed a research program to study the seismic response behavior of pile connections when the piles are embedded in soft soil sites.

An uncertainty in seismic bridge footing response is the actual load and deformation capacity of the pile to pile cap connections. In addition, design methods lack information to validate the actual capacity of the pile connections. For example, analysis of typical details show deficiencies in these connections when the piles are subjected to large tensile forces [2]. This research attempts to provide design recommendations for pile connections, and to detect distress levels and failure modes in the joint region under large seismic events.

Geometry and Design of Test Units: Current Caltrans practice does not specifically address the design of pile connections, but instead standard details are employed. To obtain a consistent procedure for the design of these types of connections four specimens are currently under experimental investigation at the University of California, San Diego. The design phase included the following: (1) selection of the pile test units, (2) height of the pile test units, (3) geometry of the pile cap, and (4) the applied axial and lateral loads.

Three standard Caltrans piles [3] were selected as the as-built test units. Test unit #1 is a precast prestressed class 70 ton pile, see Fig. 1A, test unit #2 is a composite steel jacket unreinforced concrete core class 70 ton pile, see Fig. 1B, and test unit #3 is a composite steel jacket reinforced concrete core class 200 ton pile, see Fig. 1C. The fourth test unit will be designed to investigate an improved force transfer mechanism for the design of test unit #2, see Fig. 1D.

Another issue of the design phase that required investigation was the pile cantilever length, which produces alike shear force demands in both the pile specimens and the prototype piles. To analyze the behavior of a pile embedded in soft soil and subjected to both lateral and axial loads, a 2-D finite element analysis was conducted on a model depicting the effects of soil-structure interaction for a single pile. The pile was modeled with 12" long beam elements with the pile head assumed constrained against rotation. To calculate the properties of each element an iterating analysis was performed. At each iteration element properties were updated according to the relation $EI = M/\phi$, obtained from moment curvature analysis. The soil was modeled as an array of uncoupled linear elastic springs according to the Winkler soil idealization [4] and positioned along the length of the pile. The idealized cantilever length of the pile test units was found from the top of the pile to the first point of contraflexure for the piles subjected to axial compression.

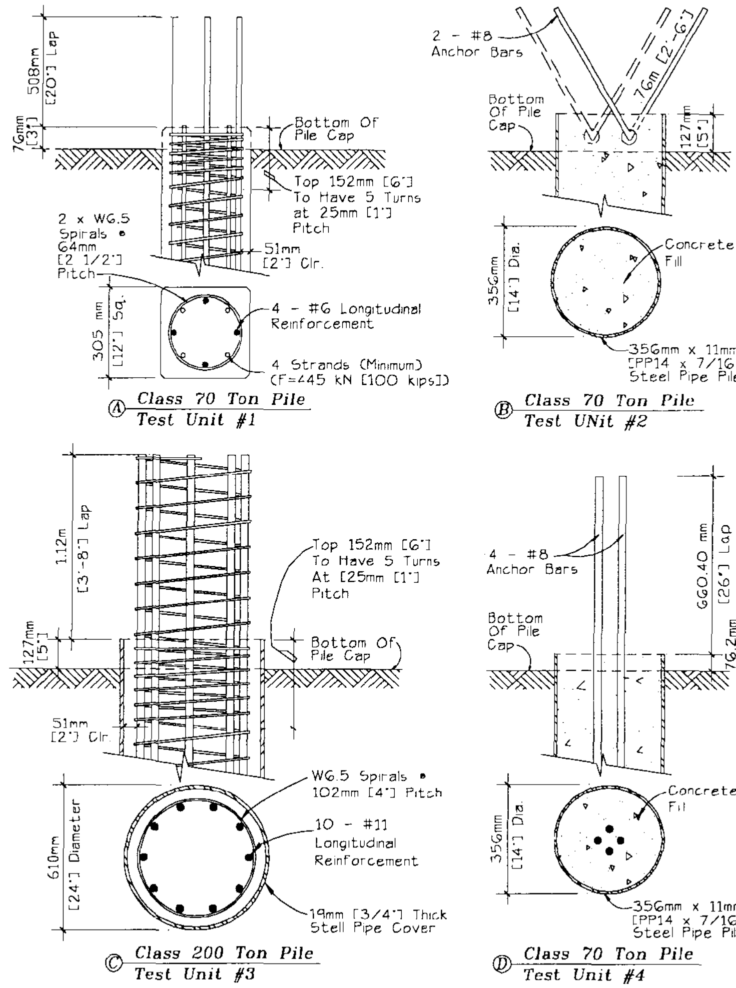

Fig. 1 Pile Test Units

The proposed pile cap geometry and dimensions were based on the route I-880 5th and 6th Street Viaduct Bent #6 (Cypress Viaduct Replacement Project in Oakland, CA). Since the critical pile in a pile group is a corner pile, each specimen was designed to correctly model the boundary conditions for typical corner piles embedded in soft soil. In soft soil the pile cap behaves much like a continuous body supported by the piles, with negligible vertical support from the underlying soil mass. To obtain this geometric boundary condition the pile caps were placed above the strong floor level and cantilevered from two support blocks, see Fig. 2.

Finally, applied axial and lateral loads on the specimens were computed to model the imposed forces on a corner pile. During any seismic event, the individual piles of a pile group experience reversed cyclic loading. The cyclic axial and lateral forces imposed on the pile section depend on the forces transferred from the bridge superstructure and columns into the pile cap, and subsequently into the pile group. Thus, a correlation between axial and lateral loads was obtained from a 3-D nonlinear finite element analysis of the prototype structure. To model the behavior of a single pile of a pile group the axial and the lateral loads were applied cyclically by means of three hydraulic actuators, see Fig. 2.

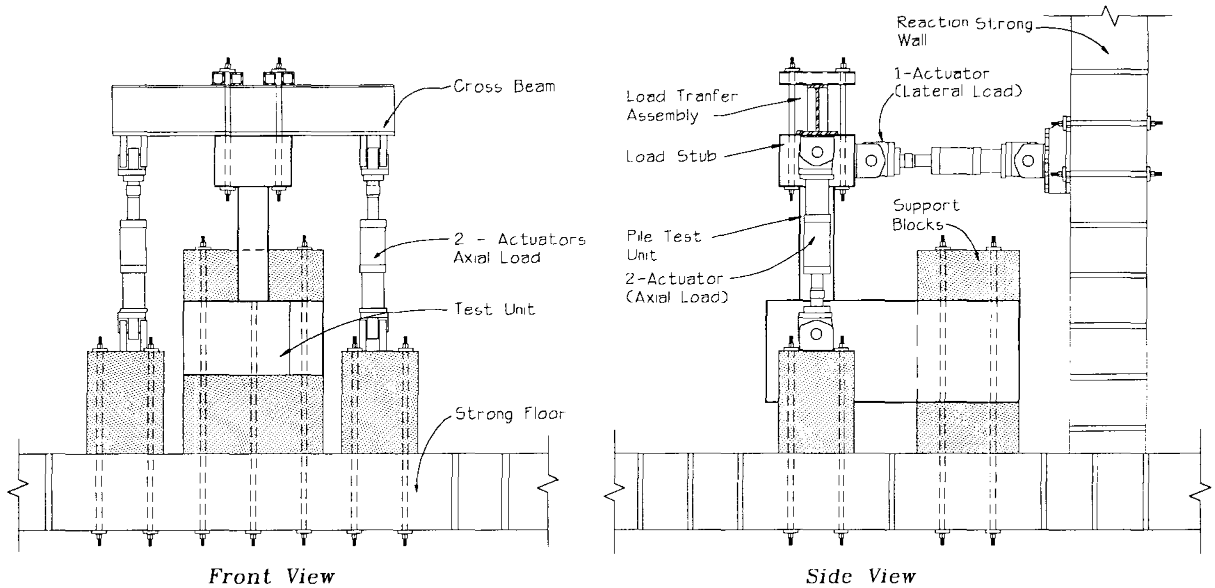

Fig. 2 General Test Setup

Preliminary Findings

The first successful test was conducted on unit #2, and a brief summary of the test results are described in this section. The seismic load simulation involved the application of fully reversed axial and lateral load cycles to the pile load stub. The horizontal load versus lateral displacement hysteresis is presented in Fig. 3. The cycles at higher load levels depict an increase of the pile top displacement while the lateral load remains nearly constant during the transition of the axial load from compression to tension. This behavior is caused by movement of the anchor bar hooks in the oversized holes of the steel shell. As the axial and lateral loads are applied to the pile, the load is transferred from the steel shell into the anchor bars. In the early loading stages, the concrete present around the holes is adequate to provide load transfer from the bars to the shell. However, at later loading stages, due to increasing load reversals, the concrete inside the holes is crushed and the load is then transferred only upon contact of the anchor

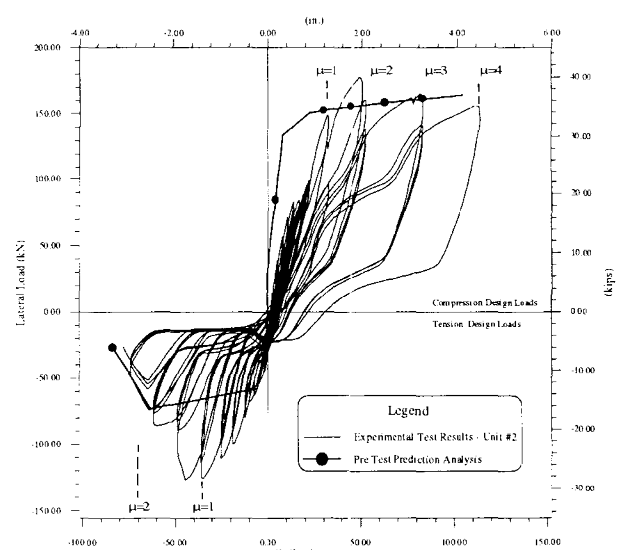

Fig. 3 Load-Displacement Hysteresis

bars with the top or bottom of the holes. During the load reversal process, the joint rotates, and the position of the holes relative to the anchor bars shifts. Thus, there is a stage in the load reversal process at which the anchor bars do not contact the holes an consequently no strain increase in the anchor bars is observed until they regain contact again with the steel shell. This phenomenon is well described in the diagram in Fig 4. In this diagram the strain in the anchor bar is plotted versus the pile top displacement.

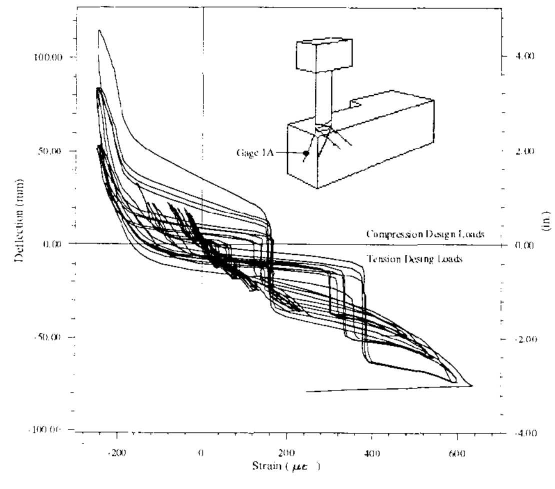

Fig. 4 Anchor Bar Strains

Additional observations during testing showed that the concrete cover of the pile cap around the steel shell was severely damaged, and exposure of the bottom layer of the pile cap reinforcement, near the pile, was observed. Failure of the pile occurred by sudden fracture of the anchor bars in the bar bends from bearing against the steel shell. Fracture of the anchor bars occurred at an axial tensile load of approximately 400 kN [90 kips]. As Fig. 5 shows this load is below the 623 kN [140 kips] tension design load proposed by Caltrans. In addition, this sudden failure resulted in a reduced ductility capacity of the pile-connection under axial tensile loads in comparison to the ductility capacity achieved in the compression side, see Fig. 3. Thus, any retrofitting or redesign measures must address the concrete cover damage, and the fracture of the anchor bars below the tensile design load.

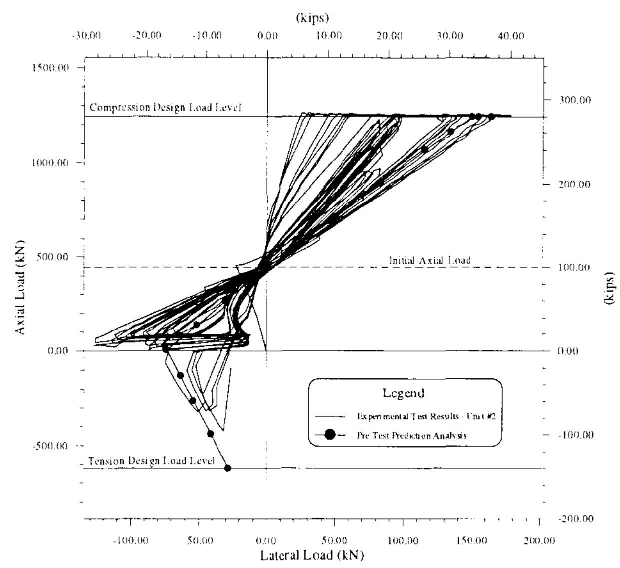

Fig. 5 Axial Load vs. Lateral Load

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[4] Priestley, M. J. N., Budek, A., Benzoni, G., "*An Analytical Study of the Inelastic Seismic Response of Reinforced Concrete Pile-Columns in Conhesionless Soil*", Department of AMES , UCSD, La Jolla, CA, March 1995.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS RESEARCH ABSTRACT

Title: Substructure Protection by Ductile End-Diaphragmsin Steel Bridges

Author(s) and Affiliation(s):

Seyed Mehdi Zahrai and Michel Bruneau
Ottawa-Carleton Earthquake Engineering Research Center
Department of Civil Engineering
University of Ottawa, Ottawa, Ontario, Canada

Principal Investigator: Michel Bruneau

Sponsor(s): Natural Sciences and Engineering Research Council, Canada

Research Start Date: May 1993
Expected Completion Date: December 1996

Research Objectives:

Steel bridges are frequently supported by seismically vulnerable substructures, as demonstrated by recent earthquakes. Current practice generally consists of seismically retrofitting these non-ductile substructures which can be, in many cases, a rather costly operation. In order to recognize the benefits provided by the presence of a steel superstructure, an innovative seismic retrofit strategy using ductile steel bridge end-diaphragms has been developed. By replacing the steel diaphragms over abutments and piers with specially designed ductile diaphragms (such as shear panels, eccentrically braced frames and TADAS devices) calibrated to yield before the strength of the substructure is reached, the substructure can be protected and need not be retrofitted, resulting in considerable savings.

Expected Products or Deliverables:

This project will provide the knowledge necessary to implement this innovative concept as an effective and economical retrofit for a type of slab-on-girder steel bridge commonly found in North America, and even as an effective passive energy dissipation system in new bridges. A simple hand calculation design procedure has already been developed to simplify this process, and preliminary non-linear inelastic analyses indicate satisfactory performance of the resulting designs during earthquakes. Experimental work will confirm the adequacy of the proposed design details.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Sayed Mehdi Zahrai is a Ph.D. candidate in Structural Engineering at the University of Ottawa where he has conducted research for the past four years on the seismic resistance of steel bridges. Prior to coming to Canada, Mr. Zahrai completed a M.A.Sc. at Tehran University in Iran studying the effect of wind on suspension bridges using aerodynamic non-linear analysis and considering P-δ effects. Mr. Zahrai also acquired practical industrial experience working for many consulting engineering firms in Iran over a four year period during which he was involved in the rebuilding of damaged bridges, the design and construction of a 300 meter long 120 meter tall earth dam, analysis and design of medium-rise buildings, and many other projects.

Michel Bruneau is an Associate Professor in the Department of Civil Engineering at the University of Ottawa, and is also Director of the Ottawa Carleton Earthquake Engineering Research Centre. Dr. Bruneau obtained M.Sc. and Ph.D. degrees from the University of California, Berkeley, with a specialization in earthquake engineering. He has experience working for the consulting firms of Morrison Hershfield Limited (North York, Ontario), and Buckland and Taylor Limited (North Vancouver, B.S.). His current research activities concentrate on the seismic evaluation and retrofit of existing steel bridges, steel buildings, and masonry buildings. On-going projects include the seismic evaluation and ductile retrofit of large steel deck-truss bridges, seismic ductile retrofit of steel highway bridges, seismic resistance of steel-plate retrofitted shear walls, seismic evaluation and retrofit of rivetted stiffened seat angle beam-to-column steel connections, and pseudo-dynamic testing of unreinforced masonry buildings having flexible wood diaphragms. He has also conducted reconnaissance visits to six earthquake stricken areas, in recent years. Dr. Bruneau is a member of the Seismic Committee of the Canadian Highway Bridge Design Code. He was awarded the 1996 Glinski Award for Best Researcher in the Faculty of Engineering at the University of Ottawa, and the Gzowski Medal for best paper in the Canadian Journal of Civil in 1994. He is also the first recipient of the University of Ottawa Young Researcher Award (1996).

Principal Investigator's Telephone Number:	(613) 562-5800 Ext. 6127
Facsimile Number:	(613) 562-5173
E-mail Address:	bruneau@genie.uottawa.ca

SUBSTRUCTURE PROTECTION BY DUCTILE END-DIAPHRAGMS IN STEEL BRIDGES

Seyed Mehdi Zahrai[1] and Michel Bruneau[2]

[1]Graduate Research Assistant, Ottawa-Carleton Earthquake Engineering Research Centre
Department of Civil Engineering, University of Ottawa, 161 Louis Pasteur, Ottawa, Ontario, Canada K1N 6N5
[2]Associate Professor, Ottawa-Carleton Earthquake Engineering Research Centre
Department of Civil Engineering, University of Ottawa, 161 Louis Pasteur, Ottawa, Ontario, Canada K1N 6N5

Research Objectives

Steel bridges are frequently supported by seismically vulnerable substructures, as demonstrated by recent earthquakes (e.g. Bruneau et al. 1996; Astaneh-Asl et al. 1994; Roberts 1992). Current practice generally consists of seismically retrofitting these non-ductile substructures; this can be, in many cases, a rather costly operation. In order to recognize the benefits granted by the presence of a steel superstructure, an innovative seismic retrofit strategy using ductile steel bridge end-diaphragms has been developed. By replacing the steel diaphragms over abutments and piers with specially designed ductile diaphragms (such as shear panels, eccentrically braced frames and TADAS devices) calibrated to yield before the strength of the substructure is reached, the substructure can be protected and need not be retrofitted, resulting in considerable savings.

Research Approach

The ductile end-diaphragm retrofit strategy proposed was developed in a capacity design perspective. This design philosophy consists of forcing all structural inelastic deformations to develop only in some judiciously selected structural elements specially detailed to absorb all seismic energy. Hence, forces in a given structure cannot exceed those present at development of the plastic mechanism, and all parts of the structure for which inelastic action is not needed for development of that mechanism do not require ductile detailing. The special ductile elements can be seen as "structural fuses", all other elements being "capacity protected". The innovative idea here is to locate these structural fuses into the end-diaphragms of steel superstructures to prevent damage from developing in the non-ductile substructural elements, foundation, and bearings. Therefore, the special end-diaphragms must be designed such that stable hysteretic ductile behavior will develop and be sustained at a reliable pre-determined load level lower than that the threshold of unacceptable damage of the substructure, as schematically illustrated in Fig. 1.

Past research has demonstrated the effectiveness passive seismic energy dissipation devices in buildings, and many such experimentally proven systems have been implemented already. Among those, eccentrically braced frames (EBF) (e.g. Engelhardt and Popov 1989), shear panel systems (SPS), and steel triangular plates added damping and stiffness devices (TADAS) (e.g. Tsai 1993) have received a particular attention in building applications. Still, to the authors' knowledge, none of these applications have been used to date in bridge applications. This can be partly attributable to the absence of seismic ductile steel detailing provision in North American bridge codes.

Expected Products

This testing program will produce the necessary knowledge to implement this innovative concept as an effective and economical retrofit for a type of slab-on-girder steel bridge commonly found in North America, and even as an effective passive energy dissipation system in new bridges. A simple hand calculation design procedure has already been developed to simplify this process, and preliminary non-linear inelastic analyses indicate satisfactory performance of the resulting designs during earthquakes. Experimental work will confirm the adequacy of the proposed design details.

Preliminary Results

Examples of how the proposed passive energy dissipation systems would be implemented in the end diaphragms of a typical bridge are conceptually shown in Fig. 2 for a shear panel system, eccentric braced frame, and TADAS system respectively. Note that bearings stiffeners of the main girders would sometimes have to be trimmed or replaced by narrower ones for reasons described elsewhere (Zahrai and Bruneau 1996). Also, to satisfy the needs of a few transportation agencies who require that diaphragms be designed to accommodate jacking forces at pre-designated points for lifting of the bridge during future bearings replacement or other repairs, some additional holes and gusset plates can be optionally introduced (as shown in Fig. 2) to permit the use of temporary bracing members for that purpose. Note that this retrofit solution is limited to bridges that do not have horizontal wind bracing connecting the bottom girder flanges as these braces could provide an alternative load path bypassing the special ductile elements. Furthermore, this retrofit method provides enhanced seismic resistance and substructure protection for the component of seismic excitation transverse to the bridge, and must be coupled with other devices that constrain longitudinal seismic displacements; transportation agencies experienced in seismic bridge retrofit have indicated that deficiencies in the longitudinal direction are typically much easier to address.

To provide computational efficiency and to allow formulation of a simple design procedure, a simplified 2-D model capturing the essence of the 3-D behavior of slab-on-girder bridges was developed. The proposed simplified 2-D model, shown in Fig. 3, consists of the ductile end diaphragm, a stub-length of two girders with their bearing stiffeners and modeled as plane flexural members, a rigid stub of the reinforced concrete deck, and a small mass/spring subsystem located at deck level and introduced to account for the longitudinal generalized mass and stiffness effects.

While the above 2-D model can be implemented directly for computer analyses, its simplicity makes it also suitable for hand calculations. Indeed, recognizing that the generalized stiffness of the entire bridge, K^*, and the total lateral stiffness of the end-diaphragms, K_t, are linked together as springs in series, the equivalent stiffness, K_e, could be written as:

$$K_e = \frac{K^* K_t}{K^* + K_t} \tag{1}$$

Note that, in some instances, the flexural resistance of the girders can potentially contribute to the lateral load resistance of the ductile system, and even to its energy dissipation capability, depending on the relative rigidities of the components of this diaphragm. If that is the case, a tri-linear hysteretic model must be considered, and a complex relationship between ductility derived using principles of equal energy must be used (previous studies by Dicleli and Bruneau 1995 showed that steel bridges of interest here have a low transverse fundamental period of vibration, justifying the use of the principle of equal energy here). Otherwise, a conventional bi-linear hysteretic behavior and relationship between ductility, μ, and the force reduction factor, R, can be used. Refer to Zahrai and Bruneau (1996) for additional information on the effect of plate girders stiffness and its potential contribution to the diaphragm's dual system behavior, as well as closed-form formulations for the above stiffness terms for various types of ductile diaphragms.

The step-by-step procedure for the design of ductile end-diaphragms requires the following steps: (i) Determine basic design parameters (mass, seismic acceleration, geometry, etc.); (ii) Calculate generalized mass and stiffness parameters; (iii) Calculate elastic and inelastic seismic base shear resistance of the diaphragm, considering design criteria and limiting capacity of substructure, and the portion of that shear to be resisted by the energy dissipation device (for trilinear hysteretic systems, this typically require an iterative calculation); (iv) Provide proper ductile detailing for the selected energy dissipating device, and design all other structural members and connections of the diaphragm to be able to resist forces 50% greater than that resulting from yielding of the ductile device; (v) Calculate the resulting period of the ductile end-diaphragm, and if different from that assumed in step iii, repeat the appropriate above steps until convergence; (vi) determine the actual force reduction factor, using the appropriate bilinear or trilinear behavior of the total diaphragm system, and compare with target design value; (vii) repeat until a satisfactory value is obtained for that force reduction factor (usually requires 2 or 3 iterations).

Note that for a slab-on-girder bridge having 4 girders for example, it is in theory possible to insert 3 energy dissipating ductile diaphragms at each end of a span if bracing is introduced between all girders, but, for short bridges, better designs and more practical member sizes for the key components of the ductile diaphragms are obtained when a single diaphragm panel is introduced at each end of a span, irrespectively of the number of girders present.

To date, the concept has been demonstrated to work using non-linear inelastic analyses. The types of bridges that most benefit from this retrofit strategy have been identified, and a simple and reliable design procedure (applicable for hand calculations) has been developed. Effective ductile diaphragms have been developed using shear panels, TADAS systems, and eccentric braced frames, but other systems are possible using the principles formulated. A program of 16 tests of full scale ductile diaphragms (to be subjected to conventional reversed cyclic testing as well as pseudo-dynamic loading) is planned to begin in early June 1996. This will be followed by testing of a 1:5 scale full-bridge specimen (late Fall 1996) to further validate findings on global behavior.

Acknowledgment

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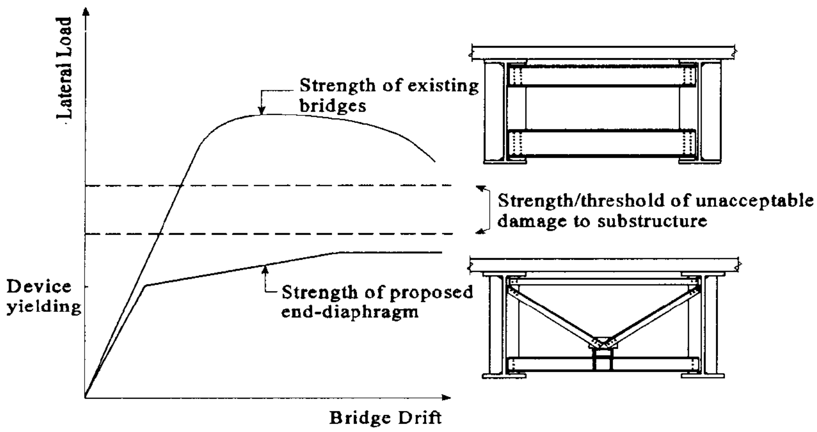

Fig. 1. Inelastic behavior of proposed ductile end-diaphragms compared to existing steel bridges

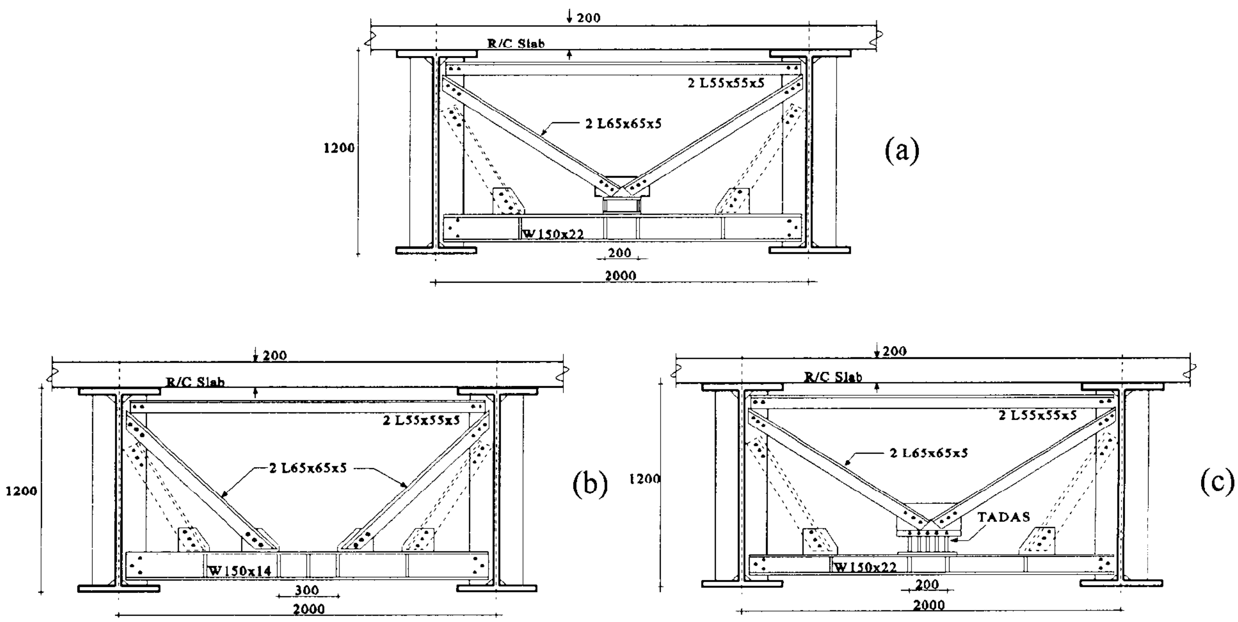

Fig. 2. Proposed ductile end-diaphragm of a typical bridge: a) SPS, b) EBF, c) TADAS

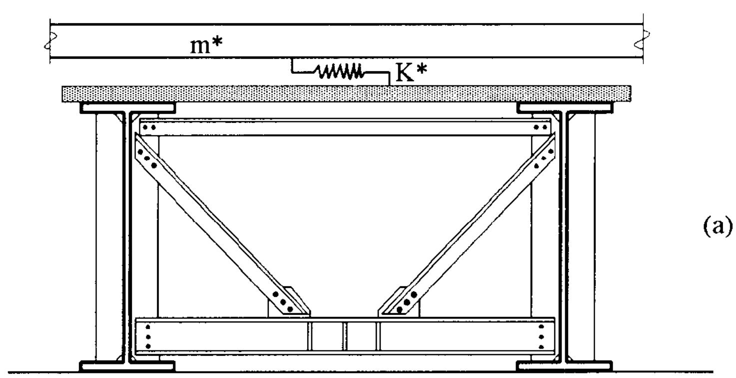

Fig. 3. 2-D Bridge Model

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Design of Seismic Restrainers

Author(s) and Affiliation(s):

Marc O. Eberhard, John F. Stanton, and Panos Trochalakis
Department of Civil Engineering
University of Washington, Seattle, WA

Principal Investigator: Marc O. Eberhard

Sponsor(s): Washington State Department of Transportation

Research Start Date: December 1994
Expected Completion Date: December 1996

Research Objectives:

This research is being conducted to evaluate the effectiveness of seismic restrainers, identify bridge characteristics that are likely to induce large relative displacements at hinges, abutments and simple supports, and develop (or modify) restrainer design methods.

Expected Products or Deliverables:

The expected products of this study will be an improved understanding of the efficacy of seismic restrainers; an improved understanding of the factors that lead to large relative displacements at hinges, abutments and simple supports; and improved design methods.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Marc Eberhard is Assistant Professor of Civil Engineering at the University of Washington. He is currently conducting research on the design of seismic restrainers in bridges, the effects of ground-motion incoherency on bridge response, the use of high-performance concrete in bridges, and nondestructive evaluation of reinforced concrete structures. Previous research includes experimental and analytical studies of the lateral-load response of reinforced concrete buildings and bridges. He received the National Science Foundation Presidential Young Investigator Award in 1991, was named the University of Washington Neilson Faculty Fellow in 1993, and received the ASCE Raymond C. Reese Research Prize in 1994. He is a member of several ACI committees that focus on earthquake engineering and nondestructive evaluation.

John Stanton is Professor of Civil Engineering at the University of Washington, where he teaches and is engaged in research in structural engineering. His research activities include earthquake engineering, and he is presently conducting studies on the deformation capacity of reinforced concrete columns, the use of hybrid reinforcing consisting of a mixture of prestressing strand and deformed bar reinforcement for concrete seismic frames, and the use of seismic restrainers in bridges. He has also worked in other areas, such as bridge bearings, structural stability and seismic isolation. Dr. Stanton is an active member of several committees of ACI and is serving on ACI and PCI task groups on the use of precast concrete in seismic zones. He is Director of the Structural and Geotechnical Engineering and Mechanics Program in Civil Engineering and the Structural Research Laboratory at the University of Washington.

Panos Trochalakis is a structural engineer with KPFF Associates, Seattle, Washington. He has also worked with the Washington State Department of Transportation. As part of his thesis studies at the University of Washington, he studied the seismic response of bridges with in-span hinges. He earned his MSCE degree in 1995.

Principal Investigator's Telephone Number:	(206) 543-4815
Facsimile Number:	(206) 543-1543
E-mail Address:	eberhard@u.washington.edu

DESIGN OF SEISMIC RESTRAINERS

Marc O. Eberhard, John F. Stanton and Panos Trochalakis

Department of Civil Engineering
University of Washington, Seattle, WA 98195-2700

Research Objectives

Seismic restrainers have been used in the United States for about 25 years. In continuous bridges, restrainers are installed to reduce relative displacements across in-span hinges during earthquakes. In bridges with simply supported spans, seismic restrainers are installed to reduce relative displacements between the spans and their supports. Despite the frequent use of restrainers, doubts remain about their efficacy, and recent research suggests that current design methods are inadequate (Saiidi et al. 1992, Yang, 1994). Therefore, the Washington State Department of Transportation sponsored this study to evaluate the effectiveness of seismic restrainers, to identify bridge characteristics that are likely to induce large relative displacements at hinges, abutments and simple supports, and to develop (or modify) restrainer design methods.

Research Approach

It is difficult to predict the seismic response of any bridge, because its response depends on the stiffness, strength, toughness and mass of the bridge elements, the resistance provided by the abutments and bent foundations, as well as the ground motion. It is even more difficult to predict the response of bridges with discontinuities, such as hinges and intermediate simple supports. The response of these bridges depends on the nonlinear response of the restrainers (with an initial gap) and the pounding that occurs at the joints. Moreover, ground motion incoherency is likely to have a significant influence on the relative displacements between adjacent frames and spans. The response of skewed and curved bridges is even more complex, because their behavior is inherently two-dimensional.

Despite this complexity, tractable methods for designing seismic restrainers are needed. In this study, three families of nonlinear models are being assembled to represent the key elements of bridge response. For each model type, a parametric study is being conducted by varying the model properties and ground motions. The goals of each parametric study are to evaluate the effectiveness of restrainers, to identify the parameters that most affect relative displacements at hinges, and to provide data with which to calibrate new design methods.

<u>Bridges with In-Span Hinges</u>: A family of one-dimensional models was assembled using the program DRAIN-2DX (Prakash 1993) to study the longitudinal response of straight bridges with in-span hinges (Trochalakis et al. 1996). The model shown in Fig. 1 represents the nonlinear force-deformation relationships of the frames (K1, F1, K2, F2), abutments (Gca, Ka, Fa), and hinge (Kr, Gr, Gch). The parametric study was conducted by varying the ground motions, the frame weights (W1, W2), and the element force-deformation relationships. For each of 216 cases considered, the maximum relative displacement at the hinge and abutments was noted.

Bridges with Simply Supported Spans: The second series of one-dimensional models was assembled to consider the longitudinal response of bridges with simply supported spans. The model represents the same contributors to response that the in-span hinge model represented. In addition, the model for simply supported bridges represents the nonlinear force-deformation relationship of the bearings that support the girders. For each of 47 cases considered, the maximum relative displacement between the span and its support was noted.

Effect of Skew: The third series of models is being assembled to study the influence of skew on displacements at hinges and abutments. This series of models considers the two-dimensional, nonlinear response of simple bridges with a variety of geometries, abutment properties and ground motions.

Ground-Motion Incoherency: The parametric studies will be repeated with new ground motions to consider the effect of ground-motion incoherency on relative displacements. A first set of analyses will use ground motions developed following the recommendations of Abrahamson (1992). A second set of analyses will be performed to evaluate the effects of topography and soil-structure interaction on the relative displacements.

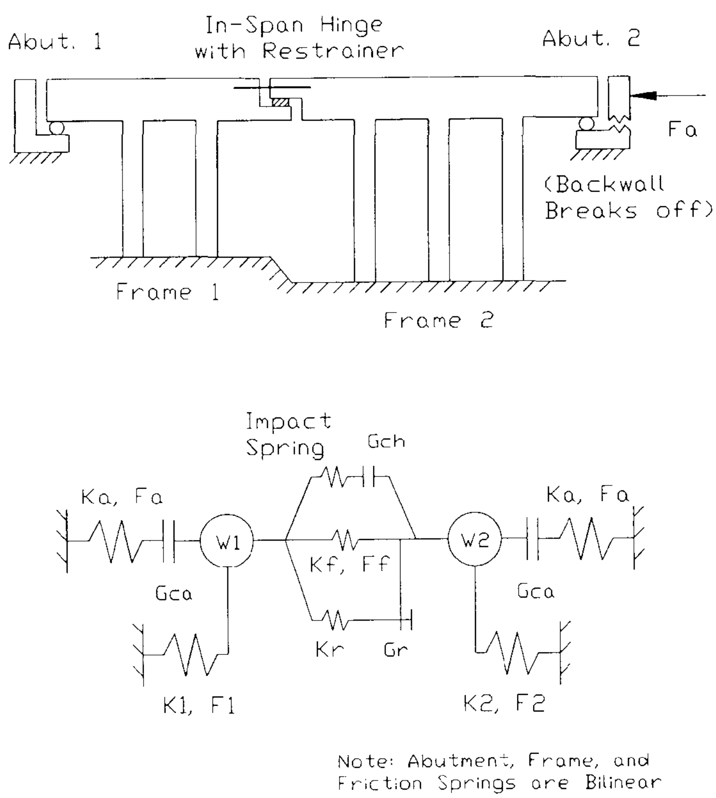

Figure 1. Prototype and Analytical Model

Expected Products

The expected products of this study will be (1) an improved understanding of the efficacy of seismic restrainers, (2) an improved understanding of the factors that lead to large relative displacements at hinges, abutments and simple supports, and (3) improved design methods.

Preliminary Results

The parametric study of bridges with in-span hinges has been completed for coherent ground motions (Trochalakis et al. 1996). The results of 27 analyses are summarized in Fig. 2. In this figure, the maximum relative hinge displacement (MRHD) and the maximum relative abutment displacement (MRAD) are plotted on the vertical axes. The ratio of the stiffness of the right frame to the stiffness of the left frame (K2/K1) is plotted on the horizontal axis. All other properties of the model and ground motion are constant. The figure shows that restrainers (represented by stiffness Kr) reduce hinge displacements, but that the abutment displacements are unaffected. The figure also shows that the hinge displacements are the smallest when the frame stiffnesses are identical. For this condition, the frames vibrate in phase.

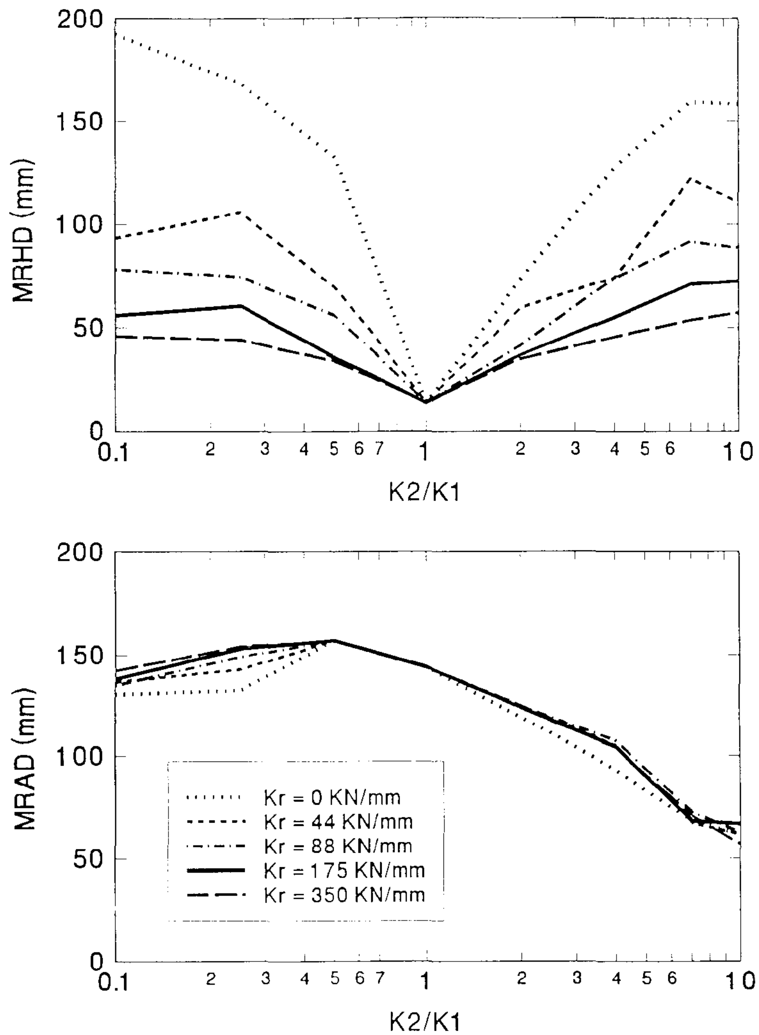

Figure 2. Effect of Frame Stiffness and Restrainer Stiffness on Relative Displacements

A method has been developed for designing restrainers at in-span hinges. The proposed method is similar to the method used by the California Department of Transportation ("Seismic" 1990), but it uses a different equation to compute the maximum relative hinge displacement (MRHD), and it provides better results. The maximum relative hinge displacement is calculated as follows:

$$\mathrm{MRHD} = \frac{D_1 + D_2}{2} \frac{T_L}{T_S}$$

where D_1 and D_2 are the nominal displacement demands for the frames on the left and right sides of the hinge. These displacement demands are calculated in the same manner as that advocated by the California Department of Transportation. T_L is the larger of the corresponding equivalent periods, and T_S is the smaller equivalent period. The displacements predicted with the design method (Fig. 3) correlate well with those computed by nonlinear time-history analysis (NLTH).

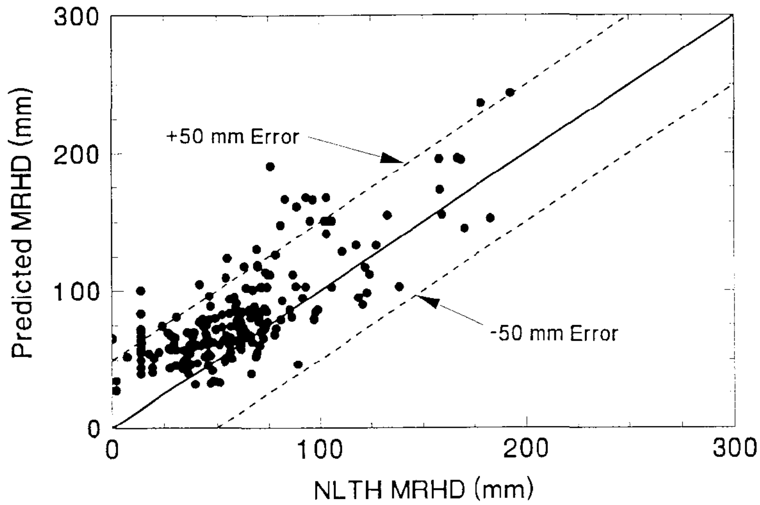

Figure 3. Comparison of Proposed Method with Nonlinear, Time-History Analysis

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FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Field and Laboratory Studies on the Seismic Performance of Bridge Systems

Author(s) and Affiliation(s):

John B. Mander, Stuart S. Chen, Dae-Kon Kim and Daniel A. Wendichansky
Department of Civil Engineering
State University of New York at Buffalo, Buffalo NY

Principal Investigator: John B. Mander and Stuart S. Chen

Sponsor(s): Federal Highway Administration and the National Center for Earthquake Engineering Research

Research Start Date: July 1993
Expected Completion Date: December 1996

Research Objectives:

The objective of this research is to investigate, both experimentally and analytically, the seismic performance of slab-on-steel girder bridges before and after rehabilitation of an in-service bridge with elastomeric bearings. The original bearings used in this bridge consist of a variety of low (sliding) and high (rocker) bearings. According to the recently published FHWA Seismic Retrofitting Manual such bearings are prime candidates for replacement due to their historically poor performance in earthquakes. The objectives of this research are to investigate at what level of seismic excitation steel bearings perform satisfactorily, whether simple retrofits to the steel bearings themselves markedly improve overall seismic resistance, and to assess the performance of different types of elastomeric bearings with and without supplemental energy dissipation capabilities.

Expected Products or Deliverables:
Recommendations regarding experimental investigation of the seismic performance of bridges of moderate size using tension-based snap back testing; development of a hybrid time domain-frequency domain system identification procedure for analyzing highly damped non-linear systems with closely spaced frequencies; experimental data for bridges seated on different bearing types possessing different degrees of nonlinearity; and mathematical modeling techniques for steel bearings as well as elastomeric bearings including temperature and strain rate effects.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

John B. Mander is an Associate Professor of Civil Engineering at the State University of New York, Buffalo, New York. In his research he has conducted numerous field and laboratory experiments on non-ductile bridges, substructure elements, bearings and dampers.

Stuart S. Chen is an Associate Professor of Civil Engineering at the State University of New York, Buffalo, New York. His research interests include field experimentation of prototype structures for earthquake, wind and traffic loading effects, and developments in applied artificial intelligence.

Dae-Kon Kim is a Ph.D. candidate in the Civil Engineering Department at the State University of New York, Buffalo, New York. He has performed research on seismic isolation and steel bearings. His research interests also include plastic and seismic analysis of buildings and bridges.

Daniel A. Wendichansky, recently completed his Ph.D. degree in the Department of Civil Engineering at the State University of New York, Buffalo, New York. Dr. Wendichansky is now an Assistant Professor in the Engineering Department at the University of Puerto Rico. His research interests are in experimental dynamics and large scale field testing and analysis of structures.

Principal Investigator's Telephone Number:	(716) 645-2114 Ext. 2418
Facsimile Number:	(716) 645-3733
E-mail Address:	jmander@ubvms.cc.buffalo.edu

FIELD AND LABORATORY STUDIES ON
THE SEISMIC PERFORMANCE OF BRIDGE BEARINGS.

J.B. Mander, S.S. Chen, D-K Kim and D.A. Wendichansky

Department of Civil Engineering
State University of New York at Buffalo, Buffalo NY 14260

Research Objectives

The principal objective of this research is to thoroughly investigate, both experimentally and analytically, the seismic performance of slab-on-steel-girder bridges before and after rehabilitating with elastomeric bearings. The original bearings used in this class of bridge (that is typical of bridges constructed in the central and eastern United States prior to the 1980's) generally consist of a variety of low (sliding) and high (rocker) bearings. According to the recently published FHWA Seismic Retrofitting Manual (Buckle and Friedland, 1995) such bearings are prime candidates for replacement due to their historically poor performance in earthquakes. The objectives of this research are three, to investigate: at what level of seismic excitation do steel bearings perform satisfactorily; whether simple retrofits to the steel bearings themselves markedly improve overall seismic resistance; and assess the performance effectiveness of different types of elastomeric bearings with and without supplemental energy dissipation capabilities.

Research Approach

Field Experiments: Two (sister) three-span slab-on-steel-girder bridges on the Rte 400 highway were rehabilitated by the NYSDOT. These bridges, shown in figure 1, were used to perform large force/large displacement quick release (snap back) tests to experimentally investigate the dynamic response both before and after replacing the steel bearings. The Northbound bridge bearings were replaced with conventional elastomeric bearing pads, and the Southbound bridge with seismic isolation bearings.

Laboratory Experiments: During the course of bridge rehabilitation steel bearings were retrieved from the field and destructively tested in the laboratory under gravity simulated constant axial load and earthquake simulated reversed cyclic lateral loading. The bearings were first tested with strong anchorages to investigate stability and strength issues pertaining to the bearings themselves. Secondly, the bearings were tested on reinforced concrete pedestals using the original swedged anchor bolts (also retrieved from the field) to investigate the steel bearing-anchorage-concrete pedestal system interaction.

Analytical Studies: The analytical studies have focused on conducting non-linear dynamic time history analyses of the two bridge seated on the different types of steel and elastomeric bearings. To enable such analyses to be conducted it was first necessary to make accurate force-displacement models of the steel and elastomeric bearings to capture the highly non-linear behavior of these elements. Various combinations of non-linear truss and link elements (that include gap effects) have been used for the steel bearings to capture sliding, prying, and keeper plate and/or anchor bolt fracture. A new thermo-visco-elasto-plastic model has also been developed for modeling elastomeric bearing behavior both with and without a lead core. With these primary sources of non-linear bridge performance accurately modeled the next step was to develop overall non-linear mathematical/structural models of the bridges. For this purpose the experimental quick release test time histories were used to validate the computational models. Once validated the computational models were used to predict expected performance for a large range of strong earthquake ground motions — not only motions that may be expected in low to medium seismic zones, but also very strong near-field motions that include fling effects observed in high seismic zones. In addition to investigating seismic behavior using non-linear time history analysis, simplified analysis techniques were advanced that use: (i) linearized elastic capacity-demand spectra, and (ii) non-linear inelastic capacity-demand spectra.

Expected Products and Deliverables

(i) Recommendations regarding experimentally investigating the seismic performance of bridges of moderate size using tension-based snap back testing.

(ii) Development of a hybrid time domain-frequency domain system identification procedure for analyzing highly damped non-linear systems with closely spaced frequencies.

(iii) A wealth of experimental data for bridges seated on different bearing types possessing different degrees of nonlinearity. Such experimental results can be used as a benchmark for validating future computational modeling developments.

(iv) Mathematical modeling techniques for steel bearings as well as elastomeric bearings including temperature and strain rate effects.

(v) Recommendations on how to model this class of slab-on-girder bridge structure using both rigorous non-linear time history techniques, as well as simplified capacity-demand spectrum approaches, for a wide range of seat conditions.

Preliminary Results

Steel Bearings: Bearing behavior can be largely described by rigid body kinematics (i.e. sliding and rocking) with some yielding of critical parts such as anchor bolts, pintles and guide plates. For each type of steel bearing where sliding is possible, the laws of Coulomb friction are obeyed. Results show that by retrofitting the existing high type steel bridge bearings it is possible to provide sufficient strength and displacement capability to withstand substantial ground shaking. The weak link in the chain of force transmission may thus become the anchor bolts and/or the reinforced concrete pedestal. Experimental results demonstrate the importance of considering the flexibility of the concrete pedestal-anchor bolt system. The bearing assembly elastic stiffness may be determined by assessing the flexural and shear flexibility of each of the constituent bearing parts. Ultimate lateral strength may be determined using either rigid body kinematics or upper bound plastic mechanism analysis.

Elastomeric Bearings: At extremely cold temperatures, rubber glass-hardens with the bearing behaving in a significantly non-linear fashion. The hysteretic performance of elastomeric bearings may be characterized by a temperature dependent non-linear Kelvin model — that is a bi-linear spring and velocity dependent nonlinear viscous dashpot. The cold temperature effects of lead-rubber bearings may also be obtained by similarly adding the visco-elasto-plastic effects of lead to the model of the rubber bearing.

Field Experimentation Observations: The replacement of the original steel bearings with elastomeric isolation bearings produced a significant change in the transverse dynamic characteristics of the bridge. The initial transverse frequency of the Southbound bridge was around

5.5 Hz (with 6% damping). This dropped to less than 2 Hz (an upper bound based on the initial elastic stiffness) and frequencies as low as 1.08 Hz (22% equivalent viscous damping) were observed due to non-linear seismic response. This latter result was obtained with a deformation of 38 mm or around 33% of the maximum bearing displacement. Even lower frequencies (and higher damping) can be expected for higher deformations that would occur under large seismic loading conditions. For the Northbound bridge the first transverse frequency dropped from a value of 5.8 Hz (6% damping) for the steel bearings to less than 1.8 Hz (9% damping) for the neoprene elastomeric bearings. Also, new rubber expansion joints were installed a few weeks before testing the rehabilitated bridges. Due to frictional restraint these joints provide a noticeable contribution to the overall transverse stiffness of the rehabilitated bridges.

Analytical Predictions and Comparisons: Summarized in figure 2, the analytical portion of this study has shown that when seated on the original steel bearings, the bridges were capable of sustaining earthquakes with peak ground accelerations of at least 0.65 g. For reasons other than seismic retrofitting, if steel bearings are rehabilitated using traditional elastomeric/neoprene bearings and also pinning one abutment to provide some anchorage for thermal expansion, then a torsional imbalance may be expected. Predictions show a seismic resistance of PGA = 0.6 g for this class of rehabilitation of the Northbound bridge. This indicates that the bearing rehabilitation of the Northbound bridge did not improve the seismic resistance of that bridge beyond the expectations of the original steel bearings. On the other hand, if the steel bearings are replaced with a well conceived and designed seismic isolation system the seismic resistance may be improved; predictive results for the Southbound bridge show an improvement in seismic capacity to PGA = 0.9 g for lead-rubber isolation bearings. These results show that current seismic evaluation techniques, recommended in the recently published FHWA Seismic Retrofitting Manual (Buckle and Friedland, 1995), underestimate the real resistance capacity of slab-on-girder bridges seated on steel bearings. This is largely because simplistic elastic analysis models, when used, ignore the beneficial effects of friction in steel bearings. Also, current retrofit techniques generally recommend conventional retrofitting or the use of protective systems. This study has demonstrated that existing bridges designed using the provisions of the 60's and 70's can sustain the actual design earthquakes forces. Therefore, by performing a detailed engineering analysis of the structure, in many cases, unnecessary retrofitting may be avoided.

For bridges located in low to moderate seismic zones such as those in the eastern and central United States the use of lead rubber bearings may not be necessary. Partial isolation can be achieved by replacing steel bearings with regular laminated elastomeric bearings. If extra damping or a torsionally imbalanced condition exists, then the behavior of these less expensive bearings can be augmented by using shock absorbing devices.

The use of the proposed methodology to evaluate the capacity/demand (C/D) ratio based on an inelastic response spectrum approach provides a rational basis for studying systems where the overall structural behavior is governed by different types of hysteretic response. The results obtained using this methodology show good agreement with the results obtained from rigorous inelastic time history analysis.

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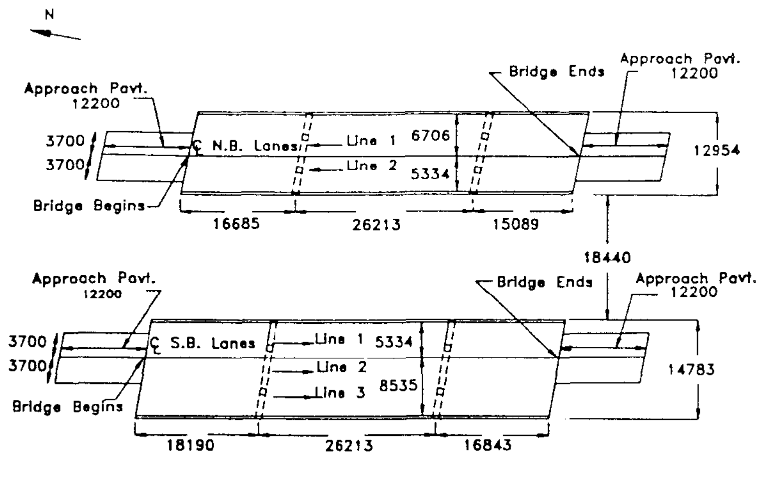
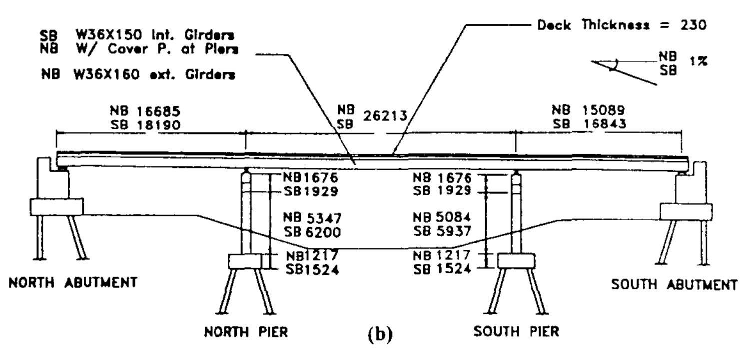


(a)

	BEARING TYPE		
	PRE-RETROFIT	POST-RETROFIT	
	S. BOUND & N. BOUND	S. BOUND	N. BOUND
N. ABUT.	LOW STL. EXP.	LEAD CORE ELAST.	STD. ELAST. (With Pin)
PIER #1	HIGH STL. FIXED	STD. ELAST.	STD. ELAST.
PIER #2	HIGH STL. EXP.	STD. ELAST.	STD. ELAST.
S. ABUT.	LOW STL. EXP.	LEAD CORE ELAST.	STD. ELAST.



(b)

FIGURE 1 Plan and Longitudinal View of the Bridges Over the Cazenovia Creek

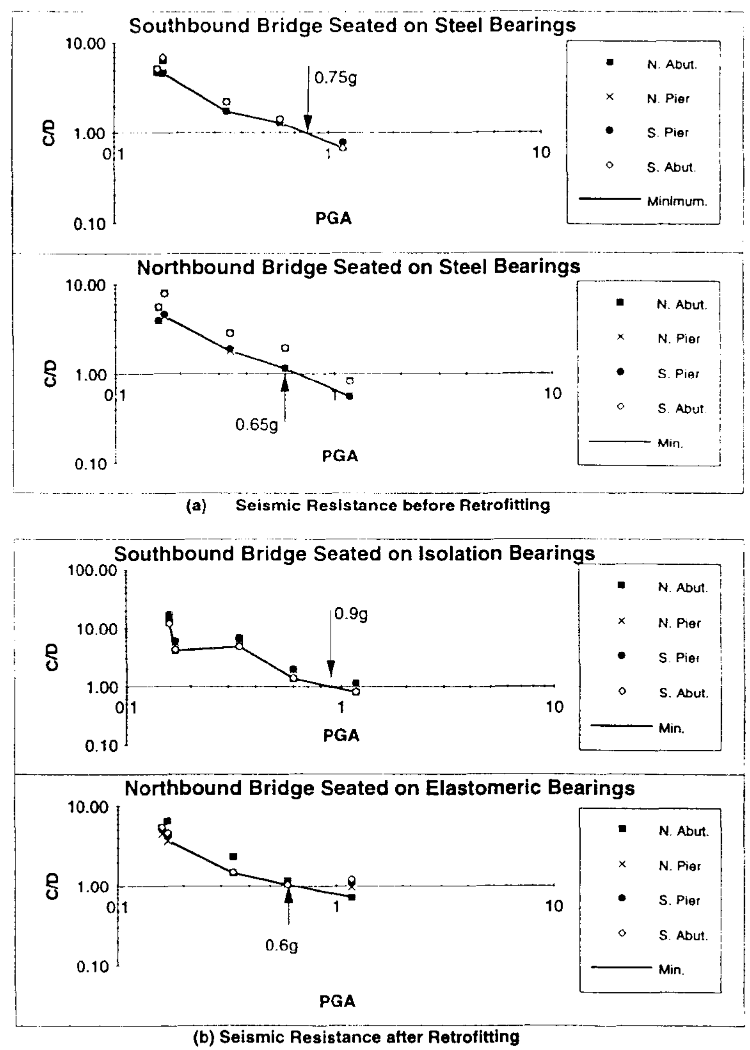


FIGURE 2 Comparison of C/D ratio With PGA Before and After Retrofitting

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Seismic Design of Bridges Using Spring-Viscous Damper Isolation System

Author(s) and Affiliation(s):

Azadeh Parvin
Department of Civil Engineering
The University of Toledo, Toldeo, OH

Principal Investigator: Azadeh Parvin

Sponsor(s): National Science Foundation

Research Start Date: May 15, 1995
Expected Completion Date: April 30, 1997

Research Objectives:

The objective of this research is to perform a feasibility study of novel bridge bearings subjected to lateral forces. It involves experimentation with steel spring bearing supporting highway overpasses that are susceptible to collapse resulting from horizontal and vertical movements caused by winds and earthquakes. The potential for using helical springs in combination with viscous damper systems will be studied. Most bridges in service today were not built strong enough to sustain earthquake loads, and consequently they may not be completely or partially functional after an earthquake. An isolation system will increase the fundamental period of vibration, and can reduce the earthquake forces on the bridge.

Expected Products or Deliverables:

The use of this type of bearing can help to reduce maintenance costs and to economically extend the service life of bridges. This will minimize post earthquake repair and could restore infrastructure to use very quickly. The conclusions drawn from data obtained during this research may eventually lead to modification of specifications, if necessary, and transfer of knowledge through scholarly media which are lacking for steel spring isolation systems in this country.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Azadeh Parvin received her Ph.D. in structural engineering from The George Washington University in 1992. She served as a research advisor for SEAS Laboratory at the George Washington University. She worked as a project engineer for DMI Engineering, Inc., and Engineering Consulting Service, Ltd., before joining the Civil Engineering Department of the University of Toledo in January 1994 as an Assistant Professor. Her current research interests include static and dynamic analysis and design of structures such as reinforced concrete. She is currently working on research related to precast and reinforced concrete beam-column connections using the finite element method. She is also involved in laboratory testing of spring and elastomeric bridge bearings. Dr. Parvin's research activities have been funded by the National Science Foundation, Ohio Infrastructure Institute and Ohio Space Grant Consortium. She is a member of the Society of Women Engineers, ASCE, ACI and The Masonry Society.

Principal Investigator's Telephone Number:	(419) 530-8134
Facsimile Number:	(419) 530-8116
E-mail Address:	aparvin@top.eng.utoledo.edu

SEISMIC DESIGN OF BRIDGES USING SPRING-VISCOUS DAMPER ISOLATION SYSTEM

Azadeh Parvin

Department of Civil Engineering
The University of Toledo
Toledo, OH 43606

Research Objectives

The objective of this research is the feasibility study of novel bridge bearings subjected to lateral forces. It involves experimentation with steel spring bearings supporting highway overpasses that are susceptible to collapse resulting from horizontal and vertical movements caused by winds and earthquakes. The potential for using helical springs in combination with viscous damper systems will be studied. Most of bridges in service today were built not strong enough to sustain earthquake load, and consequently they may not be completely or partially functional after an earthquake event. An isolation system will increase the fundamental period of vibration, and as a result it can reduce the earthquake forces on the bridge.

Research Approach

Modern high-load, multidirectional bridge bearings fall into three categories: pot bearings, spherical bearings and unconfined, pad or disc bearings.[1] The type of bearing is chosen on the basis of three parameters: bearing capacity, or maximum vertical load, maximum rotation and in case of expansion bearings, maximum expansion and contraction. Laminated elastomer elements can provide very low horizontal natural frequencies, and permit horizontal response deflections of up to 500mm.[2] The described types of bearings are very stiff in vertical direction and thus, provide no vibration isolation in this direction for typical earthquakes. In some cases, solely horizontal base isolation may provide sufficient protection but in others a three-dimension system might be required. Such three dimensional systems can be introduced using helical springs. One of the advantages of elastic springs over other types of elements is that horizontal deflection of the spring gives the restoring forces necessary to get the structure back into its original position.

Vertical motion in bridges are not as crucial as in buildings due to relative lower amount of gravity and live loads and less P-Δ of columns and buckling effect. However, vertical motion in bridges should not be ignored completely, since large uplift displacements cause loss of contact followed by impact which will lead to higher mode response and large axial forces in columns.[3] In fact, the Northridge earthquake record for vertical component was 0.6g.

In addition, traditionally, the approach to securing bridge superstructures to piers and abutments has been through various "tie down mechanisms," which to various degrees, successfully maintained integrity between the bridge and its foundation structure. Although motion is accommodated through elongation of a tie down system, no provision is made to protect the interface between the superstructure and its supports at either end. In the absence of such a protection damage has occurred to both bridge girders and bridge abutments.

We will explore the seismic buffer system (coil-spring) that would not interfere with the existing tie-down technology but would provide needed additional compliance between bridge superstructure and its supports. If successful, this concept would mitigate damage to bridge and its supports thereby enabling the bridge to be put back in service quickly following a seismic event.

The spring viscous damper isolation system considered in this study is located at the interface between a continuous superstructure and its supporting piers and abutment to secure both the bridge and the abutment. The seismic loads are reduced by increasing the overall flexibility and damping of superstructure supports. These springs are capable of withstanding horizontal and vertical motions and therefore, will take care of translations as well as rotations. Initially, we consider a rigid body model of an isolated bridge shown in Figure 1. For this model, seismic response of the bridge is taken into account. The model bridge undergoes coupled lateral-vertical response when subjected to seismic excitation. The structure is idealized as a rigid block. The helical springs have vertical stiffness K_{zi} and horizontal stiffness K_{xi}. The mathematical model for viscous dampers has been described by Makris and Constantinou.[4,5] A simplified analysis using the single degree of freedom model is employed to avoid the complexity of solving equations of motion for the coupled system. Stiffness and damping coefficients are evaluated at free vibration frequencies. Finite element analysis is employed to study the response of a multi degree of freedom model of an isolated bridge using the results obtained from the simplified model. Through finite element analysis, the response of a bridge with spring-viscous damper isolation system is compared to the response of a bridge with spring isolation system without damper.

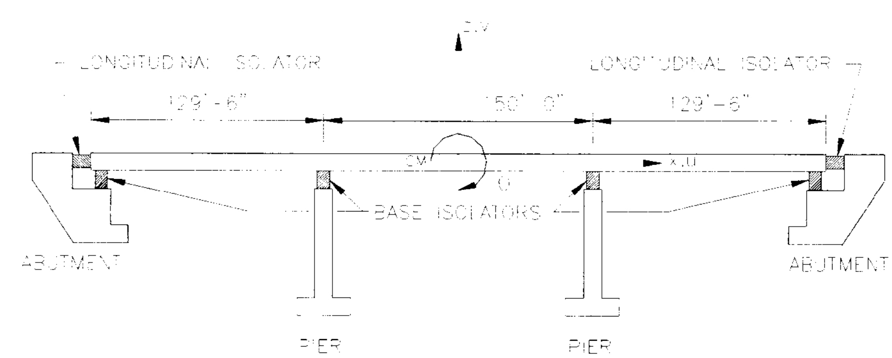


Figure 1. Rigid Body Model of Isolated Bridge

For the laboratory testing, springs with various capacities will be utilized. Different sizes and combinations (one spring can fit inside another to have more capacity to control the displacement) of springs will be tested in laboratory to provide adequate stiffness for a scale-down bridge model under seismic excitation. Initially, a compression test will be conducted for the spring followed by a shear test. Then, a combination of compression and shear testing will be performed. Finally, a spring-viscous damper isolation system under different load combinations will be tested. In the experiments, both longitudinal and base isolation systems will be used. The location of these isolators are shown in Figure 1.

Expected Products or Deliverables

The use of this type of bearing can help to reduce maintenance costs and to economically extend the service life of bridges. This will minimize post earthquake repair and could restore infrastructure to use very quickly. The conclusions drawn from data obtained during this research may eventually lead to modification of the code, if necessary, and transfer of knowledge through scholarly media which are lacking for the steel spring isolation systems in this country.

Preliminary Results

None.

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FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Longevity and Reliability of Sliding Isolation Systems

Author(s) and Affiliation(s):

Michael C. Constantinou, Panos Tsopelas and Amarnath Kasalanati
Department of Civil Engineering
State University of New York at Buffalo, Buffalo, NY

Principal Investigator: Michael C. Constantinou

Sponsor(s): Federal Highway Administration and National Center for Earthquake Engineering Research

Research Start Date: October 1, 1992
Expected Completion Date: June 30, 1996

Research Objectives:

The objectives of this study are to collect data and generate new experimental data on sliding bearings with emphasis on factors affecting their long-term service life in relation to applications of bridge seismic isolation, and to establish bounds on the frictional properties of sliding bearings for use in the analysis and design of seismically isolated bridges.

Expected Products or Deliverables:

New experimental data on sliding bearings which will, in general, be of value to designers, and a recommended procedure for establishing minimum and maximum values of the coefficient of friction for use in design.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Michael C. Constantinou is Professor of Civil Engineering at the State University of New York at Buffalo. He has performed extensive research in seismic isolation and energy dissipation systems for buildings, bridges and equipment. He is a member of TS12 for preparation of the 1994 and 1997 NEHRP Provisions, the New Technologies Team of the ATC-33 Project for preparation of guidelines for seismic rehabilitation, and the task force under AASHTO Bridge Committee T-3 for the upgrade of guide specifications for seismic isolation design.

Panos Tsopelas is a research asssociate in the Department of Civil Engineering at the State University of New York at Buffalo. He has conducted experimental work on bridge seismic isolation systems and has been a major contributor in the development of the 3D-BASIS class of computer programs for the analysis of base-isolated structures.

Amarnath Kasalanati is a Ph.D. candidate in the Department of Civil Engineering at the State University of New York at Buffalo. His doctoral work focuses on testing of bridge seismic isolation systems with emphasis on elastomeric systems which have been detailed for optimum performance in near fault, high velocity seismic shocks.

Principal Investigator's Telephone Number:	(716) 645-2114 ext 2404
Facsimile Number:	(716) 645-3733
E-mail Address:	cieconst@ubvms.cc.buffalo.edu

LONGEVITY AND RELIABILITY OF SLIDING ISOLATION SYSTEMS

Michael C. Constantinou, Panos Tsopelas and Amarnath Kasalanati
Department of Civil Engineering, State University of New York,
Buffalo, NY 14260

Research Objectives

The objectives of this research project are: (a) To collect data and generate new experimental data on sliding bearings with emphasis on factors affecting their long-term service life in relation to applications of bridge seismic isolation, and (b) to develop a procedure for establishing bounds on the frictional properties of sliding bearings for use in the analysis and design of seismically isolated bridges.

Research Approach

Background: The design of seismic isolation bearings is currently based on data obtained from testing of new specimens. Often the condition of these bearings after years of service is not considered in the design. At best, this condition is considered based on assumptions and testing, which often reflect undue conservatism or optimism. Generally, the long-term performance of seismic isolation bearings is not well understood. Moreover, certain information of significance in the design of seismic isolation systems could be rather easily generated but has not been done so far. For example, the dynamic mechanical properties of isolation bearings after significant cumulative movement has occurred or at low temperatures have not been determined. Both are of significance in the design of bridge seismic isolation systems.

This lack of knowledge represents a concern in the design of seismic isolation systems and it is an impediment in the application of the technology. The collection and evaluation of data and the generation of new data on the long-term properties and service life of seismic isolation bearings represents a requirement for the successful application of isolation systems. This project concentrates on sliding isolation systems with the purpose of collecting available data and generating new data on the long-term and other properties of interest for the design of these systems, and in developing a procedure for establishing bounds on the frictional properties of sliding bearings for use in analysis and design.

Research Tasks: The most important element of sliding seismic isolation systems is the sliding interface. In the systems that found applications, this interface consists of polished stainless steel in contact with unfilled and filled PTFE, without or with lubrication (as in flat sliding bearings), or in contact with high bearing capacity composites which contain PTFE as a solid lubricant (as in FPS bearings). The coefficient of friction of these interfaces represents the most important property of sliding bearings. It is known that the coefficient of friction depends on the materials comprising the interface, bearing pressure, velocity of sliding and temperature. Moreover, this coefficient is affected by contamination, dwell of load, aging of the bearing (e.g. corrosion) and history of movement.

This project concentrated on interfaces consisting of unfilled PTFE and high bearing capacity composites since they are in use in the United States. The research tasks were as follows:

(1) To conduct a review of research reports, journals and industrial publications relating to sliding bearings, summarize the mechanical and physical properties of the aforementioned interfaces and collect data on sliding bearings related to factors affecting long-term service life.

(2) To generate data on the effect of dwell of load on the breakaway (or static) coefficient of friction.

(3) To generate data on the effect of temperature (range -40°C to 50°C) on the breakaway and sliding coefficients of friction at high velocities of sliding.

(4) To generate data on the effect of corrosion of stainless steel by conducting tests with the stainless steel part having a range of values of surface roughness.

(5) To generate data on the dynamic friction coefficient following significant slow movement.

(6) To employ deterministic and/or probabilistic methods in establishing bounds on the frictional properties of sliding bearings and propose a procedure for establishing minimum and maximum values of coefficients of friction for use in design based on

(a) Prototype test values, (b) Variability of properties, (c) Effect of temperature, (d) Effect of history of loading (travel), and (e) Aging (includes contamination, corrosion etc).

Expected Products

Expected products include:

(1) New experimental data on sliding bearings as described in the section on research tasks, which will, in general, be of value to designers.
(2) A recommended procedure for establishing minimum and maximum values of coefficient of friction for use in design.

Preliminary Results

The experimental part of the research has been completed. A sample of experimental results is presented herein for unfilled PTFE sliding bearings without lubrication. Figure 1 presents data on the coefficient of friction for temperature in the range of -40° to 50°C. Note that the coefficient was measured at initiation of motion (breakaway) and at four different velocities following travel of 5 to 10 mm. Figure 2 shows the influence of travel on the coefficient of friction. These measurements of friction were obtained at intervals during a repetitive test, in which velocity of sliding was 0.8 mm/s for 250 m total travel and then 2.4 mm/s for another 250 m travel. A total of about 20000 cycles were performed intermittently over a period of three weeks. Note the decrease of friction coefficient and stabilization at a lower value and then a tendency for increase at total travel beyond about 500 m.

Figure 3 shows the variation of the coefficient of friction with velocity for three different stainless steel roughness (ranging from mirror finish to a rough surface-presumably the equivalent of a surface covered with small rust spots). Note the significant effect of roughness on the low velocity friction and the minor effect of roughness on the high velocity friction (the one mobilized in seismic motion).

The project is currently at the stage of finalizing a method for establishing maximum and minimum values of friction coefficient for use in design. This effort is carried out in conjunction with work done for the T-3 seismic design task group of AASHTO, which is in the process of developing a revision of the AASHTO Guide Specifications for Seismic Isolation Design.

The method, which would apply to all types of isolation hardware, establishes maximum and minimum nominal values of a property, $P_{max,N}$ and $P_{min,N}$ respectively, based on a value of property P (e.g. effective stiffness, energy dissipated per cycle or coefficient of friction) established by testing of a new specimen at normal temperature (~20°C) and at the relevant conditions of pressure, amplitude of motion, and frequency or velocity of motion. The nominal values are given by

$$P_{\mathrm{max},N} = \lambda_{\mathrm{max}} \cdot \mathrm{P} \quad , \quad \mathrm{P}_{\mathrm{min,N}} = \lambda_{\mathrm{min}} \mathrm{P} \qquad \textbf{(1)}$$

where λ_{max} and λ_{min} are factors (respectively larger and less than unity) to account for effects of temperature, variability of properties and aging.

The λ_{max} and λ_{min} factors consist of the product of several λ factors, each one of which accounts for a specific effect. For example,

$$\lambda_{\mathrm{max}} = \lambda_t \cdot \lambda_v \cdot \lambda_a \qquad \textbf{(2)}$$

where λ_t is the factor for temperature effect and so on. The T-3 group is currently in the process of establishing the λ factors based on available experimental results, engineering judgement and considerations of likelihood of simultaneous occurrence of several extreme events (i.e. lowest temperature, design earthquake, maximum variability etc.) On the latter, the prevailing opinion within the T-3 group is to use combination factors that reflect the importance of the bridge.

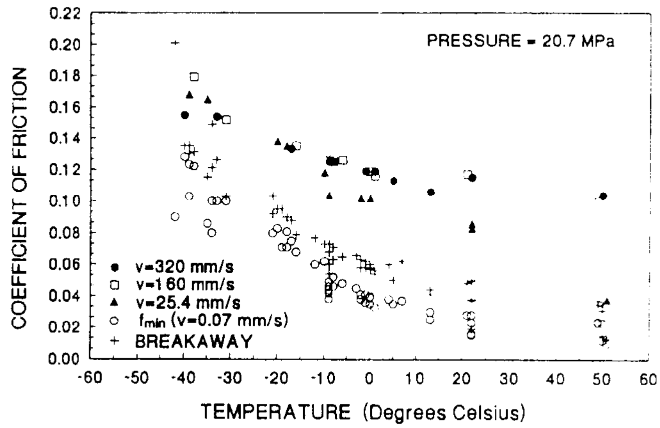


Fig. 1 Effect of Temperature on Coefficient of Friction of Unfilled PTFE

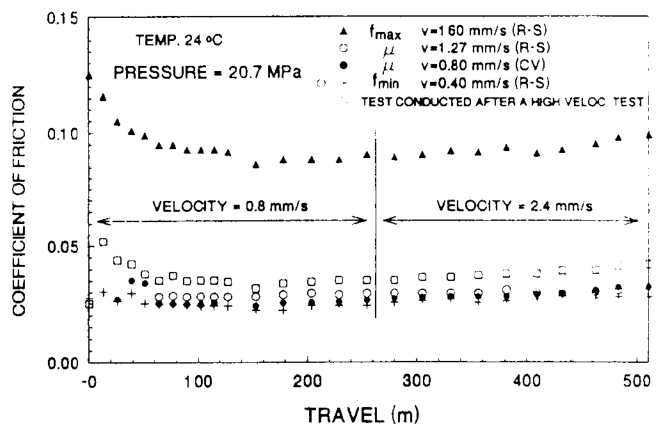


Fig.2 Effect of Travel on Coefficient of Friction of Unfilled PTFE

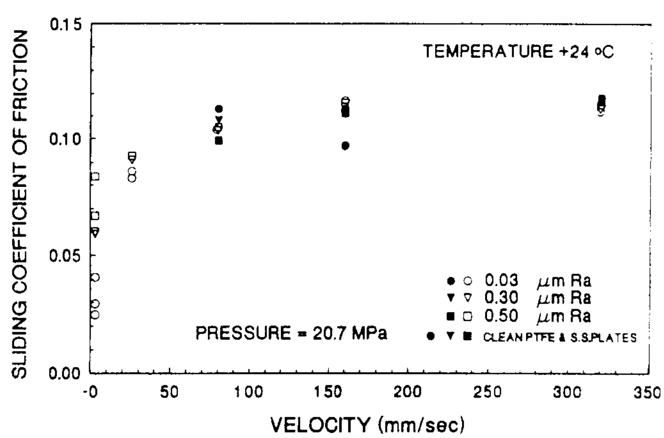


Fig. 3 Effect of Surface Roughness on Coefficient of Friction of Unfilled PTFE

Composite and Other Materials

SESSION VI

[redacted]

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Feasibility of GFRP/CFRP Prestressed Concrete "Demonstration" Bridge in the USA

Author(s) and Affiliation(s):

Nabil F. Grace, Civil Engineering Department
Lawrence Technological University, Southfield, MI
George Abdel-Sayed, Civil and Environmental Engineering
University of Windsor, Windsor, Ontario, Canada

Principal Investigator: Nabil F. Grace

Sponsor(s): National Science Foundation, Reichhold Chemical Company, and Marshall Composites Industry

Research Start Date: September 1994
Expected Completion Date: September 1997

Research Objectives:

The objective of this research is to develop a new concept for a corrosion-free and cost-effective highway bridge system that requires no shoring or construction forms and offers superior advantages compared to conventional bridge systems.

Expected Products or Deliverables:

The City of Southfield, Michigan, in conjunction with Lawrence Technological University and HRC Consulting Engineers, is currently preparing design drawings for the "demonstration" three-span bridge. The bridge will be constructed in 1997 over the Rouge River in Michigan.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Nabil F. Grace is Professor and Chairman of the Civil Engineering Department at Lawrence Technological University. Dr. Grace has published over forty papers in refereed journals and national and international conferences, and has done consulting for GM, Ford, Chrysler and ANR Gas Pipeline. His main research interest is in composite construction and soil-structure interaction.

George Abdel-Sayed is Professor of Civil Engineering at the University of Windsor, Windsor, Ontario, Canada. Dr. Abdel-Sayed is a member of ACI and is currently a board member of the Canadian Network on Advanced Composite Material in Bridges and Structures.

Principal Investigator's Telephone Number:	(810) 204-2556
Facsimile Number:	(810) 204-2509
E-mail Address:	nabil@ltu.edu

FEASIBILITY OF GFRP/CFRP PRESTRESSED CONCRETE "DEMONSTRATION" BRIDGE IN THE USA

Nabil F. Grace[1], and George Abdel-Sayed[2]

[1]Civil Engineering Department, Lawrence Technological University, Southfield, Michigan, 48075-1058.
[2]Civil and Environmental Engineering Department, University of Windsor, Windsor, Ontario, Canada.

Research Objectives

The objective of this research project is to develop a new concept of a corrosion-free and cost-effective highway bridge system that requires no shoring or construction forms and offers superior advantages compared to conventional bridge systems.

Research Approach

The developed bridge system is built of:

(i) Pre-cast modified Double-T (DT) girders, prestressed with internal carbon fiber reinforced plastic (CFRP) rods, or carbon fiber composite cables (CFCC).
(ii) Post-tensioned tendon deviators and cross beams.
(iii) Cast-in-place glass fiber reinforced plastic (GFRP) or CFRP reinforced deck connected to the DT girders through CFRP/GFRP shear connectors (stirrups), and Sikadur 32 epoxy.
(iv) Externally post-tensioning draped CFRP/CFCC strands.

The standard DT is modified with tendon deviators for externally draped strands and to accommodate the CFRP strands for prestressing the DT girders in the transverse direction. The tendon deviators and cross-beams are used to tie adjacent DT modified girders and to provide stiffness for the bridge in its transverse direction. The deviators are monolithically cast at the one-third points of the span.

Construction Concept: The construction concept of the introduced bridge system consists of four phases. In phase one, the DT girders are precast and internally prestressed in the factory. These precast/prestressed DT girders eliminate the need for extensive on-site formwork which certainly help to reduce construction time and cost. The prestressing forces in the internal strands are designed to support the dead weight of the bridge, the deck slab that is added later at the site and percentage of the live loads. The longitudinal internal prestressing forces place the entire cross-section of the DT girders in compression, making them safer to transport, and place the girders in a state of upward camber at the middle of the span. In phase two, the DT girders are transported to the site and placed side-by-side on the supports. The adjacent DT girders are transversely post-tensioned using CFRP strands through the tendon deviators and cross-beams. This links the DT girders transversely without inducing any moments. During phase three, the deck slab is then poured over the entire bridge. It should be pointed out that the CFRP stirrups in the DT webs, cross-beams and tendon deviators are extended from the precast DT girders into the deck slab to ensure the transfer of the horizontal shear forces and eliminate any slippage between the DT girders and the deck slab. Also, Sikadur epoxy is sprayed on the top surface of the DT girders prior to casting the deck slab. The presence of the epoxy and the extended stirrups ensure a full interaction between the deck slab and the DT girders. Finally, in phase four, once the deck slab is in place, the externally draped CFRP strands are post-tensioned. This restores the upward camber and places the bottom of the webs in considerable compression at the middle of the span. This developed stress at mid-span section is designed to counteract the live load stresses so that the bottom of the webs will not experience any tension and will remain in compression, thus eliminating any possibility of developing tension cracks.

Experimental Work: Four bridge models are being constructed, instrumented and tested under static, dynamic, fatigue (7 million cycles) and ultimate loadings. The first two models are rectangular (right angle) while the third and fourth models are skew bridges. CFCC strands (Tokyo Ropes) and GFRP rebars are used in the first bridge model while CFRP Leadline (Mitsubishi Chemical Corporation) rods are used for the construction of the second bridge model. CFRP stirrups are used in the construction of the third and fourth bridge models.

Expected Products

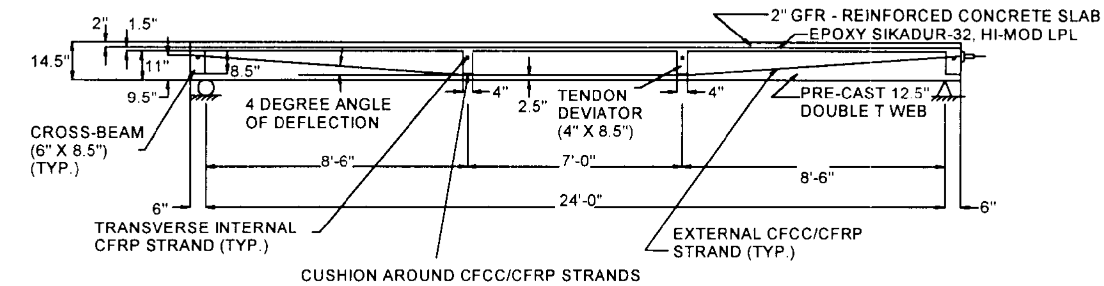
As a result of the ongoing research work, the City of Southfield, in conjunction with LTU and HRC, is currently preparing the design drawings and construction details of the prestressed concrete "demonstration" highway bridge. This bridge will be constructed in 1997 over the Rouge River and will be made of three-spans, each span will be 66 ft. long and 30 ft. wide. It will be designed to support an HS25-44 AASHTO truck. Parallel to the "demonstration" bridge, an identical three-span two-lane bridge will be constructed using conventional materials. Both bridges will be instrumented and monitored for five years. This will facilitate a fair comparison between a conventional and advanced composite materials bridge system under similar loadings and weathering conditions.

Preliminary Results

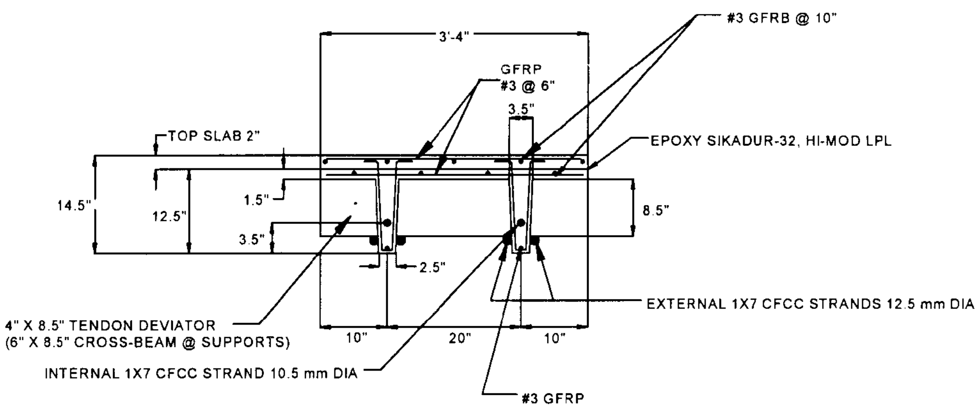
The first and second bridge models are tested under static, dynamic, fatigue and ultimate loads. The ultimate load test results indicate that the tested bridge system can be designed to exhibit similar ductility and crack patterns as the conventional bridge system, see Figs. 2 and 3.

Acknowledgement

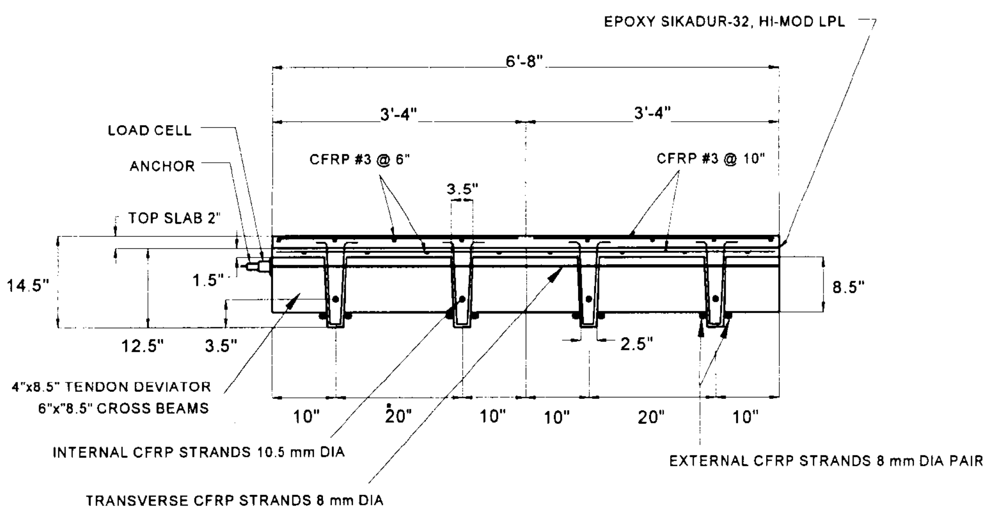
This research project is currently funded by the National Science Foundation under Grant Nos. CMS-9640570, CMS-9540657, and CMS-9401211. The support of Dr. John B. Scalzi, Program Director of Large Building System, is greatly appreciated.



Elevation View



Cross-Section of DT-1 at Midspan



Cross-Section of DT-2 at Midspan

Fig. 1 Details of bridge models DT-1 and DT-2

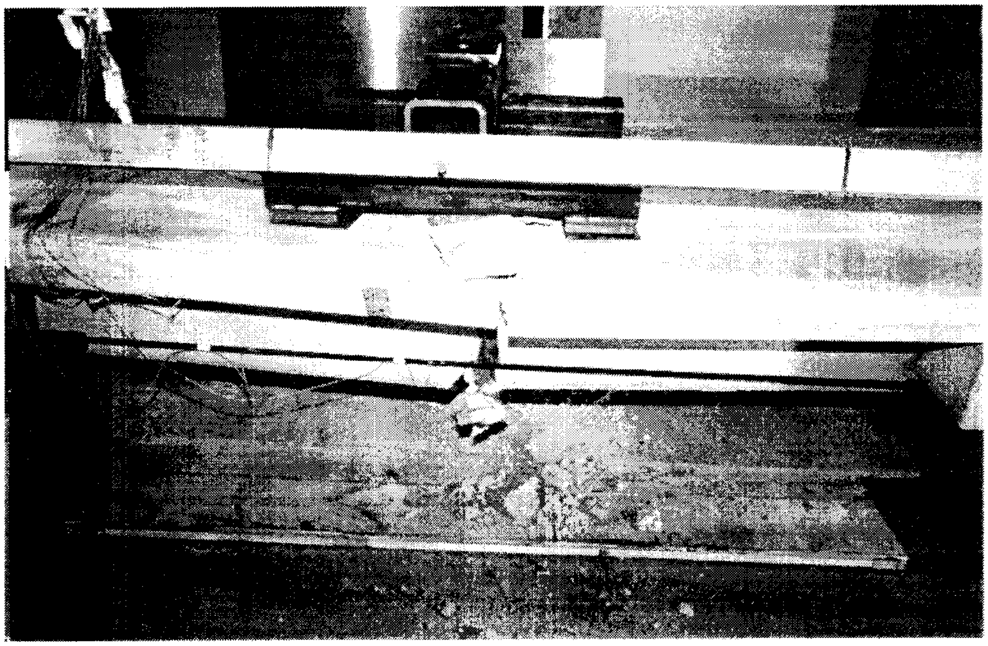


Fig. 2 Failure of bridge model DT-1

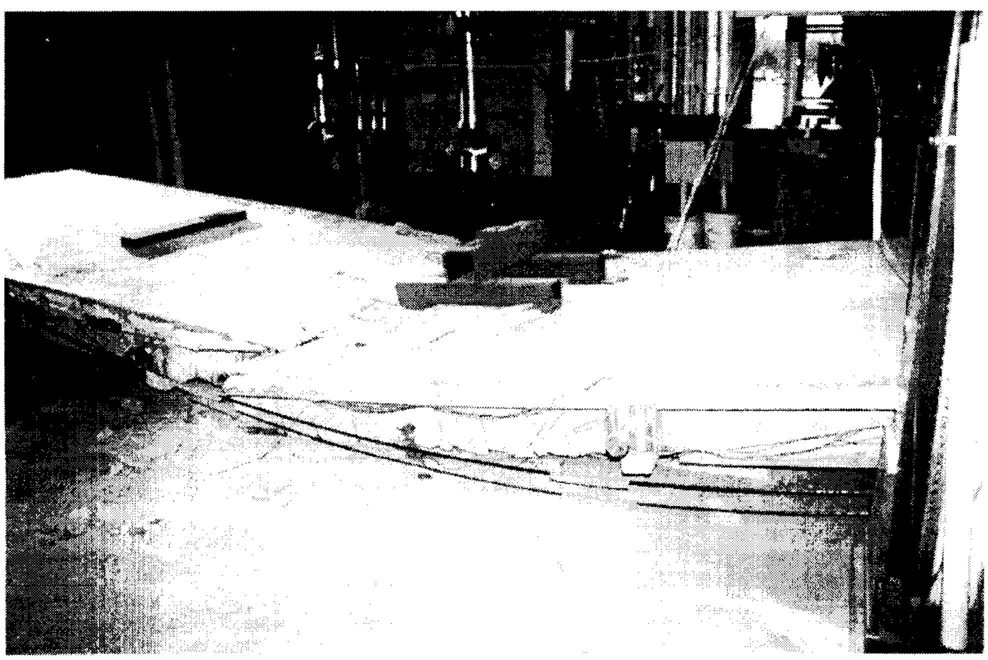


Fig. 3 Failure of bridge model DT-2

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Development of the Carbon Shell System Construction Concept for New Bridge Structures

Author(s) and Affiliation(s):

Frieder Seible, Rigoberto Burgueño, and Andrew Davol
Division of Structural Engineering
University of California, San Diego, La Jolla, CA

Principal Investigator: Frieder Seible

Sponsor(s): Advanced Research Projects Agency and the Federal Highway Administration

Research Start Date: January 1995
Expected Completion Date: June 1996

Research Objectives:

Advanced composite materials have become of great interest for civil engineering design and construction because of their outstanding mechanical and chemical characteristics. Cost-effective use of these new materials can best be achieved by taking advantage of automated manufacturing and their integration with conventional civil-structural materials such as concrete and steel. The research program aims to develop the basis for modular bridge systems made of premanufactured advanced composite carbon tubes filled on-site fully or partially with concrete. Cost-effectiveness of this new system is achieved by the dual function of the carbon shell as stay-in-place formwork and reinforcement, the easy handling characteristics of the lightweight carbon tubes, and the lower expected life-cycle costs. The proposed research is expected to lead to an alternate, cost-effective, structural framing system for new bridge structures.

Expected Products or Deliverables:

It is envisaged that complete bridge systems can be composed of linear segments of carbon shell tubes connected together by means of longitudinal and off-angle joints with appropriate mechanisms depending on the response requirements for the structural component. With the development of appropriate analytical models and proper design details, the concrete-filled carbon shell system can provide a viable design alternative for new bridge structures.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Frieder Seible is Professor of Structural Engineering at the University of California, San Diego. He received his Dipl. Ing. (Civil Engineering) from the University of Stuttgart, West Germany in 1976; his MSCE from the University of Calgary, Alberta, Canada in 1978; and his Ph.D. from the University of California, Berkeley in 1982. His research interests are in analysis and design of reinforced/prestressed concrete bridges, evaluation and rehabilitation of existing bridge structures and buildings, and development of new computer models to predict dynamic and static nonlinear response of reinforced and prestressed concrete structures under service overload and failure loads. Dr. Seible also has experience in verification of computer models by means of large or full-scale experimental testing, earthquake resistant design of reinforced concrete and concrete masonry structures, development of large-scale structural testing techniques, seismic assessment and retrofit of bridges, and the application of Polymer Matrix Composites (PMC) in civil structures. He has more than 170 papers and 90 technical reports mainly related to seismic design of bridges and buildings. Dr. Seible is a registered Professional Engineer in California.

Rigoberto Burgueño is a Ph.D. candidate at the University of California, San Diego. He received his BS and MS in structural engineering in 1993 and 1994 respectively. His research interests are in analysis and design of structural concrete structures, seismic design, assessment and rehabilitation of existing bridges and buildings, computer modeling for dynamic and nonlinear structural response, and implementation of advanced composite materials in civil structures for rehabilitation and development of new systems and components.

Andrew Davol is a graduate research assistant studying structural engineering at the University of California, San Diego.

Principal Investigator's Telephone Number: (619) 534-4640
Facsimile Number: (619) 534-6373
E-mail Address: seible@ucsd.edu

DEVELOPMENT OF THE CARBON SHELL SYSTEM CONSTRUCTION CONCEPT FOR NEW BRIDGE STRUCTURES

Frieder Seible[1], Rigoberto Burgueño[2], and Andrew Davol[2]

[1]Professor of Structural Engineering
[2]Graduate Research Assistant
Division of Structural Engineering, University of California, San Diego
La Jolla, California 92093-0085

Research Objectives

Advanced composite materials have become of great interest for civil engineering design and construction because of their outstanding mechanical and chemical characteristics. Cost-effective employment of these new materials can best be achieved by taking advantage of automated manufacturing and their integration with conventional civil structural materials such as concrete and steel. The research program aims to develop the basis for modular bridge systems made of premanufactured advanced composite carbon tubes filled on-site fully or partially with concrete. Cost-effectiveness of this new system is achieved by the dual function of the carbon shell as stay-in-place formwork and reinforcement, the easy handling characteristics of the lightweight carbon tubes, and the lower expected life cycle costs. The proposed research is expected to lead to an alternate, cost-effective, structural framing system for new bridge structures.

Research Approach

At the University of California, San Diego, an advanced composite Carbon Shell System (CSS) concept for new structural systems is being developed under an ARPA (Advanced Research Projects Agency) TRP (Technology Reinvestment Project) program and a FHWA (Federal Highway Administration) project [1].

The Carbon Shell System emerges from the unique combination of conventional civil construction materials and new polymer matrix composites. A premanufactured advanced composite carbon tube is filled fully or partially with concrete depending on the strength, stiffness, and stability requirements for the structural component. The carbon tubes are manufactured by the wet filament winding procedure, which is a proven, reliable and cost effective process for the construction of advanced composite structures. In this new concept, schematically depicted in Fig. 1, the concrete provides the compression force transfer and the carbon shell the functions of formwork for the concrete, reinforcement for the tension force transfer in bending and shear, and confinement of the concrete core. This system combination takes advantage of the excellent tailorable mechanical characteristics of advanced composite materials and the full utilization of the compressive strength of the well confined concrete core.

The advantages offered by this system can only be fully utilized when appropriate connection systems and concepts can be developed and proven. Due to the linear elastic nature of the carbon composite, the focus is on the development of a design philosophy addressing both ductility and strength connection principles. The research program aims therefore to systematically develop the basis of the CSS concept for new structural components made of filament wound carbon shells and filled on-site with concrete. The research focuses on three areas namely, (1) the analytical modeling and characterization of the concrete filled carbon shell system and the proposed connections, (2) the development of appropriate design models, and (3) the experimental large scale validation of components and assemblages to fully characterize the structural response.

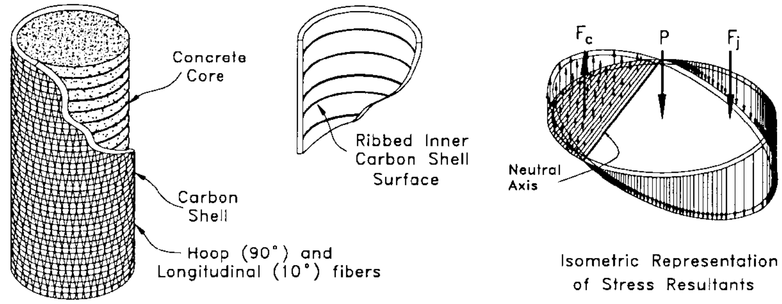


Fig. 1 Schematic of Concrete Filled Carbon Shell System

To establish experimental bench mark data, proposed experimental investigations include beam tests on carbon tubes grouted and ungrouted tested with 3 or 4 point bending test setups which will test for shear and flexural failures based on analytical predictions. The beam characterization tests are expected to be followed by splice/connection tests on similar beam specimens. These connection details are conceptualized to consist of chemically bonded inner sleeve couplers, embedded composite or steel reinforcement bars, longitudinal post-tensioning, or a combination of all of these. Analytically, appropriate models need to be developed which can characterize the interaction between the concrete core and the carbon shell for all possible serviceability and limit states. Simple design models have already been developed with the purpose of allowing rapid experimental investigation and providing the basis for further, more rigorous work. The analytical models will then be calibrated and verified by the test results and used for subsequent parametric studies to form the basis for general design model developments for the carbon shell system framing concept

Expected Products

It is envisaged that complete bridge systems can be composed of linear segments of carbon shell tubes connected together by means of longitudinal and off-angle joints with appropriate mechanisms depending on the response requirements for the structural component. With the development of the previously mentioned analytical tools and proper design details, the concrete filled carbon shell system can provide a viable design alternative for new bridge structures.

The carbon shell system concept is currently being applied as a design alternate towards the demonstration project of a 137m long cable-stayed traffic bridge at Gilman Drive across Interstate 5 in La Jolla, where all components, including deck, superstructure, pylon and cables are proposed to consist of advanced composite materials. The design concept proposes the use of the carbon shell system technology for both the pylon and superstructure systems. Recently, the carbon shell construction concept has also been used for the conceptual design of the Civil Engineering Research Foundation (CERF) 61m span Columbus Indiana Bridge [2,3]. Two design concepts were proposed, both with full clearance of the span without any intermediate piers. The Carbon Shell Space Truss Bridge, see Fig. 2, consists of a three dimensional space truss supporting a precast prestressed concrete deck. The space truss system is composed of a lower chord suspended from the deck system by means of two inclined Warren-type trusses [2]. The Shallow Dual Tied Carbon Shell Arch Bridge, see Fig. 3, consists of free standing twin arches held together by post-tensioned tie beams. The deck system consists of transverse beams connected to the tie beams in a link and log fashion supported from both arches by means of inclined hangers [3].

Although actual construction and manufacturing costs are difficult to assess based on pilot projects, the mass production of carbon shells of just one or two standard diameters can result in manufacturing costs projected by the participating manufacturing partners to $15/lb or less. Life cycle costs of these new materials and system in a civil infrastructure environment are not available at this stage, but are expected to be less than those of conventional bridge systems due to the inert carbon fiber shells.

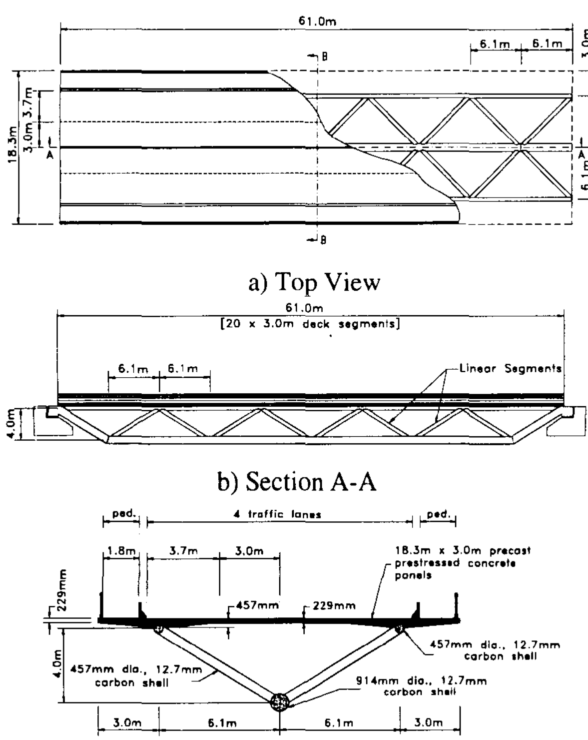


Fig. 2 Carbon Shell Space Truss Bridge

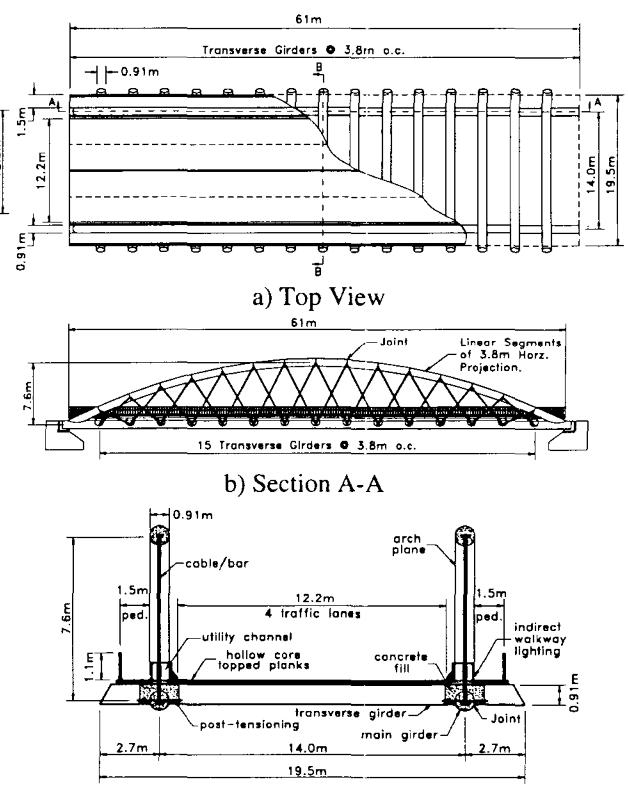


Fig. 3 Shallow Dual Tied Carbon Shell Arch Bridge

Preliminary Results

Fig. 4 Carbon Shell System Ductile Concept Test

Pilot lateral load tests on two concrete filled carbon shell bridge circular columns have demonstrated the great potential of this innovative application for the development of new framing systems [1]. A ductile design concept consisting in simple starter bar connections penetrating from the column footing and incased with a concrete filled carbon shell has resulted in a very ductile overall system, see Fig. 4. The system displayed large inelastic deformation capacities in a well confined plastic hinge matching almost exactly the response of a conventionally reinforced "as-built" concrete column reference test, see Fig. 5. Furthermore, it was verified that only short development lengths of these splice bars are required due to the high confinement from the circular carbon tube. Thus, ductile connection details utilizing mild reinforcement splice bars in the concrete core across the joint can be envisaged.

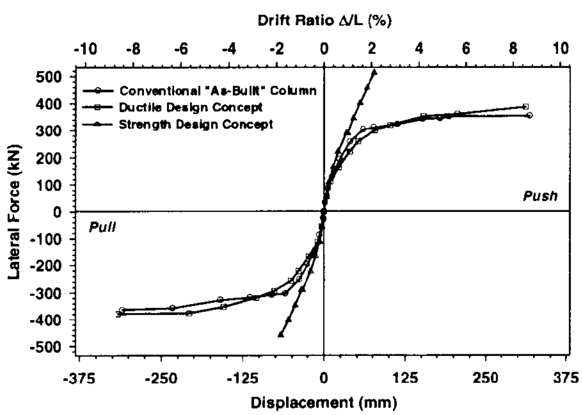


Fig. 5 Force-Displacement Envelope Comparison of Pilot Test Units

A strength approach connection relying on the capacities of the carbon shell system without the use of steel for anchorage was successfully tested displaying a linear elastic response up to failure, see Fig. 5. Connection design concepts in the form of premanufactured threaded and/or bonded inserts for the design of structural members where either simple strength design is used, or where parts of the structure are to be protected from inelastic action can thus be conceived. The testing plan for a third pilot test column with a chemically bonded inner sleeve joint is currently in progress.

References

[1] Seible, F., Burgueño, R., Abdallah, M.G., Nuismer, R., "Advanced Composite Carbon Shell Systems for Bridge Columns under Seismic Loads", *National Seismic Conference on Bridges and Highways*, San Diego, California, December 1995.

[2] Seible, F., Hegemier, G.A., Karbhari, V., Davol, A., and Burgueño, R., "Carbon Shell Space Truss Bridge," CERF 1996 Innovation Awards Program, Innovative Concepts Award Entry Form, University of California, San Diego, La Jolla, California, December 1995.

[3] Seible, F., Hegemier, G.A., Karbhari, V., Burgueño, R., and Davol, A., "Shallow Dual Tied Carbon Shell Arch Bridge," CERF 1996 Innovation Awards Program, Innovative Concepts Award Entry Form, University of California, San Diego, La Jolla, California, December 1995.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS RESEARCH ABSTRACT

Title: Fabrication and Testing of Fiber Reinforced Composite Bridge Decks

Author(s) and Affiliation(s):

Vistasp M. Karbhari, Lei Zhao and Yanqiang Gao
Division of Structural Engineering
University of California, San Diego, La Jolla, CA

Principal Investigator: Vistasp Karbhari

Sponsor(s): Advanced Research Projects Agency and the Federal Highway Administration

Research Start Date: 1995
Expected Completion Date: Ongoing

Research Objectives:

The objective of this study is the development of processing techniques and design approaches for the fabrication and use of replacement and new bridge decks using fiber reinforced composite materials. This will be done through the investigation of overall response to load and investigation of damage/failure mechanisms. Deck sections were designed to model the response of typical reinforced concrete deck panels of 9" depth. Experimental variables include core configuration, fabric architecture node strengthening methods, and processing routes. The design philosophy for the program is summarized in terms of three specific performance criteria: to develop stiffness through the use of skin-core architectures to mimic that in the range shown by cracked and uncracked concrete; to ensure equivalent energy levels at acceptable displacements; and to develop processing methods that are cost-effective and ensure uniformity and repeatability

Expected Products or Deliverables:

Design guidelines for the use of replacement composite bridge decks and optimized materials-process-configuration sets validated by testing, which will be usable for both replacement bridge decks and for new construction.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Vistasp M. Karbhari is Assistant Professor in the Division of Structural Engineering at the University of California, San Diego. His primary research interests are in the application of composites to civil infrastructure, composites manufacturing science, interphase studies, and mechanics of damage. Dr. Karbhari has a BSCE and an MS in structural engineering both from the University of Poona, and a Ph.D. in composites from the University of Delaware. Dr. Karbhari has published over 125 papers in the areas of application of composites to civil infrastructure, composites processing science, mechanics of composites, and durability of materials, and serves on the editorial boards of the Journal of Thermoplastic Composite Materials and the International Journal of Materials and Product Technology. He is the vice-chair of the newly formed ASCE Committee of Emerging Materials, and is a member of the Subcommittee on Advanced Composite Reinforcements of the ASCE Committee on Structural Composites and Plastics. Dr. Karbhari is the winner of the 1996 Charles Pankow Award for Innovation from the Civil Engineering Research Foundation (with F. Seible and G.A. Hegemier), and was awarded the Best Paper Award in Materials at the 1992 ASM/ESD Advanced Composites Conference.

Lei Zhao is a Ph.D. candidate at the University of California, San Diego and has an MS degree from the University of Wyoming.

Yanqiang Gao is an MS candidate at the University of California, San Diego.

Both students are investigating the use of composites for replacement bridge decks.

Principal Investigator's Telephone Number:	(619) 534-6470
Facsimile Number:	(619) 534-6374
E-mail Address:	karbhari@ames.ucsd.edu

FABRICATION AND TESTING OF FIBER REINFORCED COMPOSITE BRIDGE DECKS

Vistasp M. Karbhari, Lei Zhao and Yanqiang Gao

Division of Structural Engineering
University of California, San Diego
La Jolla, CA 92093-0085

Research Objectives

The overall test program is aimed at the development of lightweight, degradation resistant fiber reinforced composite decks for use in replacement and renewal. The program is investigating both design approaches, as well as processing options for the fabrication of cost-effective composite components that would satisfy three main criteria, i.e. (i) the development of component stiffnesses through the appropriate use of face sheets and internal core configurations that would fall in the range between the uncracked and cracked stiffness of existing reinforced concrete decks, (ii) the development of equivalent energy levels at acceptable displacement levels as a means of building in a safety factor due to the elastic behavior of composite, and (iii) the development of processing methods that would ensure repeatability and uniformity.

Research Approach

Background: The deterioration of decks in bridges is a critical issue that urgently needs to be addressed through the introduction of enhanced materials and technologies. Deck deterioration can be traced to reasons ranging from aging and environmentally induced degradation to poor initial construction and lack of maintenance. It is estimated that most bridges in the US on an average last 68 years, whereas their decks on average last 35 years. Some estimates from the mid west and from regions where there is extensive use of road salt, put the life of a conventional bridge deck at about 10 years, thereby requiring extensive triage, or expensive replacement within a short time period. Added to the problems of deterioration, are the issues related to the need for higher load ratings and increased number of lanes to accommodate the ever increasing traffic flow on the major arteries. Beyond the costs and visible consequences associated with continuous retrofit and repair of such structural components, are the real consequences related to losses in productivity and overall economies related to time and resources caused by delays and detours.

The high strength-to-weight and stiffness-to-weight ratios, corrosion and fatigue resistance of fiber reinforced composites, in addition to their tailorability makes them attractive for use in replacement bridge decks or in new bridge systems. Besides the potentially lower overall life-cycle costs (due to decreased maintenance requirements), such decks would be significantly lighter, thereby affecting savings in substructure costs, enabling the use of higher live load levels in the cases of replacement decks, and bringing forth the potential of longer unsupported spans and enhanced seismic resistance. The current research emphasizes the use of fiber reinforced composites for the entire deck or as part of the actual superstructure system itself.

Materials and Manufacturing Methods: The replacement bridge decks considered in this investigation are primarily fabricated using one, or a combination of the following processing methods - Wet layup/sprayup, Pultrusion and Resin Infusion. The primary reinforcement material is E-glass in the fabric form, with the resins being either Vinylesters or Polyesters. A variety of core materials are investigated including non-structural foams (used for manufacturing ease) and end-grained balsa. The use of polymer concrete as a wearing surface for these decks is being investigated concurrently.

Test Matrix and Procedures: In order to gain as much information about materials suitability, geometrical configuration and structural response a building block approach is used which incorporates tests at 4 different sizes as described in the matrix in figure 1. The components at the smallest length level were tested to assess the effect of different geometrical configurations and cell types. Tests at this level were used to determine damage and failure mechanisms, especially at nodal regions, and were also of use in assessing viability of various manufacturing schemes. The larger sections gave structural response in shear and bending respectively. The large scale panels are of size 15' x 7.5' and are fabricated in pairs. One panel from each pair is to be tested in the laboratory, and if successfully tested and found viable in terms of performance criteria, the second panel would be placed in a road-bed for monitoring under actual traffic conditions. The smaller sections were tested in three point bend, whereas the 14' sections were tested using the setup as shown in Figure 2 in order to ensure continuity conditions.

CONFIGURATION	COMPONENT SCALE			
	3' - 4'	6' - 8'	14'	Large Scale Panels
Balsa Core	●			
Foam Filled Boxes	●		●	
Foam Filled Truss	●	●	●	●
Foam Filled Hat Sections			●	●
Pultruded Profiles With Face Sheets	●	●	●	
Hybrids				●
Corrugated Core		●		

Figure 1: Matrix of Test Sections

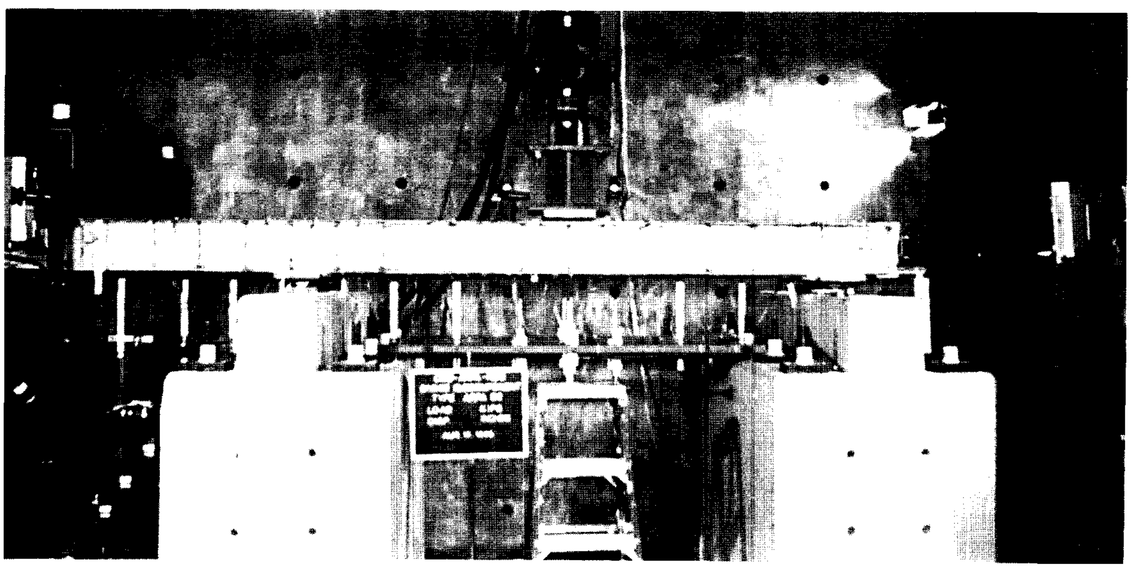


Figure 2: View of the Test Setup and a "Box" Configuration Deck Component

Expected Deliverables

The research is aimed at the development of cost-effective replacement decks fabricated using fiber reinforced composite materials. It is expected that the research will result in the selection of appropriate materials and processes, as well as the development of a design approach and methodology for their use both in areas where replacement of existing decks is necessary, and in new construction as well. The decks will be lighter than those fabricated out of conventional materials (as an example, the weight of a 15' x 7.5' panel of reinforced concrete weighs about 12,500 lbs., whereas the equivalent composite deck weighs less than one-fourth that amount, both sections being of 9 inch depth) and hence would be easier to transport, erect and will give advantages in terms of lower dead load, increased life and lower maintenance requirements. It is expected that the use of these decks will result in overall lower life-cycle costs. The research is also expected to result in the development of design guidelines of use to the civil engineering community. Developments within the current stage will also assist in defining the cost-effectiveness of such an approach, both in terms of acquisition cost and overall life-cycle cost. Although it is expected that initially the acquisition costs would be higher than those for reinforced- or precast-concrete decks, optimization of materials, configuration and processes could result in costs that are in the realm of orthotropic steel.

Preliminary Results

Figure 3 shows the overall load-midpoint displacement profiles achieved through the testing of a number of different specimens using the three-point bend test configuration. It can be seen that significant "pseudo-ductility" can be gained from the composite based on configuration. This is developed through cracking at nodes and between fabric layers. Tests conducted at various load levels below the ultimate show very little change in overall stiffness.

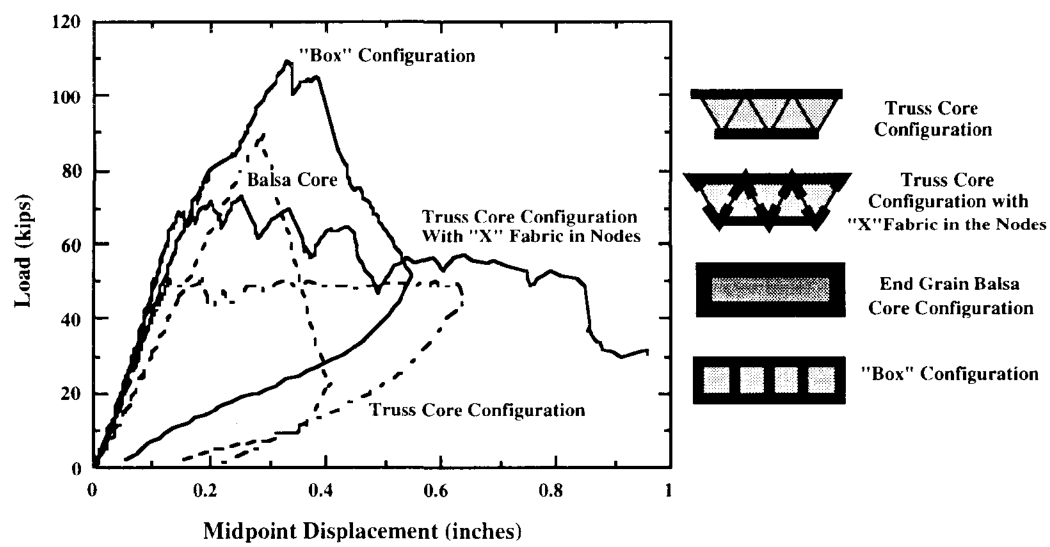


Figure 3: Load-displacement response of subcomponents tested in three point bend

A major concern in the development of large sections using "lower cost" and emerging technologies such as Resin Infusion is related to the ability to fabricate sections repeatably, not only in terms of dimensions, but also with respect to overall performance levels and failure mechanisms. Figure 4 depicts the repeatability of components fabricated using this process at the 6-7' level, tested in flexure. One of the three components was fabricated with a wearing surface of polymer concrete, which was not seen to increase structural stiffness significantly. Overall the repeatability and uniformity between sections was found to be good.

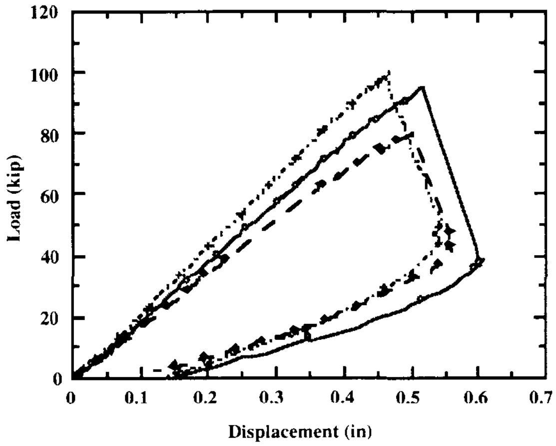


Figure 4: Repeatability of Performance (Triangular Truss Configuration)

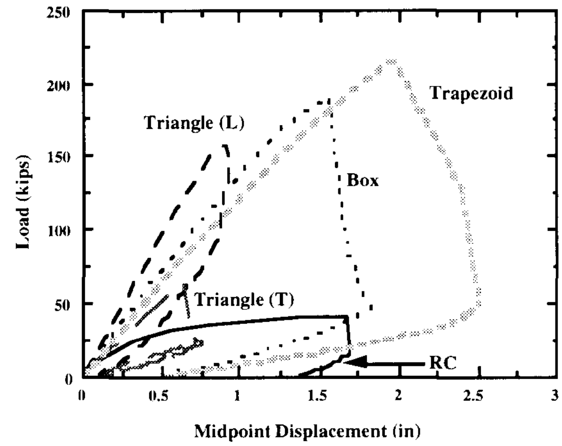


Figure 5: Comparison of overall response at the 14' level tested using the configuration shown in Figure 2 (L = Longitudinal, T = Transverse)

As can be seen from Figure 5 and Table 1, the fiber reinforced composite deck components tested using a setup as shown in Figure 2, show failure loads far in excess of that shown by the reinforced concrete specimen with initial structural stiffnesses that are comparable. The "box" configuration and the "trapezoid" configuration also show significantly enhanced energy levels. Obviously further optimization is necessary in order to achieve the appropriate performance levels at the minimum cost. Costs are directly related to the thickness of composite used in the skins and in the webs formed with the core configuration.

Configuration	Maximum Load (kips)	Displacement at Maximum Load (in)	Energy (kip-in)
Reinforced Concrete	40.52	1.68	53.99
Triangle (L)	158.2	0.91	75.81
Triangle (T)	63.95	0.65	21.24
Box	189.73	1.57	165.13
Trapezoid	215.44	1.97	230.45

Table 1: Comparison of Overall Response of Deck Components

Acknowledgments

The financial support of the ARPA and the FHWA through the ARPA/TRP Agreement No. MDA 972-94-3-0030 is gratefully acknowledged.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: A New FRP-Encased Bridge Pier Column

Author(s) and Affiliation(s):

Mohsen Shahawy, Structural Research Center
Florida Department of Transportation, Tallahassee, FL
Amir Mirmiran, Department of Civil and Environmental Engineering
University of Central Florida, Orlando, FL

Principal Investigator: Amir Mirmiran

Sponsor(s): Florida Department of Transportation

Research Start Date: September 1994
Expected Completion Date: December 1997

Research Objectives:

The primary objective of this research is to utilize full section enclosure and confinement of concrete by fiber reinforced plastic (FRP) tubes with or without internal reinforcement to enhance load carrying capacity and durability of bridge pier columns. Fiber-wrapping of existing concrete columns has proved to be an effective retrofitting measure for seismic protection (i.e., enhanced strength and ductility) as well as environmental protection (i.e., against damages due to corrosion or freeze-thaw). This research attempts to apply the same concept to the construction of new structural columns. In order to take advantage of the high strength, low weight, and excellent corrosion resistance of fiber composites, a new concrete-filled FRP tube is proposed in which the tube is the pour form, protective jacket, confining mechanism, and shear and axial reinforcement.

Expected Products or Deliverables:

The end-product in this research is a new concrete-filled FRP tube with or without internal reinforcement. Observations from experimental studies as well as results of analytical work will help determine the feasibility, cost-effectiveness, and range of applicability of the proposed system for actual design conditions. A design program with an optimization module will accompany the design guide and recommendations.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Mohsen Shahawy is the director of the Structural Research Center of the Florida Department of Transportation. He is also Adjunct Professor of Structural Engineering at the Joint College of Engineering, Florida State University/Florida Agricultural and Mechanical University. He received his Ph.D., MS, and BS degrees from Queen's University, University of Manitoba, and Cairo University, respectively. Dr. Shahawy is actively engaged in full-scale testing of bridges and structural elements, mainly for concrete structures and the application of advanced composites. He is a member of the Transportation Research Board's Committee A2CO5 on Dynamic Vibration of Bridges, and Committee A2CO7 on Structural Fiber Reinforced Plastics. He is also an active member of ASCE, ACI, and PCI. He has over 130 publications in various areas of structural engineering including proof load testing of bridges, use of FRP for prestressed concrete piles, bridge repair with fiber composites, and moment redistribution of continuous prestressed concrete bridges. His work on composites has been cited by researchers around the world. Dr. Shahawy is a registered Professional Engineer.

Amir Mirmiran is Assistant Professor of Structural Engineering at the Department of Civil and Environmental Engineering of the University of Central Florida. He received his MS and Ph.D. degrees from the University of Maryland, and his BS degree from Tehran University. He is a Professional Engineer in Maryland, Pennsylvania, and Florida with over eight years of design and construction experience. Dr. Mirmiran is the chair of the ASCE Special Structures Committee and a voting member of ACI-ASCE Committee 343, Concrete Bridge Design. He is also director of the ASCE Structural Technical Group, East Central Florida Branch. He has over 40 publications on composites and concrete.

Principal Investigator's Telephone Number:	(407) 823-3592
Facsimile Number:	(407) 823-3315
E-mail Address:	ami@engr.ucf.edu

A NEW FRP-ENCASED BRIDGE PIER COLUMN

Mohsen Shahawy[1] and Amir Mirmiran[2]

[1] Structural Research Center, Florida DOT, Tallahassee, FL 32310
[2]Department of Civil and Environmental Engineering
University of Central Florida, Orlando, FL 32816

Research Objectives

The primary objective of this research is to utilize full section enclosure and confinement of concrete by fiber reinforced plastic (FRP) tubes with or without internal reinforcement to enhance load carrying capacity and durability of bridge pier columns. Fiber-wrapping of existing concrete columns has proved to be an effective retrofitting measure for seismic protection (i.e., enhanced strength and ductility) as well as environmental protection (i.e., against damages due to corrosion or freeze and thaw). This research attempts to apply the same concept to the construction of new structural columns. In order to take advantage of high strength, low weight, and excellent corrosion resistance of fiber composites, a new concrete-filled FRP tube is proposed in which the tube is the pour form, protective jacket, confining mechanism, and shear and axial reinforcement.

Research Approach

Background:

Lateral confinement of concrete in compression members can significantly enhance their strength and ductility. Traditionally, steel is used for confinement of concrete columns as transverse ties or tubes in new construction and steel jackets for retrofitting. However, fiber composites can offer higher strength-to-weight ratio and excellent resistance to corrosion. Various fiber-wrapping mechanisms have been developed including fiberglass straps (University of Arizona), carbon fabric sheets (Florida DOT), prefabricated FRP half-cylinders (Hexcel Fyfe), or FRP cables (Penn. State University). This research intends to utilize fiber composites in new construction of bridge pier columns and piles (Figure 1). Durability, strength, ductility, and life-time costs are the main driving forces behind this research. Within the stated objectives, following tasks are identified:

1. Evaluate the effect of confinement with fiber composites on the stress-strain response of concrete. Effects of various parameters such as types of fibers and resin, fiber winding angle, tube thickness or number of plies, and diameter of the concrete core are considered.

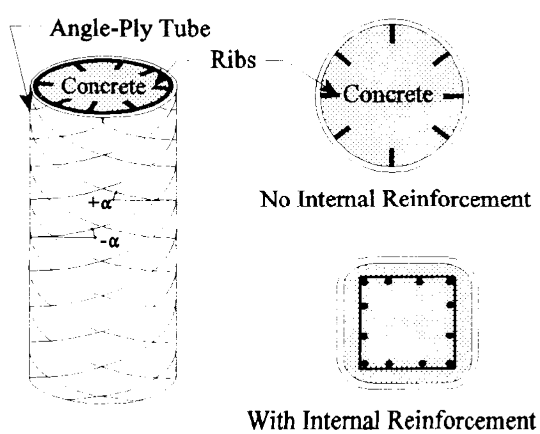


Figure 1. FRP-Encased Concrete Column

2. Evaluate the effect of slenderness (L/D) ratio on the load-carrying capacity of the FRP-encased concrete columns, where L and D are the length and diameter of the column, respectively.

3. Determine the contribution of the FRP tube to shear resistance of the composite column. Also, investigate various shear transfer mechanisms in the form of ribs or studs.

4. Study the beam-column behavior of FRP-encased concrete with eccentric or transverse loads.

5. Develop a systematic design approach to optimize the core diameter as well as thickness and winding angle of the FRP tube for any load combination.

In order to accomplish these tasks, a combination of analytical and experimental techniques are adopted that are discussed in the following sections.

Experimental Studies:

1. Material characterization tests are conducted to establish the hoop strength and stiffness of FRP tubes. Apparent tensile strength and modulus of elasticity of the tubes are measured according to the ASTM standard D 2290-92 (split-disk method) with slight modifications.

2. Uniaxial compression tests on FRP-encased concrete cylinders of standard size with various concrete strengths and different tube thicknesses. Also, effect of loading and unloading cycles on the stiffness degradation of composite cylinders is investigated.

3. Uniaxial compression tests on tall and slender FRP-encased concrete columns with various slenderness ratios.

4. Four-point flexure tests on short concrete-filled square FRP beams are conducted to evaluate the contribution of the casing to the shear resistance, and the performance of shear transfer mechanism.

5. Push-out tests to determine the bond strength in pure shear. Test parameters include interior texture, surface conditions and preparations, age and strength of concrete, and type of shear transfer mechanism.

6. Beam-column tests on FRP-encased columns to develop the experimental interaction diagrams for various levels of eccentricity with or without transverse shear forces.

Analytical Studies:
The experimental studies are augmented with detailed analytical studies as follows:

1. Classical laminate theory is used to generalize the material characterization of FRP tubes for any winding angle and number of laminae.

2. Experimental results of uniaxial compression tests are compared with the available confinement models for steel-encased concrete to investigate the accuracy and applicability of such models for FRP-encased concrete.

3. Sectional analysis and design is chosen as a simple and reliable means for assessing the performance of a prismatic FRP-encased column with various cross-sectional arrangements and with or without internal reinforcement. The model is capable of analyzing a composite column with any type of confining jacket, i.e., steel or fiber composites. It also estimates the shear, axial, and flexural strength and ductility of a single composite column for any combination of loadings. In this model, the section is divided into concrete core, exterior shell, and internal reinforcement (if any). The core and the tube are further discretized into strips or layers to facilitate the integration over the cross-sectional area. For cyclic loading, a cumulative damage model proposed by Park and Ang will be adopted. The sectional results are then integrated over the length of the column to generate the corresponding load-deflection curves for static or cyclic loadings.

4. Nonlinear 3D FEA : Available FE software (ANSYS) is utilized to study the 3-D response of the column under various loading conditions. The results are compared with those of the sectional analysis to improve the understanding of the behavior of composite columns, and to modify the sectional model.

5. Parametric study and design guides: Since clearly, not all material, geometric and loading parameters can be considered in the experimental program, a comprehensive parametric study is conducted to evaluate the effect of these parameters on the beam-column behavior of concrete-filled FRP columns. The parameters of interest are fiber type, jacket thickness, fiber orientation, concrete strength, cross-sectional shape and dimensions, presence of internal reinforcement, bond strength and shear transfer mechanism between the materials, and the column length and slenderness. Based on parametric studies and the results of sectional analysis, design guides in the form of design charts and aids will be developed.

Expected Products or Deliverables

The end-product in this research is a new concrete-filled FRP tube with or without internal reinforcement. Observations from experimental studies as well as results of analytical work will help determine the feasibility, cost-effectiveness, and range of applicability of the proposed system for actual design conditions. A design program with an optimization module will accompany the design guide and recommendations.

Preliminary Results

For the last 15 months, some of the aforementioned tasks have been performed. The experiments included split-disk tests for material properties, uniaxial compression tests, slenderness tests, and tests for shear strength. The analytical studies are focused on modeling of FRP-encased concrete, optimization of the thickness and angle of various plies in the tube, interaction diagrams for beam-columns, and design of composite columns for static loads. Here, a brief review of uniaxial compression tests are presented. For more details, references are indicated at the end of this section.

A total of 42 6"x12" concrete-filled glass fiber reinforced plastic (GFRP) specimens and 12 controls have been tested under uniaxial compression. Also, a total of 72 6"x12" cylinders were wrapped with uni-directional carbon fabric sheets. The FRP tubes were made by a filament-winding process, and consisted of E-glass fibers and polyester resins. The carbon fabric is about 0.02" thick, and is composed of AMOCO THORNEL yarns in a unidirectional weave. Figure 2 shows the characteristic cyclic response for a 14-layer GFRP specimen. The experiments generally indicate good energy-absorption characteristics for composite columns. Results confirmed that the initial bi-linear stress-strain path of FRP-encased concrete may serve as an envelope for the stress-strain curves under quasi-static loading and unloading cycles. More importantly, while the loops become wider beyond the peak strength of unconfined concrete, stiffness degradation is not as severe as that of steel-encased concrete.

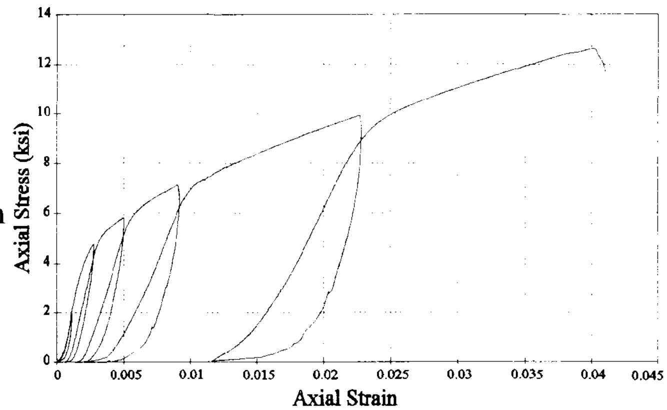


Figure 2. Cyclic uniaxial response of FRP-encased concrete

Acknowledgements:

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2. Mirmiran, A., Kargahi, M., and Shahawy, M. (1995). "FRP-encased concrete columns." *Proc. 2nd Int. Conf. Composites Engrg.*, New Orleans, LA., 509-10.

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4. Mirmiran, A., and Shahawy, M. (1996). "A new concrete-filled hollow FRP composite column." *Composites: Part B,* special issue on *Infrastructure,* in press.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Composite Rebar with Enhanced Ductility and Sensing Capability

Author(s) and Affiliation(s):

Abdeldjelil Belarbi, Department of Civil Engineering
K. Chandrashekhara, Department of Mechanical/Aerospace Engeering
Steve E. Watkins, Department of Electrical Engineering
University of Missouri, Rolla, MO

Principal Investigator: Abdeldjelil Belarbi

Sponsor(s): Mid-America Transportation Center and the Missouri Highway and Transportation Department

Research Start Date: September 1, 1995
Expected Completion Date: August 31, 1996

Research Objectives:

Limited service life as well as high maintenance and repair costs are associated with corrosion, fatigue, and other degradation in bridge and highway structures. Corrosion of steel reinforcing bars (rebar) and prestressing tendons in alkaline and saline conditions is a major source of concrete deterioration and can lead to complete structural collapse. A solution to the corrosion problem is replacing conventional steel rebar with advanced fiber-reinforced-plastic (FRP) composite rebar which consist of reinforcing fibers embedded in a binding matrix material. These composite materials have been proposed and tested in a variety of civil engineering applications. For reinforced concrete (RC) structures, the margin for safety at ultimate states is based on ductility of the rebar, while FRP composite materials exhibit highly brittle behavior. The primary objective of this research is to apply recent advances in composite material technology to develop a new type of composite rebar with the required strength and ductility.

Expected Products or Deliverables:

Development of a new type of FRP rebar, with mechanical properties comparable to that of conventional steel rebar but more corrosion-resistant. The proposed FRP rebar has two important features, ductility and smart sensing, and would serve as an ideal replacement for steel rebar in structures threatened by corrosion such as highway bridges, marine structures, wastewater plants, and other structures in which health monitoring is desirable.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Abdeldjelil Belarbi is Assistant Professor of Civil Engineering and Research Investigator with the Intelligent Systems Center as well as the Graduate Center for Materials Research. He holds an M.S. and Ph.D. from the University of Houston. His primary research interests are the constitutive modeling of reinforced and prestressed concrete, smart structures and smart sensing applied to civil infrastructure, FRP composites, and structural performance of buildings from earthquakes and wind. He has more than 30 publications in these research areas and is an active member on several technical committees of the American Concrete Institute. He was the recipient of a 1996 McDonnell Douglas Faculty Excellence Award as well as several teaching awards.

K. Chandrashekhara is Associate Professor of Mechanical and Aerospace Engineering and Engineering Mechanics and Research Investigator with the Intelligent Systems Center at the University of Missouri-Rolla. He received his Ph.D. degree from Virginia Polytechnic and State University, Blacksburg in 1985. His research interests are finite element analysis, composite materials and smart structures. He is an Associate Fellow of the American Institute of Aeronautics and Astronautics, and is a recipient of the 1995 Phillips Petroleum Faculty Excellence Award.

Steve E. Watkins is Assistant Professor of Electrical Engineering and Research Investigator with the Intelligent Systems Center at the University of Missouri-Rolla. He received his Ph.D. from the University of Texas at Austin in 1989. His research interests include fiber optic sensors and laser metrology. He was a finalist for the 1994 Outstanding Young Electrical Engineer Award.

Principal Investigator's Telephone Number:	(573) 341-4478
Facsimile Number:	(573) 341-4729
E-mail Address:	belarbi@novell.civil.umr.edu

FRP REBAR WITH ENHANCED DUCTILITY AND SENSING CAPABILITY

A. Belarbi[1], K. Chandrashekhara[2] and S.E. Watkins[3]

[1]Department of Civil Engineering
[2]Department of Mechanical and Aerospace Engineering and Engineering Mechanics
[3]Department of Electrical Engineering
University of Missouri-Rolla, Rolla, MO 65409

Research Objectives

Limited service life as well as high maintenance and repair costs are associated with corrosion, fatigue, and other degradation in bridge and highway structures. Corrosion of steel reinforcing bars (rebar) and prestressing tendons in alkaline and saline conditions is a major source of concrete deterioration and can lead to complete structural collapse. A solution to the corrosion problem is replacing conventional steel rebar with advanced fiber-reinforced-plastic (FRP) composite rebar which consist of reinforcing fibers embedded in a binding matrix material. These composite materials have been proposed and tested in a variety of civil engineering applications. For reinforced concrete (RC) structures, the margin of safety at ultimate state is based on ductility of the rebar, FRP composite materials exhibit highly brittle behavior. The primary objective of this research is to apply recent advances in composite materials technology to develop a new type of composite rebar with the required strength and ductility. It will employ a novel hybrid fiber make-up for enhanced ductility. In addition, these composites offer *smart* capability in which embedded fiber optic sensors provide health monitoring and micro-damage assessment of rebar and RC structures.

Research Approach

Background: RC structures suffer from corrosion of embedded steel rebar by expansion, cracking, and eventually spalling of the concrete cover as well as loss of bond between steel and concrete which ultimately results in structural damage. Efforts to improve civil infrastructures include the use of high-performance composite materials which feature properties such as corrosion resistance, electromagnetic transparency, and high strength-to-weight ratio. Some recent civil engineering applications using these materials are rehabilitation and strengthening of existing concrete structures as well as fabrication of some composite structural elements such as pipes, light-poles and wide-flange beams. The use of FRP as reinforcement for concrete has also seen rapid growth in recent years but, in general, it has been limited to secondary structural RC elements. Some primary structural elements of course have been recently designed with FRP on a trial basis. Current RC design provisions were developed for steel rebar and cannot be applied to FRP rebar for two reasons: unknown, probably insufficient, bond resistance and lack of ductility. Both properties are fundamental requirements in RC structural design. To standardize the use of FRP rebar in RC design, one of two approaches needs to be undertaken: (1) to develop FRP rebar with bond resistance and ductility comparable to that of steel or (2) to develop design provisions based on new definitions of safety criteria and reliability parameters. This research follows the first approach and focuses on the ductility issue.

Ductility Requirement: Ductility is a measure of safety in RC structural design. To prevent brittle failure, RC members should be capable of undergoing large defections at near-maximum load-carrying capacity. This may save lives by giving warning of impending failure and preventing total collapse. Furthermore, ductility is a requirement allowing a structure to absorb energy under seismic and reversed loads and to redistribute internal forces through formation of plastic hinges. Redistribution of internal forces in RC depends on the ductility of a member at the critical section, which in turn depends on the ductility of the material constituting the member. Since concrete is brittle in nature, the ductility of an RC structural member can only be achieved by ductility of reinforcement through appropriate proportions of the two materials. Typical stress-strain relationships for steel rebar exhibit a linear elastic portion up to yield, followed by a plastic portion until fracture. *Yield stress*, is the limit of the first linear elastic portion. *Ductility of reinforcement* is the ability of the rebar to deform beyond the yield stress. It is essential for structural safety that steel be ductile enough to undergo large deformations before fracture. Thus minimum strain in the reinforcement at fracture, as defined in steel specifications, ranges from at least 4.5 to 12 percent.

Fabrication of FRP Rebar: FRP composites consist of continuous reinforcing fibers (glass, aramid, carbon, etc.) embedded in resin matrix (thermoset or thermoplastic). The matrix binds the fibers together, transfers loads to them, and protects them. Mechanical properties can be controlled through (1) selection of fiber and resin, (2) relative volume proportion of fiber to matrix, (3) fiber orientation, and (4) fabrication method. Stress-strain behavior of composites reflects all the properties of the constituents and is intermediate to that of the fiber alone, Fig. 1(a), and the matrix alone, Fig. 1(b). Therefore, the current composite materials behave more or less as a linear elastic material up to rupture, and do not exhibit any ductility.

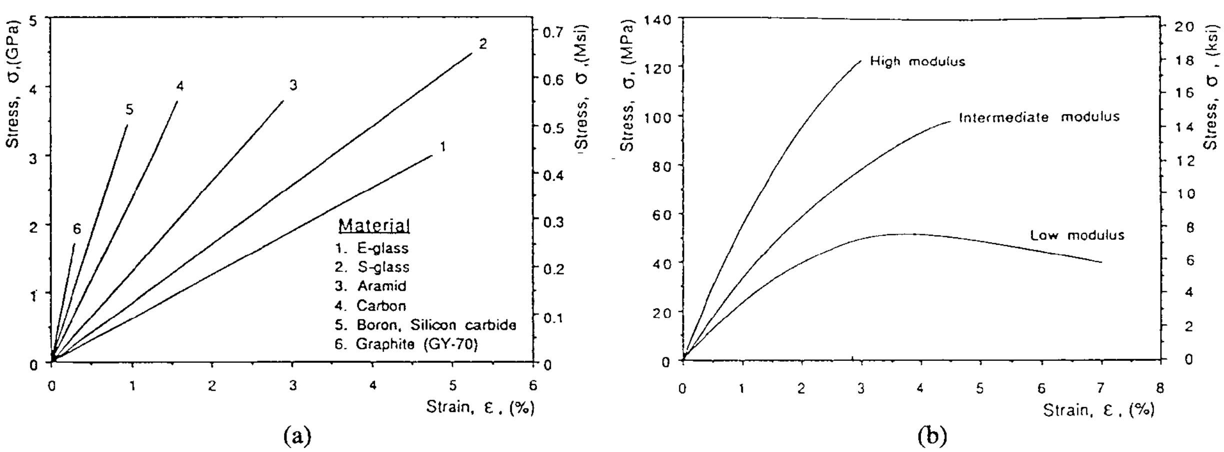

Fig. 1 Stress-strain curves of (a) reinforcing fibers and (b) epoxy matrix resin

In order to induce a pseudo-ductility in the FRP rebar we propose to use a hybrid composite design in which several types of fiber are used simultaneously in the composite, as shown in Fig. 2. This consist of fibers with varying stiffness and failure strain. The elastic region and the overall stress-strain curve will be determined by the combination of multiple fibers and thermoplastic matrix material. Beyond the yield stress point, fibers with the lowest rupture strain break first. Sequential failure of the other fibers in order of increasing rupture strain produces the desired *reinforcement pseudo-ductility* for the overall rebar while maintaining the strength. In addition to the longitudinal lay-out of the fibers, some low stiffness fibers will be helically wound around the rebar to serve two purposes: (1) to form protrusions for bond and (2) to increase the overall longitudinal stretching of helical structures under loading. Thus rebar ductility will be enhanced. Resin selection will affect the mechanical characteristics as well as the and corrosion resistance. Resin and hardener are mixed and applied to fibers using a pultrusion technique. Rebar is planned to be produced in several standard diameters.

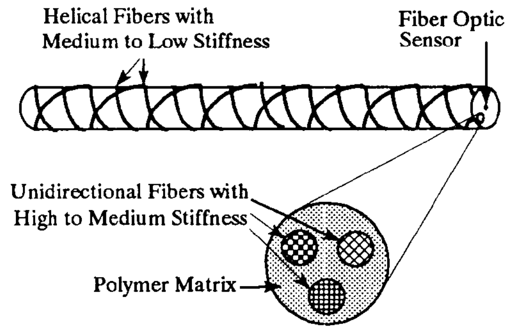

Fig. 2 Smart hybrid FRP rebar

<u>Incorporation of Fiber Optic Sensors</u>: Fiber optic sensors are ideal for smart structures applications because they are rugged, small in size, resistant to corrosion and fatigue, immune to electrical interference, and easily multiplexed. Embedded sensors have been used to measure strain in many materials including concrete and composites. They are a proven cost-effective method for monitoring the health of bridges. Extrinsic Fabry-Perot sensors (one type of fiber optic strain gage) have high sensitivity, low physical profile, and good compatibility with composites. They will be embedded along the axis of the composite rebar during fabrication and addressed by a connecting optical fiber. These sensors consist of a small interferometric cavity formed between the ends of two optical fibers inserted into a hollow-core fiber. Light intensity from the cavity is a nonlinear function of cavity length and hence cavity strain. Light from a laser diode will travel down the connecting optical fiber to the cavity. The strain-modulated signal will return over the same fiber to a photodetector for demodulation. All instrumentation will be located remotely from the smart FRP rebar.

<u>Evaluation of Stress-Strain Relationship of FRP Rebar</u>: Three coupons of each rebar size and each composite combination/type will be tested under uniaxial tension at the loading rate specified by ASTM standards. Monotonic and repeated stress-strain relationships will be determined. Rebar will be instrumented with embedded fiber optic sensors to detect possible internal micro-damage. This test will determine what composite combination (fibers and matrix) exhibits structural properties (strength, stiffness and ductility) similar to those of steel rebar. Repeated stress-strain relationships will determine the effect of loading and unloading on composite stress behavior. Further studies will include the evaluation of bond between concrete and FRP rebar and flexure tests on beams made of the developed FRP rebar.

Expected Products

A new type of FRP rebar, with mechanical properties comparable to that of conventional steel rebar but corrosion-resistant, is expected. The proposed FRP rebar has two important features, ductility and smart sensing. They would serve as ideal replacement for steel rebar in structures threatened by corrosion such as highway bridges, bridge decks, marine structures, wastewater plants, and other structures in which health monitoring is desirable. The increased service lifetime and in-service sensing capability would provide great savings.

Preliminary Results

The results of the preliminary study indicated that it is possible to incorporate pseudo-ductile behavior in FRP hybrid composite rebars. This is achieved by appropriate selection and physical placement of different carbon fibers in the resin matrix. An analytical study was carried out on five different types of fiber and one type of resin. The fibers included in this study had elastic modulii raging from 220 to 690 MPa and failure strains ranging from 0.0031 to 0.0155 while the resin had a low elastic modulus of 2.2 MPa and a failure stain of 0.081. Volume fraction, V_f, was about 43% fibers and 57% resin. The stiffness of the composite can be characterized by the rule of mixture theory. It is assumed that a perfect bond exists between the fibers and the matrix, so that no slippage can occur at the interface and the strains experienced by the fibers, matrix, and composite are equal.

From initial observations, one can conclude that the yield and ultimate strains of the composite rebar are dictated by the lowest strain (0.0031) and the highest strain (0.0155) of the types of fiber used. The composite elastic modulus and the shape of the post-yield region depend on modulus and the corresponding volume fraction of the fibers. The composite yield strain of 0.0031 is close to that of steel bars, however, the ultimate strain of 0.0155 is much less than that of steel bars (0.070 minimum). The physical placement of some fibers can further enhance the ductility. Some fibers with low elastic modulus and high ultimate strain were helically wound as shown in Fig. 1. The pitch varied for different fibers and was chosen to be inversely proportional to the fiber elastic modulus. Additional deformation (Fig. 3) was provided by stretching the helical structures under loading. Furthermore, based on the rule of mixture theory, all the fibers of one type are assumed to break at one given strain which is referred to as the *mean* failure strain. This approximation results in abrupt changes in the stress-strain curves at failure strains of different types of fiber. In reality the actual failure strains of individual fibers of one type scatter around the mean value according to Weibul's distribution. Taking this into account, the curves become smoother as shown in Fig. 3. To conclude, it has been shown analytically that the ductility can be incorporated in FRP rebar. The next phase of the project will focus on the laboratory fabrication of these hybrid FRP systems and comparison of their behavior with respect to the analytical predictions.

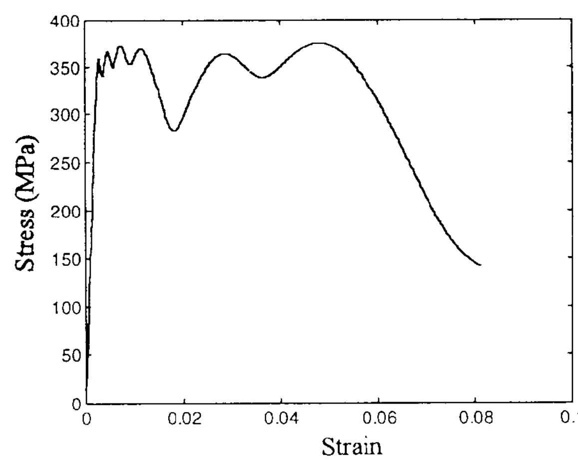

Fig. 3 Stress-strain curve of hybrid FRP rebar

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS RESEARCH ABSTRACT

Title: Advanced Composite Stay Cables

Author(s) and Affiliation(s):

Markus Wernli and Frieder Seible
Department of Structural Engineering
University of California, San Diego, La Jolla, CA

Principal Investigator: Frieder Seible

Sponsor(s): Advanced Research Projects Agency

Research Start Date: September 1994
Expected Completion Date: 1997

Research Objectives:

Stay cables are the main structural elements of cable stayed bridges. Unlike many other bridge types, these components are completely exposed to environmental conditions. It is thus of general interest to make such cables as corrosion resistant as possible. Advanced composites such as carbon or aramid fiber reinforced plastics offer properties like high specific strength and stiffness, and high fatigue and corrosion resistance. They can be an excellent alternative to conventional steel by extending service life, reducing maintenance and simplifying installation. Furthermore, due to the light weight, high strength and high stiffness, they allow the construction of more efficient long-span bridges.

This research is part of the Advanced Composite Cable Stayed Bridge Project at the University of California, San Diego. The scope of research is to assess the mechanical behavior of composite cables, with a focus on the short- and long-term performance of the anchorages, which are the key components of a cable system.

Expected Products or Deliverables:

The research (1) will show the state of the art of composite cable systems and their performance; (2) will be a source to establish guidelines for the use of composite stay cables; (3) the long term test will be a valuable reference which will increase the confidence on the use of advanced composites in infrastructure; and (4) the program should help to increase the efficiency and performance of cable components with respect to the right choice of material, geometry and anchor type.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Markus Wernli has a diploma in civil engineering (1992) from ETH in Zurich, Switzerland and an MS in structural engineering (1996) from the University of California, San Diego. He worked as a Project Engineer in the development group of VSL International Ltd., Switzerland between 1992 and 1994, and is currently a Ph.D. candidate at the University of California at San Diego.

Frieder Seible is Professor of Structural Engineering at the University of California, San Diego. He received his Dipl. Ing. (Civil Engineering) from the University of Stuttgart, West Germany in 1976; his MSCE from the University of Calgary, Alberta, Canada in 1978; and his Ph.D. from the University of California, Berkeley in 1982. His research interests are in analysis and design of reinforced/prestressed concrete bridges, evaluation and rehabilitation of existing bridge structures and buildings, and development of new computer models to predict dynamic and static nonlinear response of reinforced and prestressed concrete structures under service overload and failure loads. Dr. Seible also has experience in verification of computer models by means of large or full-scale experimental testing, earthquake resistant design of reinforced concrete and concrete masonry structures, development of large-scale structural testing techniques, seismic assessment and retrofit of bridges, and the application of Polymer Matrix Composites (PMC) in civil structures. He has more than 170 papers and 90 technical reports mainly related to seismic design of bridges and buildings. Dr. Seible is a registered Professional Engineer in California.

Principal Investigator's Telephone Number:	(619) 534-4640
Facsimile Number:	(619) 534-6373
E-mail Address:	seible@ucsd.edu

ADVANCED COMPOSITE STAY CABLES

Markus Wernli[1] and Frieder Seible[2]

[1] Graduate Research Assistant
[2] Professor of Structural Engineering
University of California, San Diego, La Jolla, CA 92093-0085

Research Objectives

Stay Cables are the main structural elements of cable stayed bridges. They are completely exposed to environmental conditions such as moisture, wind, ice, sunshine, fire hazards, impact and vandalism. Furthermore, due to the typical cyclic loading, high fatigue performance of the cable system is demanded. These requirements are poorly fulfilled by steel tendons and the bridge system has to be designed so that the tendons can be replaced typically every 20 to 30 years. These maintenance costs are mostly underestimated and usually not even considered during the first cost evaluation. In the past few years, the use of advanced composite materials for stay cables has been investigated. These materials such as carbon (CF) or aramid (AF) fiber reinforced plastics offer properties like high specific strength and stiffness, high fatigue and corrosion resistance. They are an excellent alternative to conventional steel, extending service life, reducing maintenance and simplifying installation. Unidirectional reinforced composite materials have been regularly used in aerospace industry since the sixties and may be competitive with steel tendons if the overall lifetime cost of a bridge is evaluated instead of only the initial construction costs. However, the highly anisotropic character of the material makes the anchorage of such tendons very difficult. Furthermore, only little is known about the long term behavior of such materials under this kind of loading.

The research program described in the following paragraphs is part of the Advanced Composite Stay Cable Bridge Project currently in progress at the University of California in San Diego and is sponsored by the Advanced Research Projects Agency (ARPA) [1].

Research Approach

Market Analysis: The world market was scanned for commercially available cable systems. Most of the systems were found in Japan and Europe. However, only a few companies offer cables with appropriate anchors, which could fulfill the requirements for stay cables. The contacted companies are listed in Table 1.

Two different types of anchorages can be identified. In the first one, the tendons are potted in resin and the load is introduced by bond forces between the resin and the composite tendons. The second type anchors the tendons mechanically by wedges, where the force is introduced by friction. Some systems use a combination of these two types. Tokyo Rope offers a system of twisted CF-strands anchored in either a bond anchor or a friction type anchor. However, for stay cable application they suggest the bond anchor type. Mitsubishi has a system of parallel CF-rods anchored with a mechanical wedge type anchor. Both systems were used for pre- and post-tensioning in Japan and Europe. The BBR system consists of parallel CF-wires. It has extensively been tested and is the first CF-cable replacing two steel stay cables in a new road bridge in Winterthur, Switzerland. Nippon Steel and Dywidag use twisted CF-strands. Both suggest a bond anchorage for their stay cables. However, Nippon steel is developing a friction type anchor. The system of Bridon is composed of a twisted boundle of CF-rods anchored with bond anchors. Amoco's tendon consists of rectangular CF-rods. The anchorage has not been developed yet but will consist of bonded tabs. These rods are used in oil well beam lift pumps because of their excellent chemical resistance. Each of the last four listed cable systems has no reference applications in civil engineering so far. VSL in cooperation with LCL offers a tendon made of a parallel array of dry aramid fibers. Its anchorage is of the friction type. The system has been used for stays of antennas and mooring buoys. Among its first civil applications is a pedestrian stay cable bridge in Aberfeldy, Scotland, completely made of composite materials. Akzo offers an AF-rod system anchored with friction type anchor. It has been used for pre- and post-tensioned bridges in Japan. Teijin has a twisted AF-strand system using a bond type anchorage. Also this system has already been used for prestressed bridges in Japan. Mitsui Corporation and United Ropework offer braided aramid ropes. From all these systems, only six of them were selected for further investigation. The intention was to get a large representation of each cable type for both materials, aramid and carbon reinforced plastics. Due to the required high stiffness and low creep properties for stay cables, the investigation is mainly focused on carbon fiber based systems. However, availability and cooperation of the different suppliers dictated partly the choice of the considered cable systems. The systems of the following suppliers have been selected for further investigation: Tokyo Rope, BBR, Nippon Steel, Amoco, VSL, Teijin.

Table 1. Cable Systems available on the world market.

Company	tendon type	anchorage type
Tokyo Rope, Japan	twisted CF-strands	bond anchor
Mitsubishi Kasei, Japan	CF-rods	friction anchor
BBR, Switzerland	CF-wires	bond anchor
Nippon Steel, Japan	twisted CF-strands	bond anchor
Bridon, UK	twisted boundle of CF-rods	bond anchor
Dywidag, Germany	twisted CF-strands	bond anchor
Amoco, USA	rectangular CF-rod	bond anchor
VSL, Switzerland; LCL, UK	parallel dry aramid fibers	friction anchor
Akzo, Netherlands	AF-rods	bond anchor
Teijin, Japan	twisted AF-strands	bond anchor
Mitsui Corporation, Japan	braided AF ropes	bond anchor
United Ropework, USA	braided AF ropes	friction/bond anchor

Short Term Testing Program: The purpose of the short term tests is to evaluate the stress-strain relationship, ultimate tensile load, failure mode and in particular the stresses and deformations in the anchorages, the initial creep and slip and the residual deformations of the anchor components. Short term tests were performed on three specimens of each of the selected cable systems. The concept of the test was similar to those recommended by the Post-Tensioning Institute (PTI) [2] and the Federation Internationale de la Precontrainte (FIP) [3] for steel tendons. The recommendations were adjusted for composite cables. The testing specimens had a length of 3.66 m from one anchorage to the other and had a nominal capacity of 250 to 300 kN, similar to common 0.6" diameter steel strands. During testing, the ideal testing procedure turned out to be according following pattern: The tendons are loaded to their specified maximum allowable service load $P_{ser,max}$. Then they are held at that load for one hour. After that they are cyclic loaded ten times between $P_{ser,max}$ and 5 kN before they are loaded to failure.

This specified maximum allowable service load $P_{ser,max}$ may be different for individual cable systems and is not clearly defined yet. A reasonable definition of this force could be a lower 2 % fractile of a dynamic and a static load limit. The dynamic load limit could be the force, under which the tendon withstands $2x10^6$ load cycles with $P_{ser,max}$ as upper load and with a stress range of 150 MPa. The static limit could be the load under which no creep-rupture occurs during a lifetime of 100 years. It is clear that in particular the static load limit is difficult to estimate, since there is generally a lack of long term tests. However, for this testing program, $P_{ser,max}$ was chosen to 65 % and 55 % of the nominal breaking load of the tendon for CF and AF-tendons, respectively.

Long Term Testing Program: The purpose of the long term tests is to investigate the relaxation of the tendons and the deformations of the anchorages over several months. Also the handling and the stressing procedure for field application is part of these tests. The same specimen geometry is used for the long term as for the short term testing. From each selected cable system, two specimens are built in separate testing frames. The cables are loaded with a removable jack and stressing chair to their specified maximum service load $P_{ser,max}$ defined above. Then the anchorages are set. To achieve an equal stress level in all the tendons, the cables will be restressed to $P_{ser,max}$ after one month. During several months, the deformations in the anchorages and the cables will be monitored. It is expected that these cables will remain stressed during several years and can serve as reference objects in the future.

Life Prediction: The results from the short term tests in particular the cyclic testing will be compared with the results of the long term tests to assess an appropriate method for predicting the lifetime performance of composite cables. The investigation will be supported by coupon testing, fatigue and accelerated aging testing. The coupon testing will mainly be focused on the relation between the shear, friction, tension and deformation in the anchor components. The conclusions of these tests and an analytical and computational investigation of the deformations in the anchor components should then provide an estimate for the lifetime behavior of such cable systems.

Expected Products

The expected products are manifold. First, it will show the state of the art of composite cable systems and their performance. Second, it will be a source to establish guidelines for the use of composite stay cables, like appropriate maximum allowable service load and permissible fatigue range. Third, the long term test will be a valuable reference object which will increase the confidence in the use of advanced composites in infrastructure. Forth, the program should help to increase the efficiency and the performance of cable components with respect to the right choice of material, geometry and anchor type.

Preliminary Results

The current state of the market for composite stay cables is dominated by Japanese and European products. Even though some of these products are already used in pre- and post-tensioning applications and extensive testing on the cable material has been performed, none of the suppliers can offer a satisfactorily mature stay cable system. The use of stay cables is limited to a few small reference applications.

The short term tests showed very good performance of the carbon fiber material and good performance of the high modulus aramid material. However, some anchorages show high deformations and redistribution of the shear forces during cyclic loading, which indicates slip of the tendon (see Figure 1.). Whether this characterizes a consistent trend in behavior can only be stipulated at this time. The results of the long term tests and the investigation by accelerated aging have to be evaluated first. Definitively more research has to be done in design of anchorage systems.

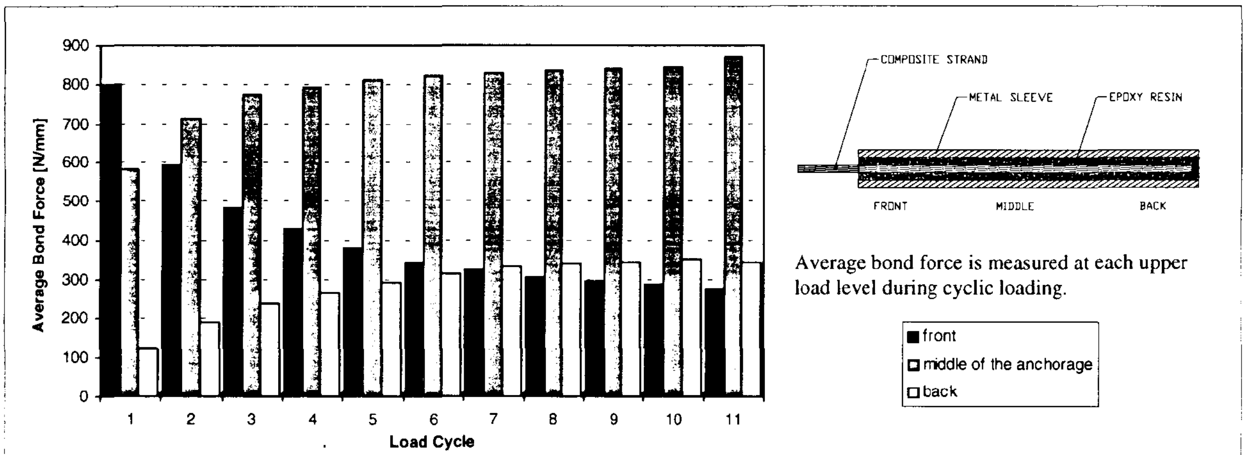

Figure 1. Typical bond force redistribution along a tendon (Ø 15mm) in a resin filled sleeve anchorage. The load introduction shifts to the back of the anchorage.

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3. Recommendations for the Acceptance of Post-Tensioning Systems, FIP Commission on Prestressing Materials and Systems, Published by SETO Ltd, London, 1993.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS RESEARCH ABSTRACT

Title: Optimization of Pultruded GFRP Composite Bridge Railing System

Author(s) and Affiliation(s):

S. Seangatith and Robert L. Yuan
Department of Civil & Environmental Engineering
The University of Texas at Arlington, Arlington, TX

Principal Investigator: Robert L. Yuan

Sponsor(s): The University of Texas at Arlington

Research Start Date: November 1995
Expected Completion Date: August 1996

Research Objectives:

The primary objective of this research is to develop an economic and effective approach for an optimum design of GFRP composite bridge railing systems, particularly targeted to its application to secondary road bridges. A reliable preliminary design should be performed prior to the computer simulation for evaluation of the bridge rails.

Expected Products or Deliverables:

The results from this research program will provide optimized cross sections for the bridge rail components which meet the strength and the functional requirements specified in the AASHTO Guide Specifications.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

S. Seangatith is a Ph.D. candidate in the Department of Civil and Environmental Engineering at The University of Texas at Arlington. He has finished all required course work and is currently working on his dissertation research.

Robert L. Yuan is Professor in the Department of Civil and Environmental Engineering at The University of Texas at Arlington. He has been teaching and conducting research in the areas of structural mechanics and structural materials for the past 30 years. His current research interest is in the application of GFRP composites in civil structures. He has received both teaching and research awards from the College of Engineering at UTA.

Principal Investigator's Telephone Number: (817) 272-2550
Facsimile Number: (817) 272-2630
E-mail Address:

OPTIMIZATION OF PULTRUDED GFRP COMPOSITE BRIDGE RAILING SYSTEM

S. Seangatith and Robert L. Yuan, P.E.

Department of Civil and Environmental Engineering
The University of Texas at Arlington, Arlington, TX 76019-0308

Research Objectives

According to AASHTO Guide Specifications for Bridge Railings, the acceptable performance of a bridge railing system is to be determined by full-scale crash tests. Since the test setup requires a special facility and the cost of crash test is prohibitively high, the cut-and-try method for design is impractical, especially for the preliminary design with prototype bridge rails. An alternative method which can be used for evaluation in the initial stage is the computer simulation. For the application of pultruded glass fiber-reinforced plastic (GFRP) composite bridge railings, a reliable preliminary design needs to be performed prior to the computer simulation due to the complex nature, properties and behavior, of the non-traditional civil structural material. Therefore, the objective of this research is to develop a simple and effective approach for an optimum design of GFRP composite bridge railings, particularly targeted on the application to the secondary road through bridges.

Research Approach

Background: In recent years, fiber-reinforced plastic (FRP) composite has been emerging to be a potential useful material for civil engineering structural components; it possesses the qualities of light weight, high strength to weight ratio, corrosion resistance, high energy absorption and high fatigue strength. The composite material for civil engineering application is mainly composed of the glass fiber embedded in a relatively low-modulus matrix and has been used for structural components such as columns, beams and panels. Feasibility studies on the use of GFRP composite for roadside safety and bridge related components have shown potential and promising applications (3).

The research program in this study was undertaken to optimize the GFRP composite bridge rails based on the ASCE Structural Plastic Design Manual (2). The optimum design procedure had considered six distinct modes of elastic failure including material failures of tension, compression, and shear, compression buckling in the flanges, shear buckling in the webs, and interlaminar shear failure in the flanges. The configurations of the bridge rails and loading patterns were based on the 1989 AASHTO Guide Specifications for Bridge Railings (1). Since only the bridge rail in the secondary road through bridge was considered in this study, the load performance level 1 criteria was used, including the transverse load of 30 kips, longitudinal loads of ± 9 kips, and vertical loads of 12 kips downward or 4 kips upward. The loads were applied over an area of 24 inches along the rail length and 12 inches in the perpendicular direction. Composite box beams supported by I-Section posts, with span in the traffic direction were used for design model which is similar to the bridge railing concept C in the Guide Specifications. The structural analysis for railing system in this study was based on the elastic theory since the behavior of the GFRP composite is generally assumed to be linearly elastic up to failure. The structural design was based on the ultimate strength of the GFRP composite with strength reduction factors due to temperature and other environmental effects. Furthermore, the beams and the posts were optimized independently, the beams were assumed to be simply supported, and the posts were assumed to be a cantilever structural element.

Optimization Procedure: The optimization procedure for the bridge railing design are formulated as follows:

determine $\{x_i\}$ to minimize $V(\{x_i\})$ $\qquad$ $i = 1, \text{----------}, N$
subject to $g_j\ (\{x_i\}) \leq 0$ $\qquad$ $j = 1, \text{----------}, K$
and $x_i, L \leq x_i \leq x_i, U$

where the design variables $\{x_i\}$ are subjected to given constraint equations gj $(\{x_i\})$ to obtain a minimum of the objective function $V(\{x_i\})$.

The design variables include:

a) Geometrical variables
  i) Dimensions of the beams and posts
  ii) Spacing and number of beams and posts
b) Material variables
  i) Ultimate tensile and compressive stresses
  ii) Longitudinal and transverse modulus of elasticity
  iii) Poisson's ratio
  iv) In-plane and interlaminar shear stresses
c) Loading variables
  i) Type of loading
  ii) Performance level
d) Environmental factors
  i) Temperature effects
  ii) Allowable deflection

In this study, the geometrical variables, (a), are the optimized design variables and the others, (b), (c), (d), are fixed.

The objective functions are the volumes of the GFRP beams and posts:

beam volume = $2\ (b_f\ t_f + b_w\ t_w) \times L$, and
post volume = $(2\ b_f\ t_f + b_w\ t_w) \times L$

where b_f and t_f are the width and the thickness of the flanges, b_w and t_w are the depth and the thickness of the webs, and L is the span length of the bridge rail components.

The constraint equations are written in the following forms:

1) The constraints based on the allowable deflections in x- and y-axes,

$$g\,(1) = -\,I_{x,\,opt} \,/\, I_{x,\,min} + 1$$
$$g\,(2) = -\,I_{y,\,opt} \,/\, I_{y,\,min} + 1$$

where $I_{x,\,opt}$ and $I_{y,\,opt}$ are the optimized moment of inertia of the cross-section about x- and y-axes, respectively. $I_{x,\,min}$ and $I_{y,\,min}$ are the minimum moment of inertia of the cross-section about x- and y-axes based on a specified allowable deflection in the x- and y-axes, respectively.

2) The constraints based on the factored ultimate stresses,

$$g\,(3) = f_b \,/\, F_b - 1$$

where f_b is tensile or compressive stress due to the applied loads,

$$f_b = M_x/S_x + M_y/S_y + f_a$$

where f_a is the tensile or compressive stress due to the longitudinal loads, F_b is the allowable tensile or compressive stress.

3) The compression buckling constraints on compression flange

$$g\,(4) = (b_f/t_f) \,/\, (b_f/t_f)_{max} - 1$$

where (b_f/t_f) is the optimized width to thickness ratio of the compression flange, $(b_f/t_f)_{max}$ is the allowable maximum width to thickness ratio of the compression flange.

4) The shear buckling constraints in the web

$$g\,(5) = (d_w/t_w) \,/\, (d_w/t_w)_{max\,1} - 1$$
$$g\,(6) = (d_w/t_w) \,/\, (d_w/t_w)_{max\,2} - (d_w/t_w)_{max\,1} \,/\, (d_w/t_w)_{max\,2}$$

where (d_w/t_w) is the optimized depth to thickness ratio of the web. $(d_w/t_w)_{max\,1}$ is the allowable maximum depth to thickness ratio of the web to develop full flexural stress at the flanges. $(d_w/t_w)_{max\,2}$ is the allowable maximum depth to thickness ratio of the web to develop full in-plane shear stress at the web.

5) The constraints based on in-plane shear stress in x- and y-axes

$$g\,(7) = f_{vh} \,/\, F_v - 1$$
$$g\,(8) = t_{vh} \,/\, F_v - 1$$

where f_{vh} and t_{vh} are the shear stresses due to the transverse and vertical loads, respectively. F_v is the factored ultimate in-plane shear stress.

6) The interlaminar shear constraint in flanges

$$g\,(9) = f_{vi} \,/\, F_{vi} - 1$$

where f_{vi} is the interlaminar shear stress due to the applied loads, F_{vi} is the factored ultimate interlaminar shear stress.

Search Method: The optimization problem in this study is generally referred to as constrained non-linear optimization problem. There are general optimization computer codes that can be used to solve this problem. The code used in this study is based on the sequential quadratic programming with several line search methods.

Expected Products

The primary product that will result from this research program is to provide an alternative selection of bridge railing system made from the pultruded GFRP composite material. The optimized cross section of the bridge components is designed to meet the strength and the functional requirements specified in the AASHTO Guide Specifications. In addition, the composite bridge rails require little maintenance and will provide high corrosion resistance and high energy absorption upon impact.

Preliminary Results

Based on the AASHTO Guide Specifications and the limitation on the dimensions of the railings and the posts, the preliminary results has been obtained. It is concluded that the optimized dimensions for the railings are 6 x 6 - inch box section with a flange thickness of 0.75 in.. a web thickness of 0.6875 in., and a span of 60 in., the optimization procedure is controlled by the allowable deflection constraints. The optimized dimensions for the posts are 6 - in. width by 8 - in. depth I-section with a flange thickness of 0.75 in., a web thickness of 0.625 in., and a height of 27 in., the optimization procedure is controlled by the allowable stress, flange buckling, and the allowable shear stress constraints. The geometric design of the GFRP bridge railing is shown in Fig. 1.

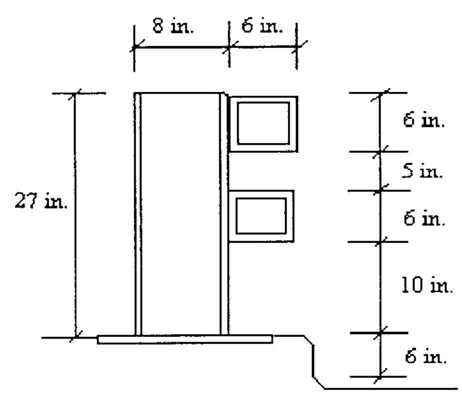

Fig. 1 Geometric design of GFRP bridge railing

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FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Accelerated Test Methods for FRP/Concrete Systems in Highway Structures

Author(s) and Affiliation(s):

Thomas E. Boothby and Antonio Nanni, Department of Architectural Engineering
Charles E. Bakis, Department of Engineering Science and Mechanics
The Pennsylvania State University, University Park, PA

Principal Investigator: Antonio Nanni

Sponsor(s): Federal Highway Administration

Research Start Date: August 1993
Expected Completion Date: August 1997

Research Objectives:

Fiber reinforced plastic (FRP) materials are showing great promise for use in highway structures because of their high strength, light weight, and resistance to corrosion compared to the materials traditionally used in highway bridges and structures. Moreover, the possibility of developing materials with a made to order set of properties is appealing to designers of highway structures. Because of the relative newness of these materials, their long term durability under adverse mechanical and environmental conditions of a highway structure has not been demonstrated. The acceptance of these materials will require the development of accelerated methods to determine their durability over the seventy year life of a highway bridge. This project is part of a three-university effort to develop suitable test methods to determine the long-term properties of FRP materials proposed for use in highway structures. The objective of the team at the Pennsylvania State University is the development of accelerated test methods for evaluation of FRP reinforcing materials to be used in concrete systems.

Expected Products or Deliverables:

It is expected that this research will result in a set of standardized testing procedures, suitable for adoption by ASTM and AASHTO, for acceleration of mechanical and environmental effects on the bond behavior of FRP reinforced concrete systems.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Thomas E. Boothby is Assistant Professor of Architectural Engineering at The Pennsylvania State University, a position he has held for the last four years. Previously, he was a post-doctoral researcher at the University of Nebraska-Lincoln, and a structural designer for various consultants in Albuquerque, NM. He has earned a Ph.D. in civil engineering from the University of Washington, an MSCE from Washington University in St. Louis and a BA in Architecture from Washington University. His research specialty is assessment, maintenance, repair, and rehabilitation of old and historic bridges and other structures.

Antonio Nanni joined The Pennsylvania State University in the fall of 1988 after three years on the faculty at the University of Miami and one year in industry with an Italian architectural-engineering firm. Dr. Nanni's interests are teaching and research in building materials and structures including analysis, experimental evaluation, and design. His area of specialization is in concrete-based materials and structures reinforced with steel or advanced composites (fiber-reinforced plastics or FRP). This interest is rooted in the growing need to enhance performance and durability of newly constructed facilities and to improve repair and strengthening techniques for existing ones.

Charles E. Bakis is an Associate Professor of Engineering Science and Mechanics at The Pennsylvania State University. He is an active member of the Composites Manufacturing Technology Center at Penn State and ASTM Committee D-30 on High Modulus Fibers and their Composites. He has supervised projects on high speed filament winding of thick composite structures, pultrusion of smart hybrid fiber rods, and characterization and modeling of elastic and inelastic behavior of fiber composites. He serves on the editorial board of the Journal of Composites Technology and Research.

Principal Investigator's Telephone Number: (814) 863-2084
Facsimile Number: (814) 863-4789
E-mail Address: axn2@psuvm.psu.edu

ACCELERATED TEST METHODS FOR FRP/CONCRETE SYSTEMS IN HIGHWAY STRUCTURES

Thomas E. Boothby[1], Antonio Nanni[1], and Charles E. Bakis[2]

[1]Department of Architectural Engineering
[2]Department of Engineering Science and Mechanics
The Pennsylvania State University, University Park, PA 16802

Research Objectives

Fiber reinforced plastic (FRP) materials are showing great promise for use in highway structures because of their high strength, light weight, and resistance to corrosion compared to the materials traditionally used in highway bridges and structures. Moreover, the possibility of developing materials with a made to order set of properties is appealing to designers of highway structures. Because of the relative newness of these materials, their long term durability under the adverse mechanical and environmental conditions of a highway structure has not been demonstrated. The acceptance of these materials will require the development of accelerated methods to determine their durability over the seventy year life of a highway structure. This project is part of a three-university effort to develop suitable test methods to determine the long-term properties of FRP materials proposed for use in highway structures. The objective of the team at the Pennsylvania State University is the development of accelerated test methods for evaluation of FRP reinforcing materials to be used in concrete systems.

Research Approach

A key factor controlling the performance of FRP/concrete systems is the bond between the reinforcement and the concrete. The research has concentrated on the development and interpretation of testing methods for determining the bond between reinforcement and the concrete matrix, and the identification of factors that accelerate the degradation of the bond. For non-prestressed reinforcement, the available test methods for determining the bond between reinforcing and concrete were reviewed and a direct pull-out test was identified as the most appropriate and the simplest type of test to implement. Moreover, the direct pull-out specimen has allowed access to the free end of the reinforcing rod for insertion of a non-invasive strain probe. A smaller testing program is also being carried out on the anchorages for post-tensioning reinforcement. The pull-out test was applied to various types of FRP reinforcing rods and backed up by an effort to develop finite element models of the pull-out tests to substantiate and extrapolate the results to situations not covered by the testing matrix. Initially, the testing and modeling effort looked at smooth rods, followed by rods with machined lugs, and finally the methods are being applied to reinforcing rods currently available on the market, as a final test of the proposed accelerated test methods.

Smooth Rods: Three types of smooth rods have been tested: glass fiber/vinyl ester matrix (GV), carbon fiber/vinyl ester matrix (CV), and carbon fiber/epoxy matrix (CE). The smooth rods, with an axisymmetric configuration, can be effectively modeled as transversely isotropic materials, with five elastic constants that were calculated from constituent data furnished by the manufacturer. The testing program on these rods permitted the calibration of the properties controlling the interface between the reinforcing material and the concrete in an axisymmetric finite element model developed for the test specimen. The finite element model allows the exploration of the properties that have the greatest influence on the bond between the FRP and the concrete, in the absence of mechanical interlock, and experimental results can be used to confirm these findings. Rod specimens and complete pull-out specimens were conditioned in deionized water, acetic acid, calcium hydroxide and ammonia. The differences in rod properties were observed and experimental and analytical results were compared. The following table represents the unconditioned and conditioned smooth rod specimens that were tested in this program.

Table 1: Test matrix for smooth rods

Type of test	FRP rod type	Rod diameter	Embed.length	No. of reps
Direct pull-out w/ probe	Glass/Vinyl ester Carbon/Vinyl ester Carbon/Epoxy	12.7 mm	10 d	3
Direct pull-out	Glass/Vinyl ester Carbon/Vinyl ester	12.7 mm	10 d & 5 d	3
	Carbon/Epoxy	6.4 mm	10 d	3
Rod-Rod pull-out	Glass/Vinyl ester	12.7 mm	10 d	3

<u>Machined Rods:</u> To investigate the effect of mechanical interlock, smooth rod specimens were machined to incorporate an axisymmetric pattern of lugs: these lugs develop a mechanical interlock with the concrete that predominates over friction and chemical adhesion in determining the bond behavior of the specimen.

Table 2: Test Matrix for machined rods

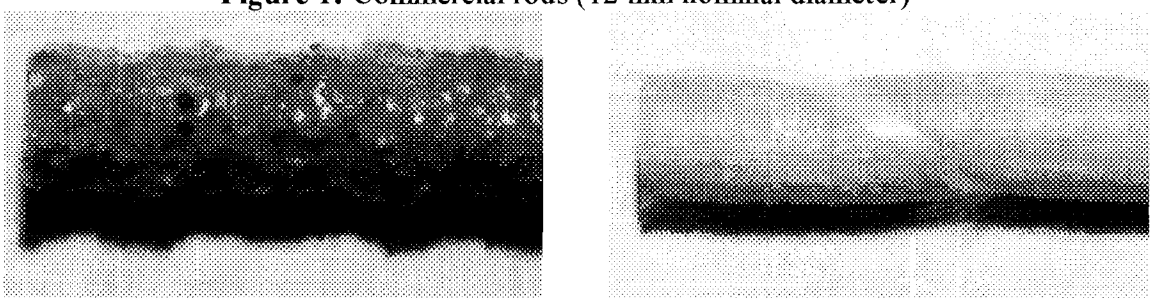
Type of test	FRP rod type	Rod diameter	Embed.length	No. of reps
Direct pull-out w/ probe	Glass/Vinyl ester	12.7 mm	5 d & 10 d	3
	Carbon/Vinyl ester Carbon/Epoxy		5 d	3
Direct pull-out	Glass/Vinyl ester	12.7 mm	5d[1] &10 d[2]	3
	Carbon/Vinyl ester Carbon/Epoxy	6.4 mm	10 d	3
Rod-Rod pull-out	Glass/Vinyl ester	12.7 mm	10 d	3

1. Three different concrete strengths were used (3 repetitions each)
2. Two different lug sizes were used (3 repetitions each)

The axisymmetric finite element model developed for the smooth rod specimens was refined to include the effect of the lugs, and the influence of the lugs on the predicted failure of the FRP and the concrete was investigated, as well as the loss of bond at the FRP/concrete interface. These specimens were tested under virgin and conditioned states and the experimental results were incorporated into the analytical model. The results from rods with multiple lugs were correlated to results from tests of a single lug, with and without environmental conditioning.

Commercial Rods: Two types of commercially available rods were tested, in order to investigate the applicability of the proposed testing methods and accelerating factors to materials available in the marketplace. Photographs of the two rod types selected, designated Manufacturer 'A' and Manufacturer 'B' are shown below. These products have proven extraordinarily difficult to model analytically because of their lack of symmetry, the irregular surface profiles and the apparent variations in matrix properties through the thickness. However empirical pull-out results can be obtained and compared to the results from accelerated tests.

Figure 1: Commercial rods (12 mm nominal diameter)



Three repetitions of each of the commercial rod types were tested in direct pull-out, both for virgin specimens and for specimens conditioned in calcium hydroxide at a pH of 13 at 60° C for 28 days.

Sustained Loading

In addition to environmental effects, the effect of sustained loading is being investigated. Lugged specimens have been prepared and the rods loaded to a fixed percentage of the ultimate pull-out force. After the load is sustained for a specified period of time, the residual pull-out strength of the specimen is tested.

Table 3: Sustained loading test matrix-glass vinyl ester machined rods

% of Ultimate Pullout Strength to be Loaded At	Time of Loading (days)	Number of reps.
(TO BE	7, 28, 84, 224	2
DETERMINED)	7, 28, 84, 224	2

Expected Products or Deliverables

It is expected that this research will result in a set of standardized testing procedures, suitable for adoption by ASTM and AASHTO for acceleration of mechanical and environmental effects on the bond behavior of FRP reinforced concrete systems.

Specific guidelines will be provided for

- Dimensions of pull-out and sustained loading specimens
- Preparation of pull-out and sustained loading specimens
- Conditioning of pull-out specimens
    - Conditioning solutions
    - Mechanical conditioning
- Testing procedure
- Reporting procedure

It is additionally expected that initial work can be done to establish prudent and practical engineering limits on undesirable bonding behavior: pull-out, slip, and internal damage to the rod to aid the bridge design and construction community in the application of FRP reinforcing materials to concrete construction.

Preliminary Results

The bond of smooth rods to concrete is due to chemical adhesion and friction. The latter has proven to be a much more important component, and the friction has been discovered to be developed by hygroscopic swelling of the rod during curing and hardening of the concrete. Specimens chilled in a freezer after curing to counteract the swelling of the rod had no residual bond strength. The bond strength is thus determined by matrix-dominated properties of transverse stiffness and coefficient of hygroscopic expansion. The concrete strength has been shown to have no influence bond strength of smooth reinforcement.

Mechanical interlock of due to lugs or surface roughness becomes an important controlling factor for the bond of the machined and the commercial rods. The ability of the rod to transfer the forces due to mechanical interlock is controlled by the strength of the matrix, as specimens with carbon fibers and a vinyl ester matrix had lower bond strength than specimens with carbon fiber and epoxy matrix. Internal load redistribution between lugs was observed, with observable sequential failures in the rods with ten lugs. Degradation of the matrix due to environmental conditioning factors has been shown to have a clear influence on the residual bond strength of the test specimens. An internal strain probe, developed for this project, has been shown to be a useful tool for investigating force transfer from rod to concrete, without influencing the pull-out load of the rod.

Acknowledgment

This research is funded by the Federal Highway Administration under Contract DTFH61-92-C-0 0012, monitored by Mr. Eric Munley.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: An In-Situ Aluminum Girder Highway Bridge Test and Laboratory Fatigue Tests

Author(s) and Affiliation(s):

Robert E. Abendroth and Wallace W. Sanders, Jr.
Department of Civil Engineering
Iowa State University, Ames, IA

Principal Investigator: Robert E. Abendroth and Wallace W. Sanders, Jr.

Sponsor(s): Iowa Department of Transportation and the Federal Highway Administration

Research Start Date: August 19, 1993
Expected Completion Date: June 30, 1996

Research Objectives:

The two primary objectives of the research were to establish the response of an aluminum girder highway bridge to truck wheel loads and to evaluate the fatigue strength of weld details in full-scale aluminum girders. The first objective involved the confirmation of the applicability of the wheel load distribution criteria contained in current bridge design specifications for a particular aluminum girder concrete deck bridge; the second objective involved the confirmation of specification models of stress range versus load cycle (S-N) behavior for Category E weld details on aluminum girders.

Expected Products or Deliverables:

The research will establish the influence of selected design parameters on the wheel load distribution for a particular aluminum girder bridge and provide additional data to evaluate the fatigue strength of Category E weld details in full-scale, aluminum girders. The field and laboratory test results will provide needed information on the performance and effectiveness of aluminum as a primary structural material for bridge construction.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Robert E. Abendroth is Associate Professor of Civil Engineering at Iowa State University. He received his BSCE in 1966 and his MS and Ph.D. degrees in structural engineering in 1968 and 1983, respectively, from the University of Wisconsin at Madison, Wisconsin. For about 10 years prior to receiving his Ph.D. degree, he was a structural engineer at several architectural and engineering firms, and at the bridge design division of the Department of Transportation in Wisconsin. Since joining the faculty at Iowa State University in 1983, he has been involved in teaching and research in the area of structural engineering. His areas of research related to highway bridges have involved aluminum girders, composite precast-concrete deck panels, intermediate diaphragms, and integral abutments. Some of his other research projects have addressed transmission line galloping, composite metal deck floor slabs, and timber connections. Dr. Abendroth has authored and co-authored many professional papers that have appeared in referred journals and have been presented at technical meetings.

Wallace W. Sanders is Professor of Civil and Construction Engineering at Iowa State University. He received BSCE from the University of Louisville in 1955 and the MS and Ph.D. degrees from the University of Illinois in 1957 and 1960, respectively. After five years on the University of Illinois faculty, he joined the faculty at Iowa State University. He has specialized in design and behavior of railway and highway bridges, and aluminum structures. Dr. Sanders served for nearly 15 years as a member of the College of Engineering and University Administration before returning to a full-time faculty position in 1993. He is a Fellow of ASCE and serves on committees of the American Railway Engineering Association and the American Welding Society.

Principal Investigator's Telephone Number: (515) 294-7435/(515) 294-6048
Facsimile Number: (515) 294-8216
E-mail Address: rabendro@iastate.edu/wsanders@iastate.edu

AN IN-SITU ALUMINUM GIRDER HIGHWAY BRIDGE TEST AND LABORATORY FATIGUE TESTS

Robert E. Abendroth and Wallace W. Sanders, Jr.

Bridge Engineering Center
Department of Civil and Construction Engineering
Iowa State University, Ames, Iowa 50011

Research Objectives

The two primary objectives of the research were to establish the response of an aluminum-girder, highway bridge to truck wheel loads and to evaluate the fatigue strength of weld details in full-scale aluminum girders. The first objective involved the confirmation of the applicability of the wheel load distribution criteria contained in current bridge design specifications for a particular aluminum-girder, concrete-deck bridge; and, the second objective involved the confirmation of specification models of stress range versus load cycle (S-N) behavior for Category E weld details on aluminum girders.

Research Approach

Background: Design specifications for aluminum structures have been developed using theoretical and experimental research involving numerous small-scale and a limited number of full-scale aluminum specimens. Many of the design criteria for aluminum members parallel those for steel members. This is particularly true for the fatigue strength of welded aluminum structures.

Aluminum girder highway bridges are unique structures. Only seven bridges that used aluminum for the major components were built in the United States (1). One of these bridges, shown in Fig. 1, was a four-span continuous, composite, welded, I-shaped, aluminum-girder bridge (2) that was built in 1957 by the Iowa State Highway Commission to carry Clive Road traffic over Interstate 80 near the northwest side of Des Moines, Iowa. Each girder was fabricated from 5083-H113 aluminum plates and contained one field-bolted and five shop-welded full-moment splice connections. The girders were connected to each other by welded, I-shaped, aluminum diaphragms. Shop-welded diaphragm connections between an exterior and an interior girder permitted the field erection of the girders in pairs. The interior diaphragms were field bolted between brackets on the girder webs. Even though the bridge had performed successfully during its 35 years of service and many years of useful life remained for this bridge, it was removed in 1993 as part of an interchange addition and a roadway widening construction project. Just prior to the start of the bridge demolition, static load tests (3) of the bridge were conducted by researchers at Iowa State University (ISU). During the bridge demolition, girders from the two end spans of the bridge were salvaged for laboratory testing.

Testing: The static load field tests of the bridge were conducted by slowly driving a heavily loaded truck to five longitudinal truck positions along six design lanes (lane lines) to produce maximum positive moments at the 0.45 point of span 1 and maximum negative moments near piers 1 and 2.

The transverse load positions were in accordance with the AASHTO Specifications for Highway Bridges (4). Electrical resistance strain gages were attached to the inside face of the flanges of the aluminum girders to measure longitudinal bending strains at the three selected maximum moment locations.

Laboratory fatigue tests were conducted on eight girder sections that were cut from the original end-span sections. Figure 2 shows the two types of specimens that were tested. The four longest specimens contained the shop-welded girder splice and diaphragm connection bracket details that were present in the original bridge. Two new aluminum flange cover plates and four new aluminum horizontal web gusset plates (two plates on each side of the web) were welded to each of these specimens to represent girder attachments that are common for plate girders. The four shortest specimens did not contain any original welded plate attachments. Two new aluminum flange cover plates and four new aluminum web stiffener plates (two plates on each side of the web) were welded to each of these specimens to represent typical girder attachments. A constant-amplitude, sinusoidal-load function with a frequency between about 3 to 5 hertz was incorporated to vary the magnitude of the two equal concentrated loads.

Expected Products

The research will establish the influence of selected design parameters on the wheel load distribution for a particular aluminum girder bridge and provide additional data to evaluate the fatigue strength of Category E weld details in full-scale, aluminum girders. The field and laboratory test results will provide needed information on the performance and effectiveness of aluminum as a primary structural material for bridge construction.

Preliminary Results

A finite-element model of the bridge that included elements for the shear connectors was developed to establish analytical predictions of the girder bending strains for various positions of the truck. A comparison of the predicted and measured girder bending strains (not shown) revealed excellent correlation between the analytical and experimental results. The finite-element model predictions of the girder wheel load distribution factors (not shown) were in close agreement with those obtained from the measured girder bending strains in the field tests. An evaluation of the appropriate wheel load distribution criteria from the Standard Specifications for Highway Bridges (4) and from the AASHTO LRFD Bridge Design Specifications (5) revealed that both specifications predicted conservative distribution factors. However, the LRFD Specification expression that includes the modular ratio for the girder and slab material produced a distribution factor that was more accurate than the factor obtained from the Standard Specification.

Figure 3 shows the test results of the Category E fatigue strengths for the flange splices, flange cover plates, and horizontal web plate attachments. The figure also shows a linear regression line that was established by applying a least-squares, linear-regression analysis to the fatigue data; a lower bound strength line that is two standard deviations below the mean of these test results; and the S-N curves specified by the Specifications for Aluminum Structures (6) and AASHTO LRFD Specifications (5).

The bilinear design curve (solid lines) represents the fatigue strength associated with a 95% confidence limit for a 97.5% probability of survival (or two standard deviations below the mean of the experimental results that were used by Ref. 6). The constant-amplitude, fatigue-threshold stress equals 12.4 MPa (1.8 ksi) at 5 million cycles. The experimental results obtained from the fatigue tests on the bridge girders are in good agreement with the predicted strength given by both specifications.

Acknowledgments

The research discussed in this paper was conducted by the Bridge Engineering Center under the auspices of the Engineering Research Institute of Iowa State University. This investigation was funded through the Iowa Transportation Center at Iowa State University by the Iowa Department of Transportation in cooperation with the Federal Highway Administration. The authors would like to thank engineers and representatives from the Iowa Department of Transportation, Polk County Engineers Office, and Jensen Construction Company and the research staff and research assistants at ISU for their expertise during the research. The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Highway Division of the Iowa Department of Transportation, nor the United States Department of Transportation, Federal Highway Administration.

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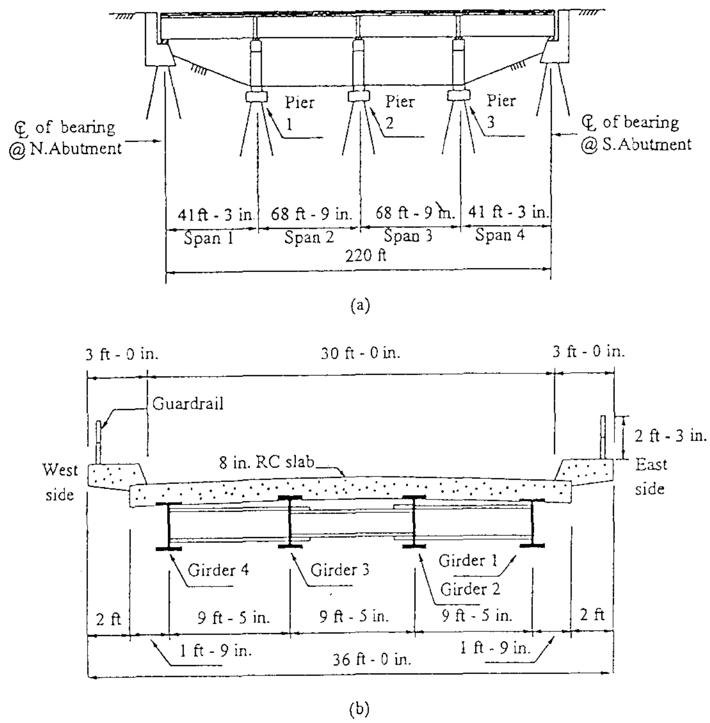

Figure 1. Test bridge: (a) West elevation; (b) Cross section looking north
(1 ft = 305 mm, 1 in. = 25.4 mm)

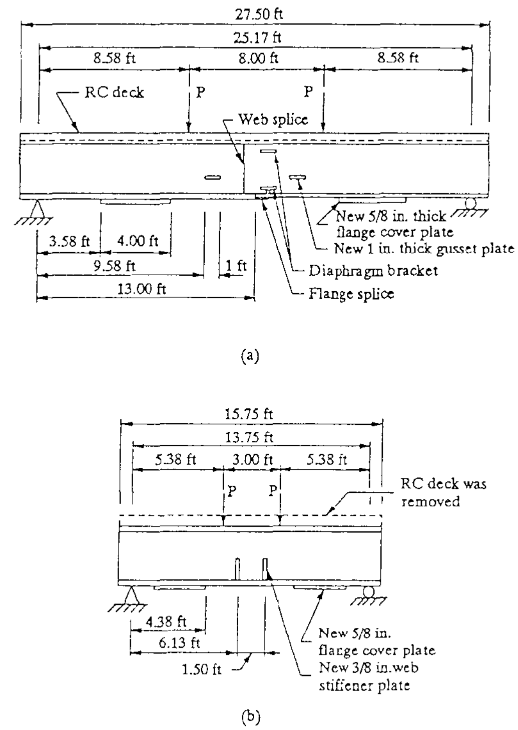

Figure 2. Aluminum girder specimens: (a) Long specimen,
(b) Short specimen (1 ft = 305 mm, 1 in. = 25.4 mm)

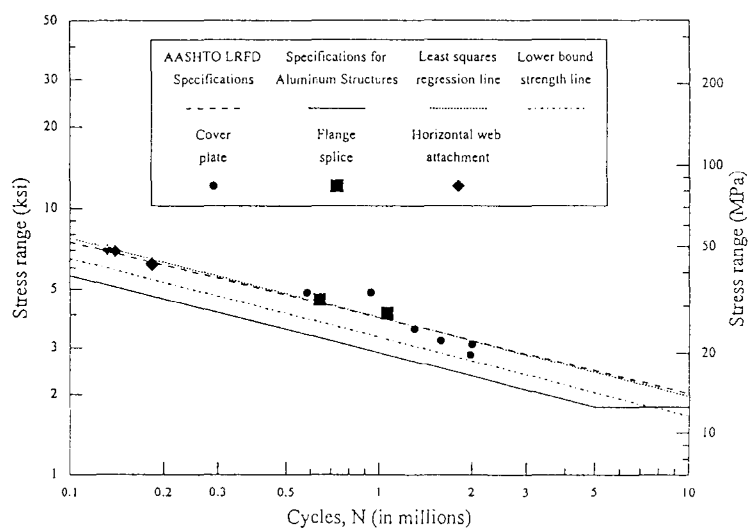

Figure 3. Stress range versus load cycles for category E details

Analysis, Loads, and Bearings

SESSION VII

[redacted]

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Continuing Research in the Analysis and Design of Long-Span Bridges for Wind Loads

Author(s) and Affiliation(s):

Nicholas P. Jones and Robert Scanlan
Department of Civil Engineering
The Johns Hopkins University, Baltimore, MD

Principal Investigator: Nicholas P. Jones and Robert H. Scanlan

Sponsor(s): National Science Foundation and various consulting firms

Research Start Date: On-going
Expected Completion Date:

Research Objectives:

In the half century since the Tacoma Narrows Bridge collapse, the art of wind design for long-span bridges has progressed considerably from the overall examination of modes to an increasingly analytical view, in which the mechanisms affecting bridge stability are emphasized. During this process, which has been aided by mathematical formulations, the essential minima of required experimental data have been identified with some degree of clarity. The engineer needs to be aware of the complex wind interaction forces at the planning, pre-design and design stages. The developed method is based on the use of aerodynamically faithful section models, coupled with analytical modeling and the use of structural dynamic information from a finite element analysis.

Expected Products or Deliverables:

The expected products from the research include answers to many of the questions raised above. In addition, a comprehensive series of guidelines for the conduct of aerodynamic investigation of long-span cable-stayed bridges will be developed. These guidelines will be supported with a computational package and database, available to designers for use and guidance in this aspect of bridge design. In addition to providing a comprehensive annotated bibliography and overview of methods, the guidelines will outline the developed procedures and provide examples of application.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Nicholas P. Jones received his undergraduate degree from the University of Auckland, New Zealand, and his MSCE and Ph.D. from the California Institute of Technology, in the area of structural dynamics. In January 1986, Dr. Jones joined the faculty at The Johns Hopkins University as an Assistant Professor of Civil Engineering. His research interests include various aspects of structural dynamics, flow-induced vibration, earthquake engineering and wind engineering. He is a member of the Dynamics Committee of the Engineering Mechanics Division of ASCE, and of the Committees on Seismic Effects (associate editor) and Aerodynamics of the Structural Division of ASCE. In 1987, he received the George Owen Teaching Award for the Homewood campus of The Johns Hopkins University. He was selected as 1988 Maryland Young Engineer of the Year and in 1989 was awarded a National Science Foundation Presidential Young Investigator award. In 1989 he received the Robert Pond Teaching Award of the Whiting School of Engineering at Johns Hopkins.

Robert H. Scanlan, of the Department of Civil Engineering at the Johns Hopkins University, received his BS and MS degrees from the Armour Institute of the Illinois Institute of Technology and the University of Chicago. He later did graduate studies at M.I.T. and the Sorbonne, Paris, receiving a doctoral degree from each of these institutions. His principal areas of interest include vibrations, stress, structural dynamics, and fluid-structure interaction, with emphasis on the effects of earthquake and wind civil engineering structures. He was formerly chairman of the ASCE Committee on Dynamics and is a member of the Executive Committee of the Engineering Mechanics Division. He also served on the Task Committee on Wind Forces and the Fluid Mechanics of the ASCE and is a member of ASME and the American Society of Mechanics. He is the author of over 150 technical papers and has been consultant to agencies of the U.S. and foreign governments, and private firms. He is co-author of three texts, most recent of which is one on wind engineering (with E. Simiu; Wiley, 1978, 1986). He has carried out research related to wind and earthquake effects upon structures, notably on the stability under wind of long-span bridges and has been consultant on the aerodynamics of a number of bridges. He has received four prizes (ASCE 1968, 1985 and 1986, and AISC 1977) and the Newmark and Croes Medals of ASCE for his work on bridge aerodynamics and wind engineering. He is a Registered Mechanical Engineer and was elected to the National Academy of Engineering in 1987.

Principal Investigator's Telephone Number:	(410) 516-7874
Facsimile Number:	(410) 516-7473
E-mail Address:	nick@jhu.edu

CONTINUING RESEARCH IN THE ANALYSIS AND DESIGN OF LONG-SPAN BRIDGES FOR WIND LOADS

Nicholas P. Jones[1] and Robert H. Scanlan[1]

[1]Department of Civil Engineering, The Johns Hopkins University, Baltimore, MD 21218-2686

Research Objectives

In the half century since the Tacoma Narrows episode, the art of wind design for long-span bridges has progressed considerably from the overall examination of models to an increasingly analytical view, in which the *mechanisms* affecting bridge stability are emphasized. During this process, which has been aided by mathematical formulations, the essential minima of required experimental data have been identified with some degree of clarity. The engineer needs to be aware of the complex wind interaction forces at the planning, pre-design and design stages. The method developed by the authors is based on the use of aerodynamically faithful section models, coupled with analytical modeling and use of structural dynamic information from a finite element analysis

Research Approach

In the past several years a number of significant advances have been made in the approaches for the analysis and design of long-span, cable-supported bridges. These include advances in the theory, in experimental techniques, and in the approach to numerical computation of the dynamic characteristics of these structures. Current specific research efforts include the application of a new multimode theory to the Akashi Kaikyo Bridge in Japan, and the prediction of the performance of the Houston Ship Channel (twin deck) bridge to wind speeds of various intensities for comparison with field data to be collected in the next several years. The overall research program has been an ongoing focus of the authors for a number of years. It is anticipated that within the next 3-5 years, a published comprehensive wind analysis and design procedure will be completed and thoroughly field tested.

This research effort includes detailed study of the following components:

- Structural dynamics and development of suitable numerical modeling procedures (finite element approaches) for the prediction of the dynamic characteristics of long-span bridges.
- Development of improved techniques, procedures, and equipment for wind tunnel experimental modeling at the section-model scale.
- Continuing development of a mathematical model to describe the complex physical mechanisms at work in the interaction of wind with bridge structures.
- Full-scale measurements on completed structures and during construction to verify numerical predictions of dynamic characteristic and (in progress) performance under wind loads.

The methods developed have been used in the design or retrofit of a number of long-span bridge structures over the past decade, including the Houston Ship Channel (Baytown) Bridge, the Kap Shui Mun Bridge (Hong Kong), the Pont de Normandie (France) and the Golden Gate Bridge, in addition to a number of competitive project bids.

While good progress has been made in the understanding of the response of long-span bridges in wind, and many of the concepts that have been developed are now commonly recognized by designers and researchers in various parts of the world, a number of significant issues remain to be addressed. These form the focus of the ongoing research efforts and are outlined below.

<u>Flutter Derivative Determination and Characteristics</u>

Of all the elements of the current modeling approach, perhaps the best-known component is the flutter derivative, now brought into common usage for this application. A number of issues still remain to be resolved, however, in the identification and use of these parameters:

- A critical appraisal of existing identification techniques for determination of flutter derivatives, development of consensus approaches, and of improved techniques.
- A number of sectional forms will be tested using a new 3-DOF force balance to ascertain heretofore unmeasured flutter derivatives, and assess their significance.
- Investigation of the dependence of flutter derivatives on a number of factors (wind angle of attack, amplitude of motion, turbulence in the incoming flow, etc.)
- Investigation of the interrelationships among flutter derivatives for non-streamlined bridge cross sections. Preliminary investigations reveal that some predictable inter-relations may indeed hold for certain types of cross section.
- There still are unresolved questions regarding the matter of Reynolds Number (and other dimensionless group) scaling effects. The use of two-dimensional section models, coupled with *detailed* prototype field measurements will aid in assessing these scaling effects.
- A data base has been developed for aeroelastic and aerodynamic parameters. This data base now needs to be extensively interrogated to look for common characteristics among various section shapes, and to investigate their physical bases.
- The relation of the standard flutter derivatives (linear model) to cases of vortex excitation requires further elucidation, with possible extension to nonlinear models.

<u>Aerodynamic Admittance Determination</u>

The equations of motion currently used are based upon steady-state values of the drag, lift, and moment coefficients. However, these terms actually represent time-varying quantities through the incorporation of the wind gust velocity components. Lift, drag, and moment expressions should include aerodynamic lag effects via convolutions of time-varying functions with their corresponding indicial force-lag functions. Methods for establishing them for bridge decks from experimental data form a focus of the research. Further, their relations to the flutter derivatives remain to be explored.

Signature Turbulence Effects

Existing theory for modeling gust response of bridges does not explicitly account for signature (structurally self-induced) turbulence effects. Only the velocity components of oncoming wind turbulence are currently modeled. However, some representative bluff bridges (such as the original Deer Isle, Maine and the original Tacoma Narrows, for example) responded importantly to self-induced turbulence, even under smooth wind. Such effects can be detected and identified in the wind tunnel using light, rigid bridge deck section models mounted upon high-frequency, motion-sensing supports, as is currently frequently done with models of tall buildings. Appropriate incorporation of these effects into buffeting theory is being made.

Analytical Model Development and Response Prediction Methodology

Recent efforts have investigated extension of the response prediction methodology using a fully coupled, multimode formulation. The new approach is being extended to incorporate some of the new ideas discussed herein, and fully exercised to assist in order to better understand the mechanics of the process. To date, most analyses have focused of the stability and serviceability of primarily the deck structure, although this forms an integral part of the entire structure (and is modeled that way.) The effect of wind on towers and cables, for example, is usually treated with a separate analysis. In reality, the entire structure is excited by the wind simultaneously, and the modeling procedure adopted should incorporate these other components as an integral part of the analysis. The extension is proposed as part of the current effort.

Vortex Shedding

Analytic means for treating bridge response to vortex shedding forces have been outlined earlier where that method is based upon deck section model response studies. Since vortex-induced response is a generally undesirable phenomenon, and it does not occur with all deck cross-sections, finding knowledge-directed means for its suppression in the initial stages of design is a desirable and achievable goal, given the freedom of appropriate aerodynamic treatment. Useful indices as to susceptibility to vortex-induced excitation are available through use of the associated flutter derivatives.

Computational Fluid Dynamics

The complex interaction of wind flow with moving bridge decks of even simple geometry poses significant challenges for numerical flow simulation and force determination. Recent efforts in this area have verified that existing approaches and codes are not well suited to this application, and that new, more reliable and more efficient techniques are needed. At this juncture, improved CFD tools for this application are being investigated. This approach is seen — at this stage in its development — to provide additional detailed insights into the mechanics of the fluid-structure interaction that are difficult to accomplish with analog (physical) modeling.

Super-Long-Span bridge concepts

Currently, the maximum length of a cable stayed bridge is in the 800-900 meter range (Pont de Normandie in France and Tatara Bridge (in construction) in Japan.) The Akashi-Kaikyo bridge in Japan will be the world's longest suspension bridge when completed at 1990 meters. It is clear that as we push to longer and longer spans, wind effects will become even more important than they are currently due to the inherent increased flexibility of the structure. New concepts (e.g., hybrid cable-stayed/suspension bridges) and materials present interesting possibilities for sites such as the Straits of Gibraltar. Before serious consideration can be given to these structures, novel wind design or countermeasures must be developed. It is planned as part of the current efforts, and using the tools developed therein, to investigate some of these issues and propose configurations or approaches which render consideration of these super-long spans feasible.

Wind Related Problems of Cables

A cable-stayed bridge presents several large cable "harps" upon which the wind plays. These cable arrangements, both singly and in groups, give rise to numerous vibration problems such as classic vortex excitation and galloping of several types, including those that are ice-, wind-rain-, wake- and sheared-flow-induced. Cable interaction with (and participation in) full-bridge modes is a related area of interest. Means for suppressing these undesirable oscillations have included various types of tuned absorbers, passive dampers, cable cross ties, various serrated or baffled cable sheathings, flow deflectors, etc. Cable vibrations are known to be deleterious to the internal grouting of cable sheathing, and to cable wires in proximity to their terminations. The present project will examine and compare existing methods for the suppression of cable vibrations and analyze the most effective methods of vibration suppression with a view to the prolongation of stay cable life.

Expected Products

The expected products from the research clearly include answers to many of the questions raised above. In addition, it is proposed to develop a comprehensive series of guidelines for the conduct of aerodynamic investigation of long-span cable-stayed bridges. These guidelines will include all the developments to date and will be supported with a computational support package and database available to designers for use and guidance in this aspect of bridge design. In addition to providing a comprehensive annotated bibliography and overview of methods, the guidelines will outline in detail the developed procedures and provide examples of application. It is hoped that these guidelines and tools will assist in making U.S. bridge designers more competitive in the international long-span bridge market.

Preliminary Results

Due to space limitations, preliminary results are not included in the paper, but will be outlined at the Workshop. Papers published by the authors in recent years provide background and relatively current outlines of results of the ongoing research efforts.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Aerodynamic Investigations of the Deer Isle-Sedgwick Bridge

Author(s) and Affiliation(s):

Harold R. Bosch
Turner-Fairbank Highway Research Center
Federal Highway Administration, Mclean, VA 22101-2296

Principal Investigator: Harold R. Bosch

Sponsor(s): Federal Highway Administration

Research Start Date: August 1981
Expected Completion Date: August 1998

Research Objectives:

The objectives of the study are to evaluate the sensitivity of the bridge to ambient wind conditions at the site, develop retrofit measures which improve the structure's behavior, and elevate our overall understanding of how wind interacts with long-span bridges.

Expected Products or Deliverables:

This research is expected to produce a retrofit recommendation which will enhance the aerodynamic performance of the Deer Isle bridge. The research will lead to improved experimental techniques, more reliable analytical tools, and enhanced technology for structural monitoring. Additionally, the study will produce long term site-specific weather records as well as an extensive database of bridge response data before and after retrofit.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Harold R. Bosch is a Research Structural Engineer and is director of the G.S. Vincent Aerodynamics Laboratory at Turner-Fairbank Highway Research Center of the Federal Highway Administration. He received his BSCE in 1971 from the University of New Mexico. He is currently enrolled in the MS degree program in structures at Colorado State University and in the advanced studies in structures program at the University of Maryland. Mr. Bosch is responsible for the planning, coordination, and conduct of a national program of wind research aimed at ensuring the aerodynamic stability of long-span bridges and other highway structures. The wind engineering program involves analytical research, theoretical research, wind tunnel model studies, in the FHWA Aerodynamics Laboratory, and field measurements on bridges and other structures. Mr. Bosch has published more than 40 articles, technical papers, and research reports on bridge aerodynamic stability, bridge dynamics, wind tunnel model testing, full-scale structural testing, and automated instrumentation systems. He is an active member of the American Association of Wind Engineers and serves as secretary of the Transportation Research Board's committee on Dynamics and Field Testing of Bridges.

Principal Investigator's Telephone Number: (703) 285-2753
Facsimile Number: (703) 285-2766
E-mail Address:

AERODYNAMIC INVESTIGATIONS OF THE DEER ISLE-SEDGWICK BRIDGE

Harold R. Bosch

Turner-Fairbank Highway Research Center
Federal Highway Administration (HNR-10)
6300 Georgetown Pike, Mclean, VA 22101-2296

Research Objectives

Wind loads should be a major factor in the design of any long-span bridge. Even with the most careful attention by the designers and researchers, bridges sometimes still exhibit wind performance problems after being placed into service. This paper will provide a brief overview of a study into the performance of a wind-sensitive suspension bridge. The objectives of the study are: to evaluate the sensitivity of the bridge to ambient wind conditions at the site; to develop retrofit measures which improve the structure's behavior; and, to elevate our overall understanding of how wind interacts with long-span bridges.

Research Approach

Background: In 1981, the Federal Highway Administration (FHWA) entered into a cooperative agreement with the Maine Department of Transportation (MEDOT) and Steinman Consultants to investigate the aerodynamic behavior of the Deer Isle-Sedgwick Bridge. This work was to be part of a program of rehabilitation for the aging structure. This girder-stiffened suspension bridge is situated on the North Atlantic coast of Maine and has had a long history of sensitivity to the wind [1]. The structure has been modified several times since its dedication in 1939. The research program in progress is a fairly extensive one involving wind tunnel tests, full scale measurements, and analytical studies.

Wind Tunnel Tests: Experiments are being performed in the FHWA Aerodynamics Laboratory on a 1:25 scale section model of the bridge. These tests are designed to explore the significance of wind turbulence, bridge shape, structure stiffness and damping, curb grates, and deck venting. Static and dynamic test set-ups are used to evaluate the existing bridge configuration as well as modifications proposed for the bridge. For static tests, the model is mounted in a high-frequency force-balance and subjected to a range of wind speeds and attack angles. For the dynamic tests, the model is mounted in a tuned spring suspension system and response measurements are recorded while simulating a variety of wind conditions. Fixed grids are inserted into the flow for generating small scale turbulence and the active turbulence generator is used when large scale turbulence must be simulated. [2]

Full Scale Measurements: Field measurements are being taken at the site to characterize the wind environment and bridge response. Six tri-axis anemometers are installed along the bridge spans to monitor wind speed and direction at deck level. Two prop-vane type anemometers are mounted at the top and bottom of one tower to measure undisturbed wind conditions near the water and above the bridge superstructure. To measure bridge motion, pairs of single-axis accelerometers are deployed along the spans and at the top of one tower. These sensors are connected to an intelligent data acquisition system which can trigger automatically on selected events. Although data is recorded locally on tape, the system can be interrogated and reprogrammed remotely from the FHWA research lab. [3]

Analytical Studies: Static, dynamic, and aerodynamic analysis is being performed using commercial as well as in-house finite element software. These studies will investigate the structure in each of its stages of evolution as well as any modifications proposed for the bridge. Calculated properties will be used for experiment design and will be compared with full scale measurements. As finite element models are refined, they will be used to predict the dynamic response of the structure to vortex shedding, galloping, buffeting, and flutter. Again, predictions will be compared with measurements and historical records where possible. This analysis will also be used to optimize the implementation of retrofit measures identified during model testing. [4]

Expected Products

This research is expected to produce a retrofit recommendation which will enhance the aerodynamic performance of the Deer Isle bridge. Toward this end, ambient wind conditions will be established, wind sensitivity of the structure will be identified, practical retrofit measures will be assessed, and a recommendation will be made to MEDOT. In the larger picture, this research is expected to provide a better understanding of wind interaction with bridge structures. The complex nature of this structure, storm activity at the site, and history of vibration problems make this a fertile test bed for investigating bridge aerodynamics issues. The research will lead to improved experimental techniques, more reliable analytical tools, and enhanced technology for structural monitoring. Additionally, the study will produce long term site-specific weather records as well as an extensive database of bridge response data before and after retrofit. Results of this work will be diseminated to the profession through published reports, conference and journal papers, and presentations.

Preliminary Results

At this point in the research, most of the wind tunnel testing has been completed, the site has been monitored continuously for 15 years, and the structure has been retrofit with fairings. Wind tunnel experiments indicate that the bridge has poor aerodynamic properties. The existing shape generates strong vortex shedding and local turbulence. There are tendencies for vortex-induced and galloping responses. The curb grates play an important role in balancing pressure differentials and must be kept open in winter months. The structure exhibits poor aerodynamic damping with a relatively low threshhold for vertical flutter. The addition of non-structural fairings should improve the aerodynamic properties. Field measurements indicate that winds in the 15-25 mph (24-40 km/h) range are quite frequent, with violent storms of 40-60 mph (64-97 km/h) occuring periodically during the year. Bridge motion does occur in low, steady winds as well as higher, more turbulent winds. Motion is primarily vertical in the first symmetric and/or antisymmetric modes and motion is augmented when the curb grates become covered with snow. Analysis of the bridge in each of its stages of modification clearly demonstrates the inherent flexibility and wind sensitivity of this structure. The addition of longitudinal, tower, and transverse stays has increased stiffness and damping thus enabling the bridge to resist the wind for many years and avoid catastrophe. The recent addition of fairings to the structure is expected to make it more compatible with the wind environment. This is expected to minimize vortex-induced and galloping motions, reduce turbulent buffeting forces, and increase the margin of safety against flutter instability. [3,4,5]

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[2] Bosch, H.R., "A Wind Tunnel Investigation of the Deer Isle-Sedgwick Bridge (Phase I)", *Report No. FHWA/RD-87/027*, FHWA, Washington, DC, August 1987.

[3] Bosch, H.R., and Miklofsky, H.A., "Monitoring the Aerodynamic Performance of a Suspension Bridge", *Proceedings*, 7th U.S. National Conference on Wind Engineering, Los Angeles, CA, June 1993.

[4] Cai, C.S., *Prediction of Long-Span Bridge Response to Turbulent Wind*, Ph.D. Dissertation, University of Maryland, College Park, MD, June 1993.

[5] Bosch, H.R., "Section Model Studies of the Deer Isle-Sedgwick Suspension Bridge", *Proceedings*, 6th U.S. National Conference on Wind Engineering, Houston, TX, March 1989.

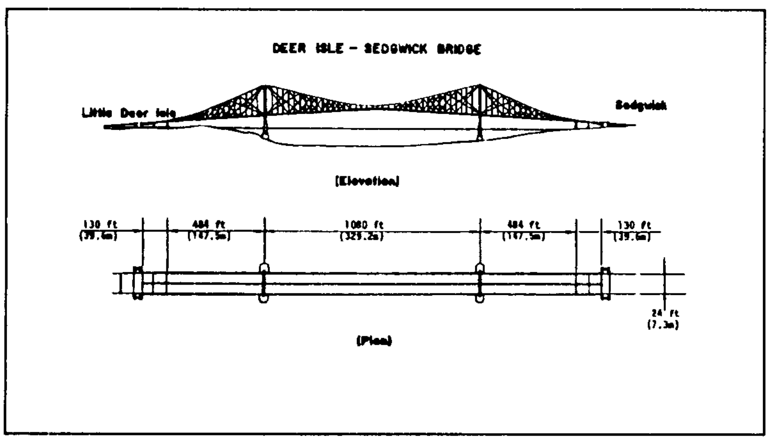

Figure 1. Bridge configuration.

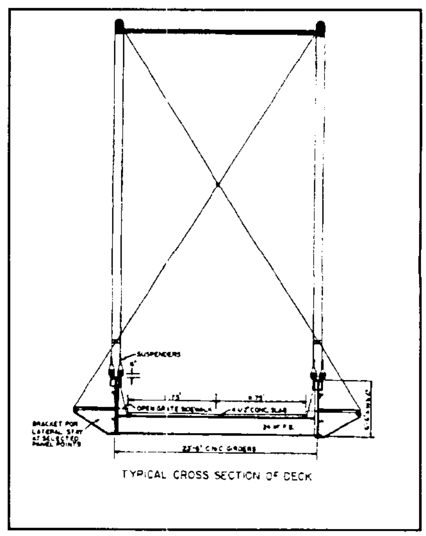

Figure 2. X-section.

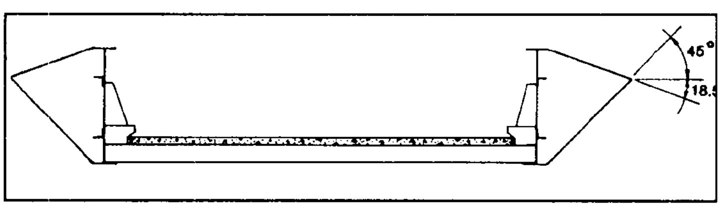

Figure 3. Bridge with fairings.

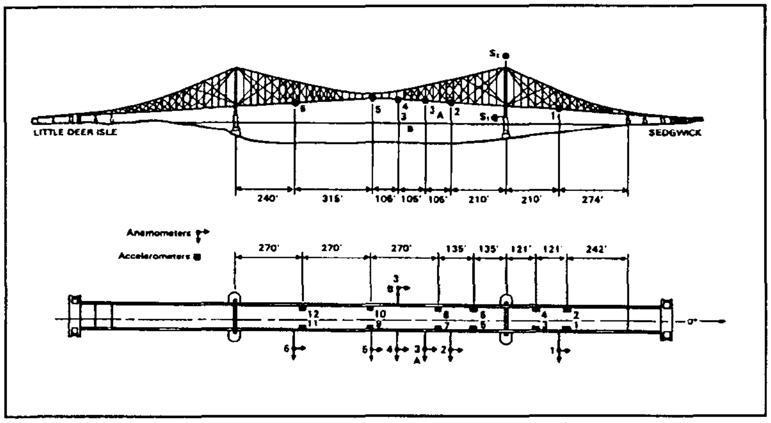

Figure 4. Bridge instrumentation.

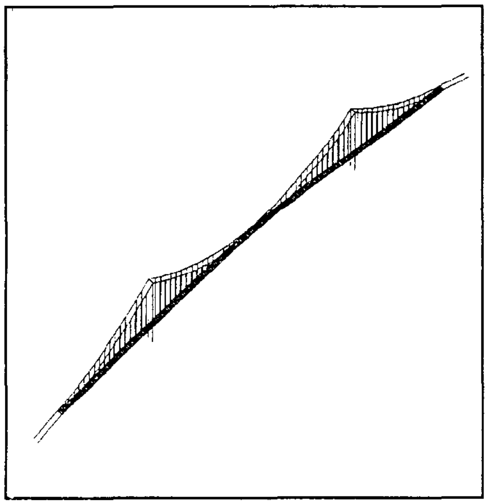
Figure 5. FE model.

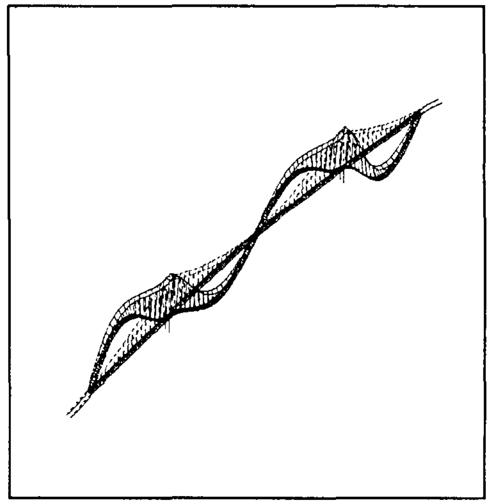
Figure 6. Mode analysis.

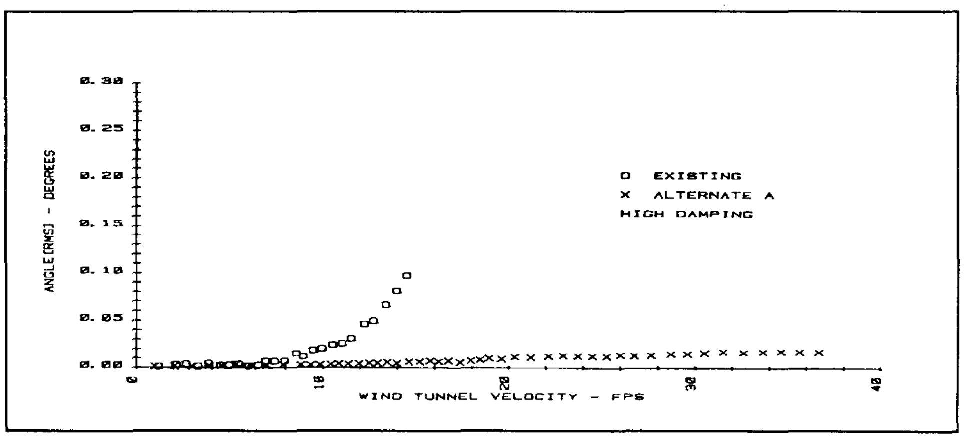

Figure 7. Torsional response with fairings.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS RESEARCH ABSTRACT

Title: Bridge Pier Analysis for Ship Impact and Other Loads

Author(s) and Affiliation(s):

Marc I. Hoit, Mike C. McVay and Clifford O. Hays, Jr.
Department of Civil Engineering
University of Florida, Gainesville, FL

Principal Investigator: Marc I. Hoit and Mike C. McVay

Sponsor(s): Florida Department of Transportation and the Federal Highway Administration

Research Start Date: October 1993
Expected Completion Date: December 1997

Research Objectives:

Bridge pier analysis and design have always been time consuming and difficult jobs for engineers. The analysis should include the effects of the nonlinear structural and soil behavior as well as soil-pile-structure interaction. In order to properly model these effects, the structural model must include the effects of concrete cracking, prestressing, yielding of steel and beam-column stability effects. The soil models must consider both the axial and lateral load-deformation response of the piles or shafts and the spacing and arrangements of the piles or shafts. The nonlinear response is especially important when designing for super events such as ship impact or a severe earthquake. The objective of this long term research program is to develop a computer software package that will be used to accurately model the nonlinear response of bridge pier foundations to three dimensional loads.

Expected Products or Deliverables:
The primary result of this research will be a robust user-friendly program that will be capable of the analysis and design of bridge pier foundations for a variety of loading including super events such as ship impact. The program will operate in a Microsoft Windows environment and include other similar foundation structures such as highmast lighting and sound walls. Cross-sections and materials will be very general, including prestressed and reinforced concrete and structural steel tubes with or without concrete fill. Loadings will be automatically generated.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Marc I. Hoit is Associate Professor of Civil Engineering at the University of Florida. Dr. Hoit has been with the University of Florida for 12 years. He has authored numerous papers on analysis of structures using finite element methods and in the areas of structural optimization and artificial intelligence. Dr. Hoit has developed numerous finite element codes such as SIMPAL and SSTAN, which is included in his textbook, "Computer Assisted Structural Analysis and Modeling," Prentice Hall, 1995.

Michael C. McVay is Professor of Civil Engineering at the University of Florida. Dr. McVay has been at the University of Florida for 15 years. He has authored numerous papers in the areas of soil behavior, modeling of pile foundations, and centrifuge testing.

Clifford O. Hays, Jr. is Professor of Civil Engineering at the University of Florida where he has been for 25 years. He is a registered Professional Engineer in the States of Georgia and Florida. He has authored numerous papers on bridge analysis and nonlinear analysis of frames with slender members. Dr. Hays was the principal developer of the BRUFEM system for bridge rating using the finite element method.

Principal Investigator's Telephone Number:	(352) 392-5006/(352) 392-8697
Facsimile Number:	(352) 392-3394
E-mail Address:	mhoit@ce.ufl.edu/mmcva@ce.ufl.edu

Bridge Pier Analysis for Ship Impact and Other Loads

Marc I Hoit, Mike C. McVay, and Clifford O. Hays Jr.
Department of Civil Engineering
University of Florida, Gainesville, FL 32611

Research Objectives

Bridge pier analysis and design have always been time consuming and difficult jobs for engineers. The analysis should include the effects of the nonlinear structural and soil behavior as well as soil-pile-structure interaction. In order to properly model these effects, the structural model must include the effects of concrete cracking, prestressing, yielding of steel and beam-column stability effects. The soil models must consider both the axial and lateral load-deformation response of the piles or shafts and the spacing and arrangements of the piles or shafts. The nonlinear response is especially important when designing for super events such as ship impact or a severe earthquake. The Florida Department of Transportation and the Federal Highway Administration are sponsoring a long term research program at the University of Florida to develop a computer software package that will be used to accurately model the nonlinear response of bridge pier foundations to three dimensional loads.

Research Approach

The current state of the art design methods usually involve an iteration between several different analysis and design programs in order to create a final design. Pile programs like COM624 (Wang and Reese, 1993) are used to model the soil-pile behavior. Then an approximate, often linear, pier model is developed using a standard structural analysis package. The design is iterated upon between these two analysis tools until a final configuration is choosen. The final loads and moments are then transfered to a pier design program that incorporates the AASHTO load cases and design checks for the concrete and steel sections. The nonlinear effects of structural behavior are only accounted for in the crudest of ways in such a procedure.

The bridge designers at the Florida Department of Transportation thought a better approach could be developed where the entire structure could be analyzed together with all important structural and soil behavioral modes considered, The designer would be able to specify the structure configuration from a simplified input using "design" parameters. As a result, the Florida Pier Analysis Program (FLPIER) is being developed at the University of Florida.

The FLPIER analysis program is a nonlinear finite element program for analyzing bridge pier structures composed of pier columns and cap supported on a pile cap and nonliear piles with nonlinear soil. This analysis program couples standard structural finite element analysis with nonlinear static soil models for axial and lateral loading to provide a robust system of analysis for a coupled bridge pier structure and foundation system. FLPIER performs the generation of the finite element model internally given the geometric definition of the structure and foundation system. Coupled with the analysis program is the graphical pre-processor PIERGEN and post-processor PIERPLOT. These programs allow the user of FLPIER to view the structure while generating the model as well as view the resulting deflections and internal forces in a graphical environment. PIERGEN provides an efficient method to define the configuration of the structure to be analyzed. After analysis, PIERPLOT can plot the structure, the deflected shape under the load conditions and the internal stresses, moments and forces in the members. The major components of the FLPIER model can be seen in Figure 1.

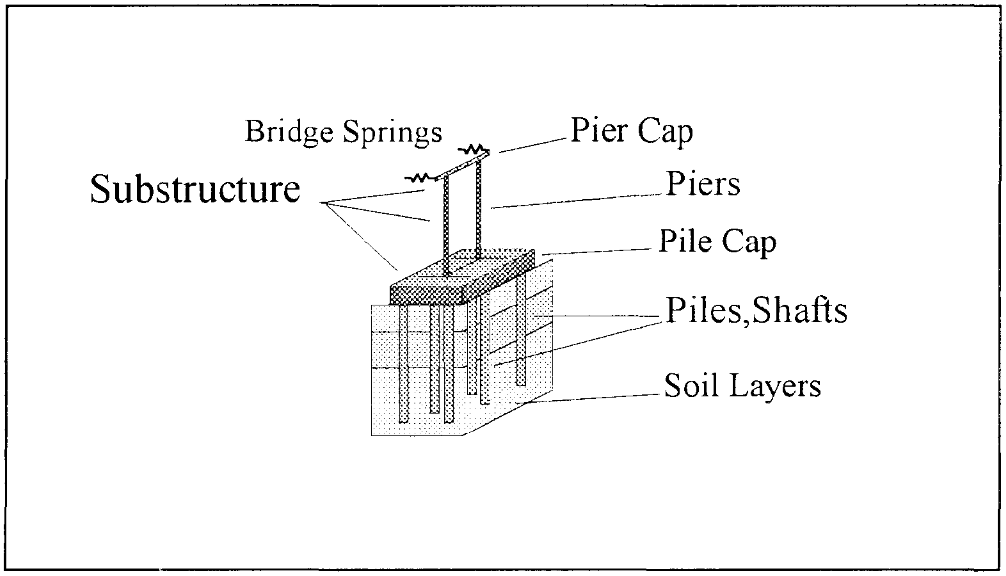

Figure 1. FLPIER Model Components

FLPIER has been designed to be flexible so that it can model many different pile and pier configurations. The piles options include the ability to have individual piles battered, variable spacing between the piles, and piles missing from the group. The soil modeling provides the ability to define the layers of soil at varying depths. Each layer can be either a sand or clay using differnet built in P-Y curves or with user supplied P-Y curves. Both axial and lateral soil interactions are modeled by nonlinear soil springs whose axial and lateral stiffnesses are obtained from the P-Y and T-Z curves. The pile-soil-pile interaction is characterized through the use of user defined P-Y multipliers which are input by row. Research is ongoing to evaluate the soil models and pile-soil-pile interaction by doing centerfuge testing (McVay, et al., 1996).

Structural Finite Element Modeling

The pile cap is modeled using 9-node shell elements. The shell elements are based on the Mindlin theory and use eight point reduced integration to account for shear deformations. They also have in-plane torsion effects included using an equivalent width beam analogy. The pile positions make up the four corner nodes of each element. Additional nodes are placed at the mid points of the elements (between adjacent piles) to give sufficient flexibility for the elements

Each slender member is modeled with 2-node, 3-dimensional discrete elements. These elements model bending in both planes, torsion and axial deformations. The pile connection with the pile cap can be either a pinned or have a fixed head condition.

Discrete Element Model: The discrete element model (Figure 2) considers the nonlinear material and geometric behavior of the piles. The discrete element model originally developed by Mitchell (1973) was chosen because it allows for a simple but accurate model of the nonlinear three dimensional behavior of the slender elements. The nonlinear material behavior is modeled by using input or default stress strain curves which are integrated over the cross-section of the piles. Bending deformations are concentrated at the locations of the discrete rotational springs in the elements. The nonlinear geometric behavior is modeled using the P-Δ moments (moments of the axial force times the displacements of one end of element to another) on the discrete element. And since the user subdivides the pile into a number of sub-elements, the P-Y moments (moments of axial force times internal displacements within members due to bending) are also considered.

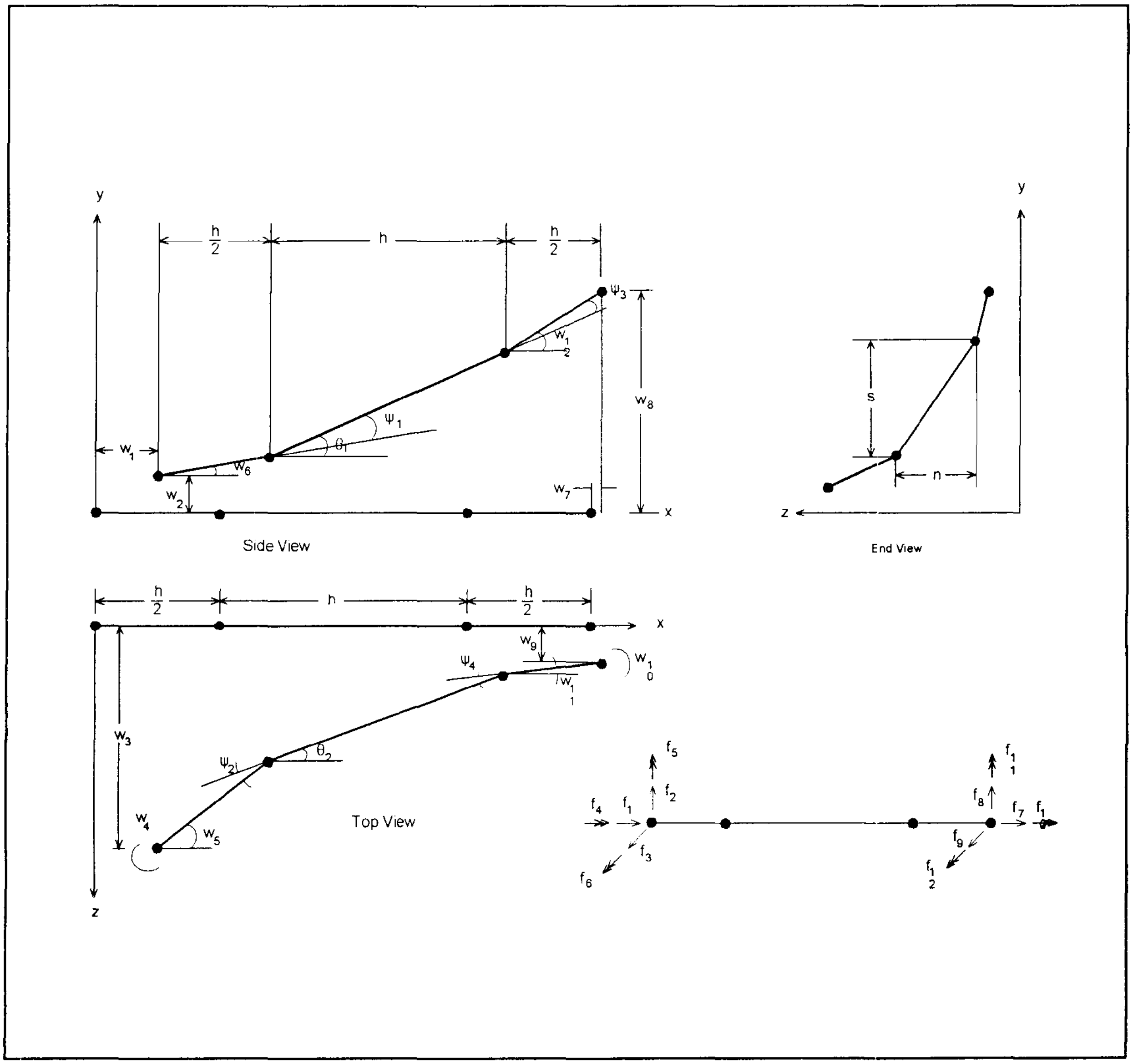

Figure 2. Discrete Element Model

Solution Procedures: Since the soil and pile models are non-linear, FLPIER performs an iterative solution using a secant method approach for the solution of the nonlinear equations. This allows for a more robust solution when using elastic-plastic soil models. At each iteration, FLPIER finds the stiffness of the soil and the piles given the current approximations to the displacements, assembles the stiffness matrix and solves for new displacement values. These displacements are then used to find the internal loads in the pile elements. FLPIER uses the largest value of the out-of-balance forces as the measure of the convergence of the analysis.

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Expected Products

The primary result of this research will be a robust user-friendly program that will be capable of the analysis and design of bridge pier foundations for a variety of loadings including super events such as ship impact. The eventual program will operate in a Windows environment and include other similar foundation structures such as highmast lighting and sound walls. Cross-sections and materials used for the slender structural member will be very general, including prestressed and reinforced concrete and structural steel tubes with or without concrete fill. Loading will be automatically generated from LRFD code criteria and the input of basic data, such as superstructure spans, vehicular traffic and wind parameters.

Preliminary Results

A preliminary version of the FLPIER program is currently being distributed by the Florida Department of Transportation. It is a DOS based program executing on 486 computers with a minimum of 16 megabytes of RAM. This version is primarily analytical and needs to be run several times to develop a design and does not have the generality of the final product. The user must specify the loads completely. However the nonlinear soil characteristics are automatically considered simultaneously with the nonlinear behavior of the pile and shafts. A report on the current program is available (McVay, et al., 1996)

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS RESEARCH ABSTRACT

Title: Dynamics of a Cable-Stayed Bridge During Construction

Author(s) and Affiliation(s):

Nicholas P. Jones and Chris Pettit, Department of Civil Engineering
The Johns Hopkins University, Baltimore, MD
John C. Wilson and Ashraf ElDamatty, McMaster University
Hamilton, Ontario, Canada

Principal Investigator: Nicholas P. Jones and John C. Wilson

Sponsor(s): National Science Foundation and NSERC Canada

Research Start Date: Ongoing
Expected Completion Date:

Research Objectives:

This ongoing research program examines the dynamic characteristics of the eight lane, 1250 ft center-span twin-deck Baytown Bridge (also known as the Fred Hartman Bridge) in Texas. An ambient vibration survey (AVS) was conducted on the bridge just prior to closure of the superstructure at center span. The AVS was complemented by detailed finite element modeling of the bridge at the same construction stage. The results of the AVS identified numerous modes of vibration of the bridge, and many of these modes were reliably identified in the computer model. Reliable predictions of the dynamic behavior during construction is an important aspect in determining the response of the partially completed structure to wind loads.

Expected Products or Deliverables:

The expected products from the entire project include the following: verified finite element models of the structure at two erection stages and in the completed form (the latter AVS has been performed and data are currently being analyzed); development and installation of a monitoring system for the recording of wind loading and wind-induced response; and verification of wind modeling procedures currently in use for the prediction of wind-induced response of long-span bridges.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Nicholas P. Jones received his undergraduate degree (B.E. (Civil) with First Class Honors) from the University of Auckland, New Zealand, and his MSCE and Ph.D. (1986) from the California Institute of Technology, in the area of structural dynamics (flow-induced vibration). In January 1986, Dr. Jones joined the faculty at The Johns Hopkins University as an Assistant Professor of Civil Engineering. His research interests include various aspects of structural dynamics, flow-induced vibration, earthquake engineering and wind engineering. Working with Professor Robert H. Scanlan, he started an experimental research program on aeroelasticity and aerodynamics of civil engineering structures using the low turbulence Corrsin wind tunnel at Johns Hopkins. He is a member of the Dynamics Committee of the Engineering Mechanics Division of ASCE, and of the Committees on Seismic Effects (associate editor) and Aerodynamics of the Structural Division of ASCE. In 1987, he received the George Owen Teaching Award for the Homewood campus of The Johns Hopkins University. He was selected as 1988 Maryland Young Engineer of the Year and in 1989 was awarded a National Science Foundation Presidential Young Investigator award. In 1989 he received the Robert Pond Teaching Award of the Whiting School of Engineering at Johns Hopkins.

Chris L. Pettit, Jr. received his BS in Aerospace Engineering at UCLA in 1991. From 1991 to the present, he has been employed as a Research Aerospace Engineer by the U.S. Air Force in the Palace Knight Program. With support provided in this program, he earned an MS in Aerospace Engineering from UCLA in 1993. From 1993 to 1994, he worked in the Aeroelasticity Section at Wright Laboratory, Wright-Patterson Air Force Base. With additional support from the Palace Knight Program, he is currently pursuing a Ph.D. in civil engineering at The Johns Hopkins University.

John C. Wilson is an Associate Professor of Civil Engineering at McMaster University, Hamilton, Ontario, Canada. He received his undergraduate degree from McMaster, and his MS and Ph.D. (1985) in civil engineering from the California Institute of Technology, in the area of structural dynamics (earthquake engineering).

Ashraf ElDamatty is a postdoctoral research fellow in the Department of Civil Engineering at McMaster University, Hamilton, Ontario, Canada.

Principal Investigator's Telephone Number:	(410) 516-7874
Facsimile Number:	(410) 516-7473
E-mail Address:	nick@jhu.edu

DYNAMICS OF A CABLE-STAYED BRIDGE DURING CONSTRUCTION

Nicholas P. Jones[1], Chris Pettit[1], John C. Wilson[2] and Ashraf ElDamatty[2]

[1]Department of Civil Engineering, The Johns Hopkins University, Baltimore, MD 21218-2686
[2]Department of Civil Engineering, McMaster University, Hamilton, Ontario, Canada L8S 4L7

Research Objectives

This ongoing research program examines the dynamic characteristics of the eight lane, 1250 ft center-span twin-deck Baytown Bridge (also known as the Fred Hartman Bridge) in Texas. An ambient vibration survey (AVS) was conducted on the bridge just prior to closure of the superstructure at center span. The AVS was complemented by detailed finite element modeling of the bridge at the same construction stage. The results of the AVS identified numerous modes of vibration of the bridge, and many of these modes were reliably identified in the computer model. Reliable predictions of the dynamic behavior during construction is an important aspect in determining the response of the partially completed structure to wind loads.

Research Approach

Inherent to the use of ambient vibration measurements to estimate dynamic response characteristics of large structures are several assumptions about the nature of the excitation and response, as well as the behavior of the structure itself. These assumptions allow substantial insight to be obtained by performing the bulk of the data reduction and analysis in the frequency domain. Furthermore, this insight is obtained without a quantitative assessment of the excitation itself.

In ambient vibration studies of large structures, the *output* response power spectral density (PSD) and cross-spectral density are typically the only functions that can be obtained. However, the PSD (under suitable assumptions) may be viewed as an estimate of the system's frequency response function from which the structure's dynamic properties may be calculated. Specifically, the structure's modal frequencies may be estimated from the locations of peaks in the response spectra and modal damping values may be estimated from the half-power points for each peak. Mode shapes can be estimated from cross-spectral data analysis using data from different locations on the structure.

Two Sundstrand QA-700 accelerometers and two Kinemetrics FBA-11 accelerometers were used to measure dynamic response to the ambient wind loads. The Sundstrand QA-700's sensed vertical accelerations at the deck's surface; the Kinemetrics FBA-11's measured lateral accelerations at the deck's surface. Ten primary accelerometer configurations were used to measure the ambient vibrations of the bridge decks. In each case, the accelerometers were affixed directly to the deck's surface with plaster of paris. Measurements were concentrated on the south half of each deck's center span. Each configuration consisted of two accelerometers connected to a dual-channel spectrum analyzer. Accelerometer configurations 1 through 5 were selected to obtain response measurements for use in classifying modes as bending, torsional, lateral, or some combination thereof. The corresponding mode shapes were obtained from measurements in configurations 6 through 10, which were composed of "a" and "b" portions devoted to recording the east deck's vertical and horizontal motion, respectively.

In each transducer configuration, the acceleration power- and cross-spectral density functions were computed in the field by the spectrum analyzer. The resulting data files were stored on diskettes for later analysis.

Frequencies and mode shape estimates were obtained for each *distinct* response peak[1] in accelerometer configurations 6 through 10. Acceleration spectra from configurations 1 through 5 were examined using similar techniques to characterize the motion corresponding to each spectral peak as bending, torsion, or lateral. In general, lower frequency mode shapes were more easily identified than those at higher frequencies. The calculation of mode shapes from the response measurements in configurations 6 through 10 was facilitated by using one fixed station for each of these configurations while the other accelerometer was positioned at various locations along the center span.

Expected Products

The expected products from the entire project include the following:

- Verified finite element models of the structure at two erection stages and in the completed form (the latter AVS has been performed, and data are currently being analyzed.).
- Development and installation of a monitoring system for the recording of wind loading and wind-induced response.
- Verification of wind modeling procedures currently in use for the prediction of wind-induced response of long-span bridges.

Preliminary Results

Included below are some preliminary results from the erection-stage AVS. It is evident from these data that while a reasonable comparison is possible, there are still several discrepancies that need to be addressed, and are currently under investigation.

Modal predictions from a pre-construction finite element model of the bridge were available for direct comparison with the AVS modes discussed above. The model was produced by Greiner Engineering Inc., the bridge's designer, and will be referred to as "FE1". An independent finite element model of the same erection stage was employed for more extensive comparison with the AVS modal estimates. This model was developed at McMaster University and will be referred to as "FE2".

Table 1 lists the correspondence between modes from the ambient vibration survey and both finite element models. Composite descriptions of each mode in Table 1 were chosen to reflect those traits observed in each modal estimate when a mode was captured by at least two of the three approaches. Because Table 1 is constructed around the AVS results, these descriptions attempt to separate response of the center span from other portions of the structure, which were

[1] Invariably, some modes are not properly excited or captured in ambient vibration studies. The consequent lack of consistent measurements precludes them from further examination.

examined only in the finite element models. For example, significant sidespan bending in a particular FE mode is cataloged in parallel with any associated center span bending. Note that some sidespan only modes are not included for compactness of the table.

Figure 1 illustrates the shape of the first two modes observed in the AVS as well as the associated finite element mode shapes. The AVS mode shapes were normalized to a maximum displacement of unity. The corresponding FE mode shapes were scaled to a displacement of unity at the node closest to the location at which the maximum displacement was recorded in the AVS mode.

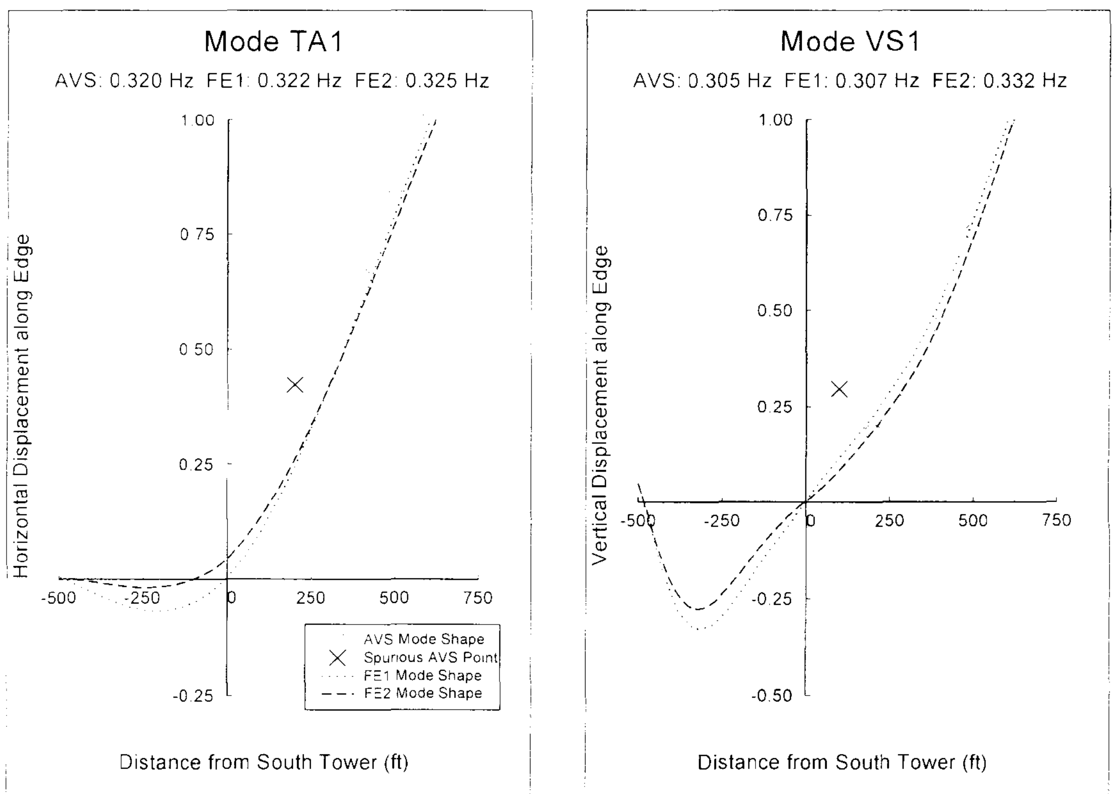
Examination of all the data shows generally good agreement between the finite element and AVS modes at the lower frequencies. Higher modes show less conformity, although certain of these are well correlated. A majority of the AVS modes in Table 1 correspond to finite elements but gaps exist in the FE mode tables for which no matching AVS modes were identified. They occur primarily because both FE models incorporate sidespan motion, which was not measured directly in the AVS. The bridge decks are relatively stiff in torsion, for Table 1 show that the lowest torsion frequency was in each model at least twice the fundamental bending frequency. This conclusion is further supported by the difficulty encountered in documenting the lowest torsion mode in the AVS.

Table 1. Group listing of all AVS and FE modes.
(T: transverse; V: vertical; S: symmetric; A: antisymmetric)

AVS Mode	FE1 Mode	FE2 Mode	Frequency (Hz)			Composite Description
			AVS	FE1	FE2	
-	1	1	-	0.281	-	TS1
1	2	3	0.305	0.307	0.332	VS1
2	3	2	0.320	0.322	0.325	TA1
3	4	4	0.350	0.330	0.340	VA1
-	5	-	-	0.417	-	Tower Spanwise Bending (S)
4	7	5	0.565	0.628	0.609	VS2
5	6	6	0.580	0.618	0.615	VA2
-	8	-	-	0.708	-	Tower and Pier Transverse (S)
-	-	7	-	-	0.716	XS1
-	-	8	-	-	0.716	XA1
-	-	9	-	-	0.760	VS
-	-	10	-	-	0.760	VA
-	9	-	-	0.784	-	TS2, VA3, Sidespan Bending (A),Tower Spanwise and Transverse Bending
-	10	-	-	0.784	-	TA2, VS3, Sidespan Bending (S), Tower Torsion
-	11	-	-	0.790	-	VS3, Sidespan Bending (S)
-	12	-	-	0.798	-	VA3, Sidespan Bending (A), Tower Transverse
6	-	-	0.875	-	-	Transverse
7	13	11	0.890	0.901	0.907	Torsion, TS
-	14	-	-	0.903	-	TA, Tower Torsion
-	15	-	-	0.924	-	Anchor Pier Transverse (S), Tower Transverse (S)

AVS Mode	FE1 Mode	FE2 Mode	Frequency (Hz)			Composite Description
			AVS	FE1	FE2	
8	19	12	0.935	1.016	0.971	VS3, Sidespan Bending (S)
-	16	-	-	0.972	-	TA, Tower Torsion (sim. to Mode 14, FE1)
-	17	-	-	0.973	-	Anchor Pier Transverse (S), Tower Transverse, VA3
-	18	13	-	1.010	0.978	VA3, Sidespan Bending (A)
9	-	14	1.130	-	1.023	XS2
-	-	15	-	-	1.026	XA2
-	20	-	-	1.050	-	TA
10	-	21	1.280	-	1.328	VS4
11	-	22	1.305	-	1.330	VA4

Figure 1. First two AVS and FE modes.



FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS RESEARCH ABSTRACT

Title: Longitudinal Loads in Railway Bridges

Author(s) and Affiliation(s):

Douglas A. Foutch and Daniel H. Tobias
Department of Civil Engineering
University of Illinois at Urbana-Champaign, Urbana, IL

Principal Investigator: Douglas A. Foutch

Sponsor(s): Association of American Railroads and the National Science Foundation

Research Start Date: August 1, 1995
Expected Completion Date: October 31, 1996

Research Objectives:

AC-traction locomotives have recently been introduced into service on heavy-haul rail lines. These locomotives have a higher tractive effort than standard DC-traction locomotives. The larger than usual longitudinal tractive and braking forces transmitted through the track structure to railway bridges has caused some concern in the industry. The forces and stresses in typical bridges that are caused by AC-traction locomotives are currently under investigation. A number of analytical studies on common types of railway bridges are currently under way. In the near future, an experimental study of a railway bridge subjected to AC-traction locomotive loadings is planned.

Expected Products or Deliverables:
An estimate of the longitudinal loads transmitted to critical members of typical railway bridges by AC-traction locomotives is the primary focus of this research program. Further utility is gained from the finite element modeling of the track structure. In general, most studies of the longitudinal loads in track structures caused by passing trains have tended to concentrate on more localized effects. The currently developed models employ modern finite element techniques which allow researchers and practicing engineers to better simulate actual loading and response conditions in the field.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Douglas A. Foutch received his BSCE from the University of Illinois at Urbana-Champaign in 1970. After working for Caterpillar Tractor in Peoria, IL for one year, he attended the University of Hawaii where he received his MSCE in 1973. He received his Ph.D. from the California Institute of Technology in 1976, and then joined the faculty of Civil Engineering at the University of Illinois, where he currently holds the rank of Professor. Dr. Foutch's research interests cover a broad area of structural engineering. His major areas include earthquake engineering related to the design and behavior of buildings and bridges, fatigue evaluation of railway bridges and wind loads on structures. A current research project is on the use of viscoelastic dampers to reduce the earthquake response of non-ductile reinforced concrete buildings. He is also studying the longitudinal forces transferred to railway bridges under revenue traffic, and is investigating ways to retrofit steel buildings for seismic loads. Dr. Foutch has served on many national committees and as chair of the ASCE Committee on Seismic Effects and the Administrative Committee on Dynamic Effects. Dr. Foutch is currently head of the steel team on a national program funded by FEMA to develop a national design specification for rehabilitation of buildings for seismic loads. Dr. Foutch has over 70 publications in the area of structural engineering and has won two ASCE awards for papers on seismic behavior of buildings and bridges under seismic loads. He is a licensed Structural Engineer in Illinois.

Daniel H. Tobias received his BSCE (Magna Cum Laude) in 1988 from Virginia Tech. He received the degrees of MS (1990) and Ph.D. (1994) from the University of Illinois at Urbana-Champaign (UIUC), where his work focused on structural engineering. Dr. Tobias was the primary research assistant in a large scale railway bridge research program. Over a four year period, 10 bridges were instrumented and tested. Loading spectra for various current types of freight and response spectra for the bridges under long term loadings were developed. The culmination of the program was a new fatigue evaluation methodology for girder bridges, invented by Dr. Tobias. In addition, Dr. Tobias consulted on a research program investigating a new seismic base isolation system based on pendular action while at UIUC. Dr. Tobias helped form Structuredyne Consultants, Inc. (SCI) and is the current president. To date, SCI's main work has been the development and teaching of a seminar on the design of highway bridges for seismic loads. Dr. Tobias is currently studying longitudinal loads in railway bridges. He is the author or co-author of over 20 publications related to bridge and earthquake engineering, and a member of ASCE and AAR's railway bridge research steering committee.

Principal Investigator's Telephone Number:	(217) 333.6359
Facsimile Number:	(217) 333.9464
E-mail Address:	dfoutch@civilgate.ce.uiuc.edu

LONGITUDINAL LOADS IN RAILWAY BRIDGES

Douglas A. Foutch[1] and Daniel H. Tobias[1]

[1]Department of Civil Engineering
University of Illinois at Urbana-Champaign

Research Objectives

AC-traction locomotives have recently been introduced into service on heavy-haul rail lines (Foutch, et al, 1996). These locomotives have a higher tractive effort than standard DC-traction locomotives. The larger than usual longitudinal tractive and braking forces transmitted through the track structure to railway bridges has caused some concern in the industry. The forces and stresses in typical bridges that are caused by AC-traction locomotives are currently under investigation. A number of analytical studies on common types of railway bridges are currently under way. In the near future, an experimental study of a railway bridge subjected to AC-traction locomotive loadings is planned.

Research Approach

An open deck through plate girder bridge with a span of 75 ft. was chosen as the primary structure for study in the analytical investigations. A plan and profile view of the bridge is presented in Fig. 1. As shown, the bridge is comprised of 4 floor beams at intervals of 25 ft. and dual stringers between the beams which are 5 ft. from each main girder. Other structures under investigation include open deck through girder bridges with spans of 100 and 150 ft., open deck plate girder bridges with spans of 50 ft. and 75 ft., and a Warren truss with a span of 156 ft.

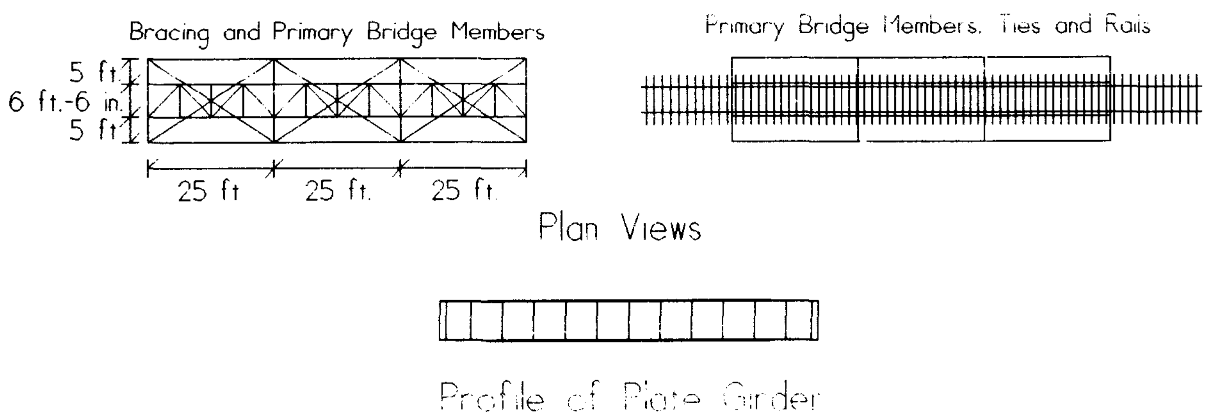


Fig. 1 Schematic of the Primary Bridge Chosen for Study

Two primary types of structural models were employed to determine the forces and stresses in critical bridge members. Models of the track structure which simulated typical types of anchoring patterns were first used to determine longitudinal and vertical tie reactions. The reactions were then applied to a finite element model of a bridge which is constructed of standard spaceframe elements. The track structure models used four plane strain isoparametric elements for each tie, beam elements for the rails and boundary spring elements at the bottom of the ties and at the ends of the rails. A generalized schematic of a track structure model is presented in Fig. 2. Only a portion of the model is shown in the figure because each one was close to 1500 ft. in length. This length was necessary because longitudinal loads caused by passing trains are present in the track structure over very long distances. Appreciable longitudinal rail force 600 to 800 ft. ahead of lead locomotives has been measured in the field (El-Sibaie, et al, 1994). Train loadings with AC-traction locomotives were placed on the models such that maximum longitudinal loads were transmitted to the bridges under investigation.

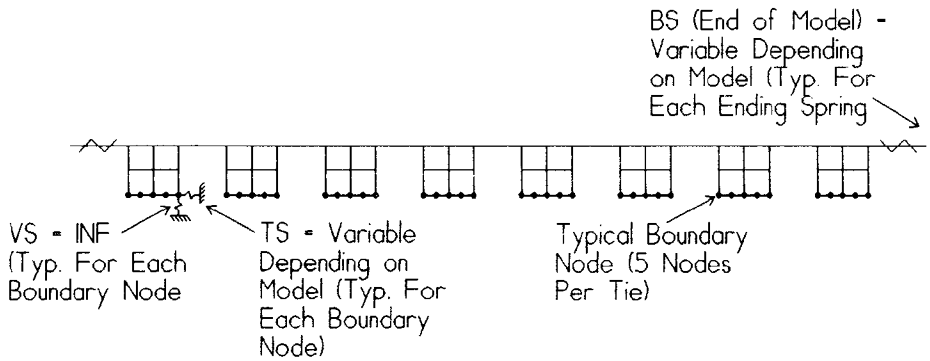


Fig. 2 Schematic of a Track Structure Model

Expected Products

An estimate of the longitudinal loads transmitted to critical members of typical railway bridges by AC-traction locomotives is the primary focus of this research program. Further utility is gained from the finite element modeling of the track structure. In general, most studies of the longitudinal loads in track structures caused by passing trains have tended to concentrate on more localized effects. The currently developed models employ modern finite element techniques which allow researchers and practicing engineers to better simulate actual loading and response conditions in the field.

Preliminary Results

The longitudinal rail forces for track models 6 and 7 for a 75 ft. span bridge are presented in Fig. 3. The resulting tie reactions are shown in Fig. 4. Track models 6 and 7 are two of 9 that have currently been developed. These two models most closely emulate the anchoring pattern which is prevalent on many railroads in North America. The loading is designated T2. This loading had the three lead locomotives approximately centered on the bridge and is for tractive effort. As shown in Figs. 3 and 4, the heavy line indicates model 7 and the lighter line indicates model 6. The longitudinal boundary springs for the ties on the bridge in model 7 were infinitely stiff and for model 6 the stiffness was equivalent to 5 kips/in. for all ties. In both models, every other tie off the bridge and every tie on the bridge was considered to be anchored. The tie reactions from these models bound expected results for the most typical anchoring pattern.

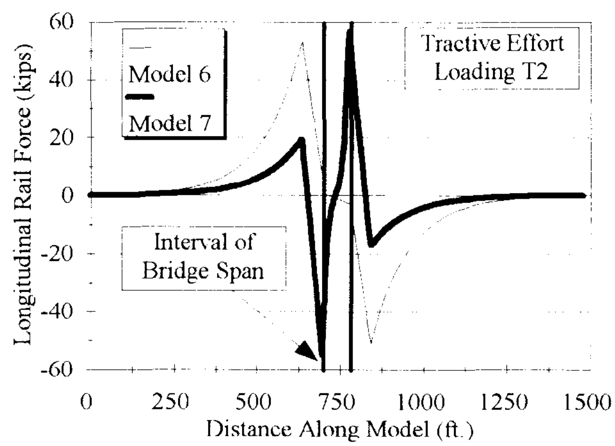


Fig. 3 Longitudinal Rail Forces for Models 6 and 7 Subjected to Loading T2

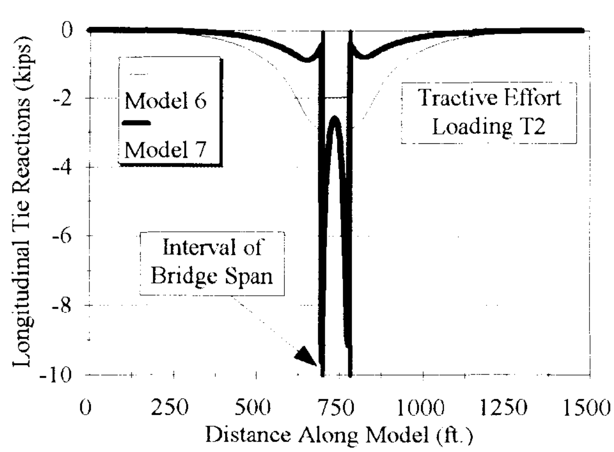


Fig. 4 Longitudinal Tie Reactions for Models 6 and 7 Subjected to Loading T2

Table 1 presents the forces for important bridge members for the 75 ft. open deck through girder span under study. As in the previous two figures, the results are for loading T2. The longitudinal force in the fixed bearing and the maximum force in the lateral bracing are considered critical. The bearing and brace forces reported here are larger than those currently assumed for design.

Bridge	Model	Loading	Bearing Long. Reac. (kips)	Bearing Vert. Reac. (kips)	Long. Brace Force (kips)	Long. Brace Stress (ksi)	Max. Long. Tie Reac. (kips)
75 ft. Through Girder	6	T2	97.9	138.0	25.7	4.2	2.0
	7	T2	219.0	138.0	58.1	9.5	9.6

Table 1. Forces in Critical Members for an Open Deck Through Girder Bridge Subjected to Tie Reactions from Models 6 and 7 for Loading T2

Acknowledgements

The funds for this project were provided by the Association of American Railroads and the National Science Foundation under NSF grant number CMS 94-02224. The support of both of these organizations is acknowledged. The information and cooperation provided by member railroads of AAR is also acknowledged. All observations, findings and conclusions are those of the authors and do not necessarily represent those of the sponsors.

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FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Tests of Kevlar Prestressed Timber Beams for Bridge Girder Applications

Author(s) and Affiliation(s):

Terrel Galloway, Christian Fogstad, Jay A. Puckett and Charles W. Dolan
Department of Civil and Architectural Engineering
University of Wyoming, Laramie, WY
Mike Ritter, Forest Products Laboratory, Madison, WI

Principal Investigator: Christian Fogstad

Sponsor(s): USDA Forest Products Laboratory and DuPont

Research Start Date: January 1995
Expected Completion Date: May 1996

Research Objectives:

This research examines the performance of glued-laminated timber beams reinforced with unstressed and prestressed Kevlar. In particular, the flexural strength of both unstressed and prestressed beams are compared against unreinforced control beams. The shear strength of the Kevlar-wood bond is also examined.

Expected Products or Deliverables:

Expected products include new knowledge on the behavior of Kevlar reinforced and prestressed glued-laminated timber beams. This knowledge may lead to timber beams which may be used to fabricate bridges or building girders. In addition, the prestressing method may be used to rehabilitate existing timber girders.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Terrel Galloway is a Graduate Research Assistant for the Department of Civil and Architectural Engineering at the University of Wyoming, Laramie, Wyoming.

Christian Fogstad is an Engineer at Centennial Engineering, Gillette, Wyoming.

Charles W. Dolan is a Professor in the Department of Civil and Archtectural Engineering at the University of Wyoming, Laramie, Wyoming, and is a registered Professional Engineer.

Jay A. Puckett is a Professor in the Department of Civil and Architectural Engineering at the University of Wyoming, Laramie, Wyoming.

Mike Ritter is a Research Engineer with the USDA Forest Products Laboratory, Madison, Wisconsin, and is a registered Professional Engineer.

Principal Investigator's Telephone Number: (307) 766-2223
Facsimile Number: (307) 766-4444
E-mail Address:

TESTS OF KEVLAR PRESTRESSED TIMBER BEAMS FOR BRIDGE GIRDER APPLICATIONS

Terrel L. Galloway[1], Christian Fogstad[2], Charles W. Dolan P.E.[3], J. A. Puckett P.E.[4], and Mike Ritter P.E.[5]

[1,2,3,4] Department of Civil Engineering
University of Wyoming, Laramie, WY 82070

[5] Forest Products Laboratory
Madison, WI 53705

Research Objectives

The ability to utilize laminated wood members for certain structural applications is often limited by their relatively low bending strength and stiffness when compared to other materials such as concrete and steel. A possible resolution is to use high strength fiber reinforced plastic (FRP) to reinforce timber glued laminated timber (glulam) members. Advances in fiber-reinforced plastics coupled with the increased availability of synthetic fibers have made fiber reinforced wood composites a viable alternative for reinforcement and prestressing tendons in timber. Glulam beams reinforced with FRP materials, when designed to fail plastically, generally exhibit higher bending strength, bending stiffness, and ductility while also displaying a reduction in mechanical variability. In addition, pretensioning the FRP reinforcement may further increase the bending strength of the member. Initial prestress of the member may be used to control long term deflections and tension failures in much the same way it does for prestressed concrete. These advances in the structural properties and behavior of glulam beams may enable smaller wood members or members with lower grades of wood to be substituted for larger members currently manufactured with higher quality materials. Alternatively, it may allow large timber structures, such as long span bridges, to be as strong, durable, and economically feasible as those constructed from other materials.

This research examines the performance of glulam beams reinforced with non-stressed and prestressed Kevlar reinforcement. The primary objective is to determine how the Kevlar reinforcement and prestress affect the flexural strength and stiffness of glued laminated timber beams. The flexural strength and stiffness of both non-stressed and prestressed Kevlar reinforced glulam timber beams are compared to non-reinforced control beams. Other research objectives address the bond strength of the Kevlar fiber reinforcement to wood interface, creep behavior, and finger joint effects.

Research Approach

The main objective of the research is to compare the flexural strength and stiffness of both non-stressed and prestressed Kevlar reinforced glulam timber beams to non-reinforced control beams. In order to achieve this objective, a series of prestressed reinforced, non-stressed reinforced and control beams were constructed and tested in flexure. The test matrix, shown in Figure 1, summarizes the test beams.

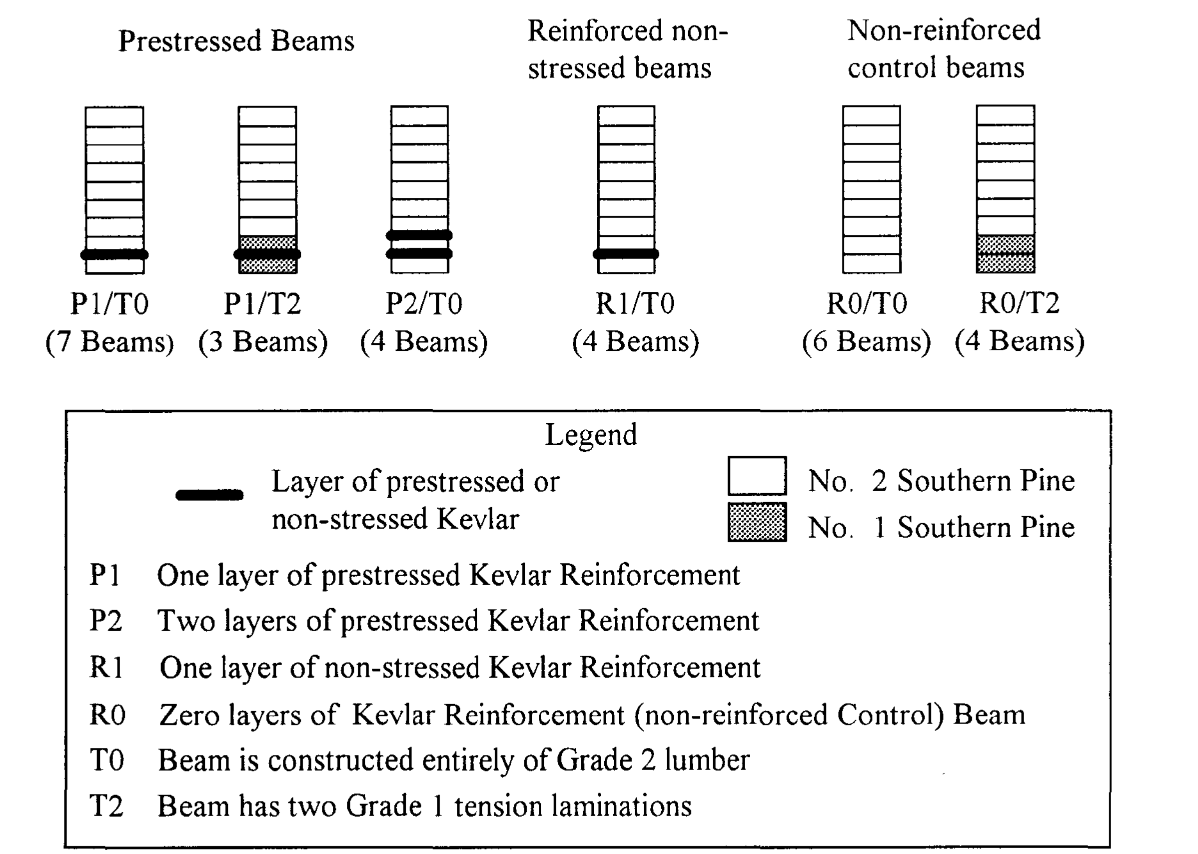

Figure 1 Test matrix of prestressed, reinforced, and control beams.

The beams were constructed from 9 laminations of Southern Pine (SP) lumber. The test beams have a width of 3.5 inches (8.9 cm), a height of 12.375 inches (31.43 cm), a length of 18 feet (5.49 m), and were glued with Indspec R600 adhesive. The Kevlar reinforcement consists of woven tapes of bare unidirectional Kevlar fibers having a cross sectional area of 0.0456 square inches (0.294 sq. cm) per tape. The Kevlar reinforcement was pretensioned prior to gluing it between selected laminations in the beams. Typical prestressing forces ranged from 9,000-10,500 lb.(40-47 kN) for beams with one layer of prestressed reinforcement to 15,000-17,700 lb.(67-79 kN) for beams with two layers of prestressed reinforcement.

The beams were tested in accordance with the flexure test standard specified by ASTM D 198 - 84 " Standard Method of Static Tests of Timber in Structural Sizes", Figure 2. The load P was measured using a MTS 55 kip (245 kN) actuator and load cell and recorded using a Keithley data acquisition system. The deflection was measured using a direct current displacement transducer (DCDT). The rotational strains due to flexure determined by mounting three DCDTs longitudinally on one side of the beam in the constant moment section of the beam. The load, P, was used to calculate the moment. The moment, rotational strains, and deflection were recorded for each beam.

In addition to the flexural tests, a series of ASTM D 905 - 86 ".Standard Test Method for the Strength Properties of Adhesive Bonds in Shear by Compression Loading," were conducted to determine the bond strength of the FRP-wood interface. Shear block specimens were constructed by gluing wood blocks together using different adhesives. Additionally, shear block specimens contained either non-stressed or prestressed Kevlar. Results from the shear research determined the limiting level of initial prestress that can be applied so that the Kevlar-wood bond will not fail due to the shear from the prestressing force.

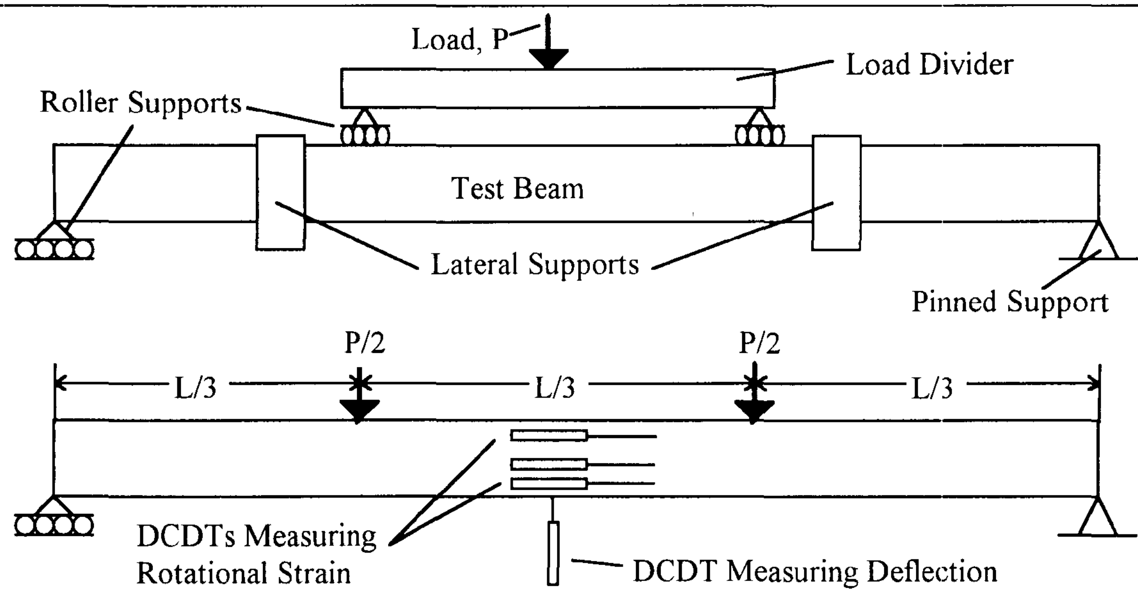

Figure 2. ASTM D 198 -84 Flexure Test for Timber in Structural Sizes

Expected Products

This knowledge will lead to development of prestressed Kevlar reinforced glulam beams which may be used to fabricate bridge or building girders. In addition, this method may also be used to rehabilitate existing structures. The area fraction of reinforcement could be designed to increase the strength, stiffness, ductility, and reliability of such girders. In addition, the initial prestress can be used to create a moment which partially or completely counteracts the moment due to applied loads. The initial prestress may also be designed to create an initial camber that compensates dead load deflections and helps control long-term deflections.

Preliminary Test Results

Experimental results from the ASTM D 198 flexure test did not demonstrate the full predicted strength gain. The ultimate moment capacity of the prestressed beams without tension laminations (P1/T0 and P2/T0) is much lower than predicted. On the other hand, the reinforced beams (R1/T0) exhibited a significant increase in bending strength compared to the anticipated strength gain. The prestressed beams with number 1 grade SP also exhibited larger strength gains than anticipated, Table 1.

Table 1. Strength increases for reinforced beams.

Beam Type	Average Strength Gain (%) (From Flexure Test)	Predicted Strength Gain (%) (Based on FPL mean values for SP lumber)
P1/T0	1.3	20
P2/T0	16.3	32
R1/T0	22.9	3
P1/T2	24.3	15

One explanation for the deviation of experimental results from the theoretical predictions is the influence of finger joints and knots. Every beam failed within the elastic region due to a tension failure of the bottom laminations. Results indicate that 15 of 28 test beams failed at a finger joint on the bottom lamination, 5 of the 28 test beams failed at a knot on the bottom lamination, and 8 of the 28 test beams failed on the bottom lamination due to wood tension failure. Therefore, the comparative tensile strength of the wood, knots, and finger joints are being examined in current research.

Results from the bond-shear tests are shown in Table 2. ASTM D 905 shear tests reveal that the shear strength of the Kevlar-wood interface decreases with increasing pretension force in the Kevlar. Shear block specimens with Kevlar tape glued between them show a higher shear stress than shear block specimens without Kevlar, if the pretension is low (0-1680 lb.). However, as the pretension in the Kevlar ribbon increases (4500-11750 lb.) the average bond-shear strength is lower than the shear tests without Kevlar.

Table 2. ASTM D 905 Shear stress tests with Kevlar

Number Tested	Adhesive	Hardener	Kevlar Pretension (lb)	Kevlar Pretension (kN)	Average Stress (psi)	Shear (MPa)
18	Indspec R600	H30M	No Kevlar	No Kevlar	693	4.8
20	Borden LT-75	FM 260	No Kevlar	No Kevlar	536	3.7
5	Indspec R600	H30M	0	0	1131	7.8
5	Borden LT-75	FM 260	0	0	1232	8.5
10	Indspec R600	H30M	1680	7.5	816	5.6
10	Borden LT-75	FM 260	1440	6.4	818	5.6
9	Borden RS 240 MD	FM 124 D	1680	7.5	904	6.2
8	Indspec R600	H30M	4500	20.0	414	2.9
9	Indspec R600	H30M	11750	52.3	327	2.3

Acknowledgments

This research is sponsored by the Forest Products Research Laboratory, Madison, WI. The conclusions are those of the authors and do not necessarily represent the Forest Products Research Laboratory.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Performance of Timber Bridge Railing Under Vehicle Impact

Author(s) and Affiliation(s):

R.J. Wollyung, Andrew Scanlon, M. Carpino, Brian J. Gilmore
Pennsylvania Transportation Institute
The Pennsylvania State University, University Park, PA

Principal Investigator: Brian J. Gilmore

Sponsor(s): Lichenstein and Associates and the New Jersey Department of Transportation

Research Start Date: July 1995
Expected Completion Date: August 1996

Research Objectives:

This project is investigating the use of timber bridge rails applied to existing timber bridges. The motivation for the project is to provide a bridge rail that is aesthetically pleasing and fits in with the historical ambiance of the locale. The bridges are timber-bent-trestle type construction. The objective of the project is to evaluate proposed timber guardrail designs in terms of their performance under vehicle impact. Prior to conducting full scale crash tests, preliminary evaluations are being conducted using computer simulation, static testing, and impact pendulum testing.

Expected Products or Deliverables:

The primary outcome of this project will be the validation of the design of a timber guardrail for a particular application. The project will also provide useful information on the use of test data to provide input for computer simulations and correlation between computer simulation and full scale crash testing for timber guardrails.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Robert J. Wollyung is a Research Associate at The Pennsylvania Transportation Institute of The Pennsylvania State University involved in research related to vehicle crash testing.

Andrew Scanlon is Professor of Civil Engineering at The Pennsylvania State University and Director of the Transportation Structures Program at the Pennsylvania Transportation Institute with research interests related to bridge engineering.

Marc Carpino is Associate Professor of Mechanical Engineering at The Pennsylvania State University. His research interests include behavior of mechanical systems.

Brian J. Gilmore is Associate Professor of Mechanical Engineering and Director for the Vehicle Crash Safety Center at The Pennsylvania State University. His research interests focus on performance of vehicles in crash tests.

Principal Investigator's Telephone Number:	(814) 863-0736
Facsimile Number:	(814) 865-3039
E-mail Address:	bjg5@psu.edu

Performance of Timber Bridge Railing under Vehicle Impact

R. J. Wollyung, A. Scanlon, M. Carpino, B.J. Gilmore

Pennsylvania Transportation Institute
The Pennsylvania State University
University Park, PA 16802

Research Objectives

The Pennsylvania Transportation Institute is presently investigating the use of timber bridgerails applied to existing timber bridges. The motivation of the project is to provide a bridgerail that is aesthetically pleasing and fits in with the historical ambiance of the locale. The bridges are a timber-bent-trestle type of construction. The objective of the project is to evaluate proposed timber guard rail designs in terms of their performance under vehicle impact. Prior to conducting full scale crash tests, preliminary evaluations are being conducted using computer simulation, static testing, and impact pendulum testing.

Research Approach

Computer Simulation: Barrier VII is a two dimensional, limited finite-element based, vehicle impact simulation program used in the evaluation of roadside safety features. The program was developed at University of California, Berkeley in the early seventies by G. H. Powell for the Federal Highway Administration (1970, 1973). Over the past two decades the program has been used to analyze and develop roadside barriers, mainly guardrails, guardrail-bridgerail transitions, and crash attenuation systems. While the FORTRAN program is simple compared to current workstation-level finite element programs, the simulations have been proven to be very effective in determining barrier deflections, vehicle exit velocities, and impact forces (Tuan, 1989). The following paragraphs briefly describe Barrier VII's methods and limitations.

The impacting vehicle and barrier are modeled through node points defined by X and Y coordinates, in the case of the vehicle, a second set of coordinates based on a vehicle-fixed coordinate system is also used. Vehicle structural stiffness is defined by these points, the nodes function as a deformable geometric boundary as well as dual-stiffness springs for modeling crush. Various barrier elements are used to construct the target structure, these include beams, posts, cables, viscous and friction dampers, pinned links, simple hinges and yielding hinges. Posts and beams will be discussed with greater detail later in this section. These elements are used to connect the nodes, posts are used to connect the nodes to the ground. The geometry is limited to two dimensions.

The simulation is processed as a dynamic, inelastic, large displacement structural analysis problem in two dimensions, which is solved using a step-by-step method. During impact the automobile slides along the barrier. Forces between the automobile tires and the pavement are taken into account as well as the interaction forces between the automobile and barrier.

- Loads are applied to the barrier only at the nodes.

- Masses specified for the barrier are lumped at the nodes.

- Bilinear elastic-plastic behavior is assumed for all yielding members.

A dynamic step-by-step analysis is carried out, in which linear structural behavior of the barrier and automobile is assumed within any time step. The conditions at the beginning of any step are known, including the following:

a) The positions, velocities and accelerations of all points on the automobile and barrier;

b) The magnitudes and directions of the normal and friction forces exerted between the automobile and the barrier, and the positions at which they act; and

c) The axial forces and bending moments in the barrier members.

This data is produced as output by the program at specified intervals. The data can then be used to generate plots of vehicle position, barrier deflection and forces, and vehicle dynamics. However, as indicated above Barrier VII was developed primarily for steel structures.

Two major problems occur when directly applying Barrier VII to a timber post and rail system installed on a bent-pile, timber trestle bridge. First Barrier VII was produced to model materials with elastic-plastic behavior such as steel. While wood such as Douglas Fir has considerable strength, its modes of failure do not resemble a homogeneous elastic-plastic material. Secondly, the structures modeled by Barrier VII are by the most part fixed to compacted earth, bituminous pavement or concrete, not to a structure assembled of components with their own stiffnesses and strengths. Some of these complexities can be compensated for by adjusting the barrier element's parameters and through evaluation of the simulation output. Other complexities and questions must be left for experimental analysis and full-scale crash testing. Barrier VII is an extremely useful but limited analysis tool for evaluating a design. Issues such as three-dimensional vehicle dynamics, strength of mechanical connections, intrusions into the passenger compartment and other unforeseen risks must be evaluated through experimentation.

Barrier VII has been frequently been used to model wooden posts. Accurate knowledge of a post's sheer strength and stiffness is essential to successful models. This information is readily available in design handbooks and through experimentation [Michie, et al, 1971]. It should be noted that most testing is performed with the post implanted in soil and not fixed to a structure. This data is also available for wooden beams as well. The main problem concerns yield and failure criteria used by Barrier VII. Yielding for steel members, means entrance into the plastic range of behavior. The member can bend or extend while the load remains constant at the yield limit. For wooden members this is not the case, once reaching the "bursting limit," the member fractures no longer supporting loads or providing structural continuity. Beam element strength is specified as a yield limit, therefore any yielding detected in wooden rails must be treated as a failure. Post elements have a shear force limit that when reached, the post is considered completely failed. This property should be used as the limiting strength in wooden post elements.

The second issue in this application of Barrier VII relates to the stiffness of the structure. Post stiffness can easily be obtained from direct testing or published literature. However when a guardrail for a wooden bridge is loaded, the substructure of the bridge responds to the loading as well. Procedures for modeling the overall system compliance are being investigated.

Static and Dynamic Tests: Static loading tests of a section of guardrail and deck will be conducted to determine strength and stiffness properties of the assembly for use in computer simulations. In addition dynamic testing using a large-scale pendulum will be conducted to provide information on performance under dynamic loading. This information will be used to refine the modeling of the structure for computer simulation.

Full-Scale Crash Testing: After the final design of the guardrail has been determined based on the results of computer simulation, static testing, and impact testing, a full-scale crash test will be conducted in accordance with AASHTO PL-2 and NCHRP 350 level 4 test criteria.

Expected Products

The primary outcome of this project will be the validation of the design of a timber guardrail for a particular application. The project will also provide useful information on the use of test data to provide input for computer simulations and correlation between computer simulation and full scale crash testing for timber guardrails.

Preliminary Results

Preliminary simulations have been conducted for the following conditions

- 100 feet of continuous rail was modeled. This was done to eliminate end effects and determine how far significant dynamic loads are distributed within the structure.

- 21 posts were used at 5 foot intervals.

- 69 nodes defined 223 elements. Rail elements were 2 feet in length outside the impact area and 1 foot in length close to the impact point.

- Per NCHRP 350 test 4-11 which corresponds to AASHTO PL-2, a 4500 lb vehicle was simulated to crash into a central post of the barrier at a 25 degree angle at 62 mph.

A typical resultant force applied to the barrier posts vs time history is shown in Fig.1.

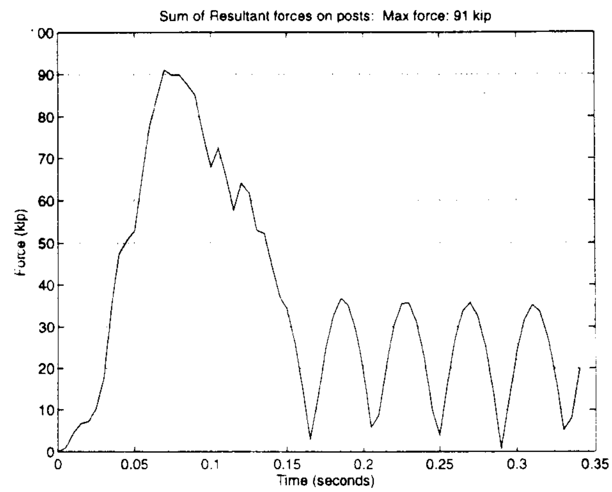

Figure 1. Example of total resultant force applied to the barrier during simulation

References

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FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: An Experimental Study of Flat and Tapered Elastomeric Bridge Bearings With Design Recommendations

Author(s) and Affiliation(s):

Joseph V. Muscarella, Department of Civil and Mechanical Engineering
United States Military Academy at West Point
Joseph A. Yura, Department of Civil Engineering
University of Texas at Austin, Austin, TX

Principal Investigator: Joseph A. Yura

Sponsor(s): Texas Department of Transportation

Research Start Date: September 1, 1991
Expected Completion Date: August 30, 1996

Research Objectives:

The purpose of this study is to analyze elastomeric bridge bearing performance on the basis of elastomer hardness, shape factor, reinforcing shim orientation, degree of taper, and compressive stress level with the goal of developing a simple design procedure which standardizes as many of these parameters as possible. Particular emphasis is placed on comparing the behavior of flat and tapered bearings. Experiments includes shear, compressive, and rotational stiffness tests, shear and compression fatigue loading, long-term compressive loading, and tests to determine compressive stress limits. In many cases, bearings are intentionally loaded non-uniformly to define safe limits for bearing-girder slope mismatches.

Expected Products or Deliverables:

Based upon experimental results and a finite element study, a simplified, standard design procedure will be recommended. Additionally, changes to the 1992 AASHTO Specification will be recommended which will allow for greater flexibility in the use of elastomeric bearings, especially with regard to the employment of tapered pads.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Joseph V. Muscarella is a Lieutenant Colonel in the United States Army Corps of Engineers and an instructor in the Department of Civil and Mechanical Engineering, United States Military Academy at West Point. He received his BS from the US Military Academy in 1978 and his MS and Ph.D. degrees in structural engineering from the University of Texas at Austin in 1986 and 1995 respectively. His Ph.D. research was on the behavior of elastomeric bridge bearings. Since being commissioned as an officer in the Corps of Engineers in 1978, he has completed a wide variety of assignments in field construction, project and contract management, and testing and evaluation.

Joseph A. Yura is The Warren S. Bellows Centennial Professor in Civil Engineering, Department of Civil Engineering, University of Texas at Austin. He has conducted extensive research in the fields of structural stability, plastic design, structural connections and offshore structures. He serves on specification and advisory committees for the Structural Stability Research Council, the American Institute of Steel Construction, the Research Council on Structural Connections, and the American Society of Professional Engineers.

Principal Investigator's Telephone Number:	(914) 938-4030
Facsimile Number:	(914) 938-5522
E-mail Address:	ij5317@westpoint-emh2.army.mil

AN EXPERIMENTAL STUDY OF FLAT AND TAPERED ELASOTOMERIC BRIDGE BEARINGS WITH DESIGN RECOMMENDATIONS

Joseph V. Muscarella, P.E.[1] and Joseph A. Yura, P.E.[2]

[1]Department of Civil and Mechanical Engineering, United States Military Academy
[2]Department of Civil Engineering, University of Texas at Austin

Research Objectives

This paper describes an experimental investigation into the behavior of elastomeric bridge bearings sponsored by the Texas Department of Transportation. A particular effort will be made to show that current AASHTO Specifications are, in many cases, overly restrictive and that using tapered elastomeric bearings to account for girder end elevation differences is fully justifiable. Tapered bearings, which have been used by TxDOT for many years and had been recognized by AASHTO, were disallowed in 1992 even though none of the current research includes tests on tapered pads.

Research Approach

Elastomer Hardness/Shear Modulus Tests: Twelve different designs from one manufacturer and 13 different designs from another were tested for comparison of hardness rating (Shore A Durometer points) to shear modulus. A test apparatus shown in Figure 1 was designed and constructed to duplicate the dead weight and daily thermal cycle of the bridge girder. The apparatus could be configured to subject bearings to both uniform and non-uniform loading. Tests were conducted at compressive stresses of 3.85 MPa (550 psi) and 7.69 MPa (1100 psi) and 50% shear strain.

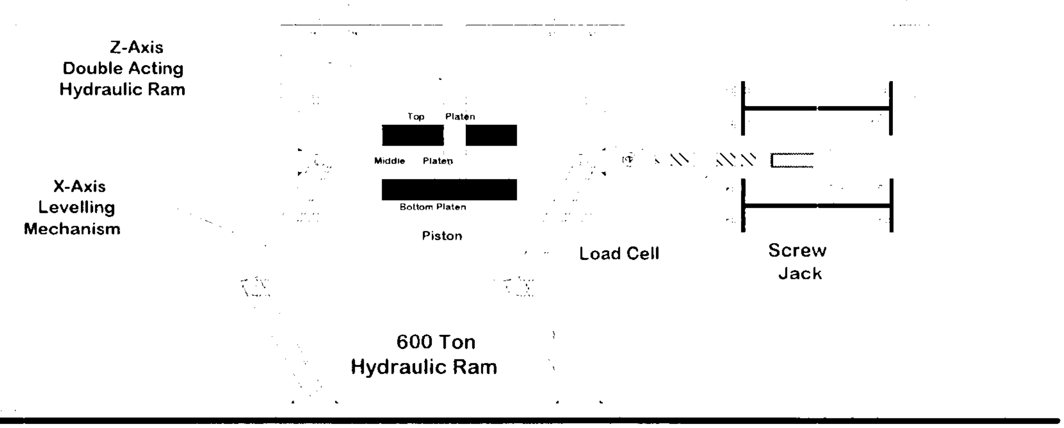

Figure 1 - Schematic of the Bearing Test Setup

Evaluation of shear strains: Figure 2 shows a bearing in direct shear due to girder contraction. The shear strain, γ, (Δ_s divided by total elastomer thickness), the bearing's plan area, A, and the shear modulus, G, determines the shear force transmitted to the girder flange and the abutment. (The AASHTO limit on direct shear strain is 50% of the elastomer thickness):

$$\mathbf{H = \gamma G A} \tag{1}$$

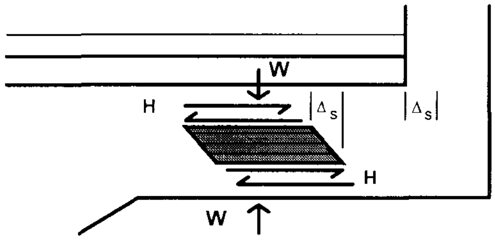

Figure 2 - Elastomeric Bearing in Shear

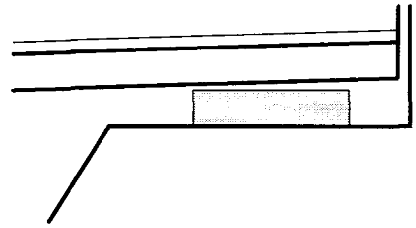
Figure 3 - Non-Uniform Loading

Shear strains also result from compression and rotation. These strains can be controlled by placing limitations upon the bearing's loading. The AASHTO limit on total shear strain is 3.0 (300%) (2):

$$\gamma_s + \gamma_c + \gamma_r \leq \mathbf{3.0} \qquad \mathbf{(2)}$$

Shear strain due to compression is given as (2):

$$\gamma_c = 6\, S\, \varepsilon_c \qquad \mathbf{(3)}$$

where S is the shape factor, the ratio of loaded plan area to area free to bulge. Shear strain from rotation (where C_r is depends on the bearing length to width ratio) is given as (2):

$$\gamma_r = C_r S\theta \frac{L}{2h_{rt}} \qquad \mathbf{(4)}$$

Non-Uniform Loading Tests (Figure 3): The effects a girder/bearing slope mismatch were studied in shear modulus, compression fatigue and compression failure tests.
Compressive Modulus Tests: Compressive stiffness was determined from load-displacement tests and compared with calculations using Equation 5 where 'k' is a factor based on the elastomer (3):

$$E_c = 3G\,(1 + 2\,k\,S^2) \qquad \mathbf{(5)}$$

To limit compressive deformations a harder elastomer may be used or the shape factor may be increased by bonding into the elastomer reinforcing shims of steel or other axially stiff material.
Number of reinforcing shims (Shape Factor): - The influence of shape factor on all aspects of the bearings' performance (stiffness, fatigue, etc.) was studied. AASHTO dictates an increase in the bearing's shape factor at higher compressive stress to control shear stress due to compression (3).
Rotational Stiffness/Rotation Capacity Tests - Rotation tests were conducted to evaluate the bearing's ability to accommodate girder rotations (Figure 4). Tests yielded a moment-rotation relationship and allowed observation of lift-off (rotation capacity exceeded). The bottom corners of the bearing's less compressed edge were observed for upward movement (uplift) during rotation. AASHTO limits the girder rotation that a bearing may accommodate to prevent "lift-off".

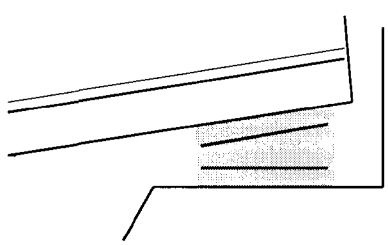
Figure 4 - Rotation Capacity

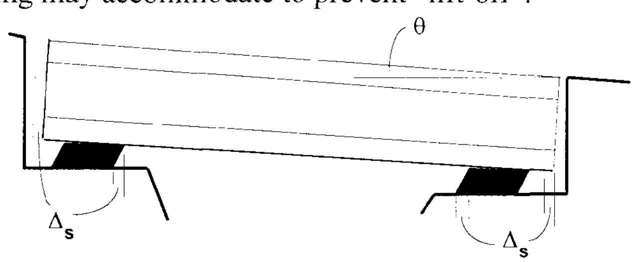

Figure 5 - Deflection Due to Girder Dead Weight

Maximum Taper: When elastomeric bearings support sloped girders, horizontal as well as vertical displacements occur causing direct shear strains before any thermal cycle is considered. (Figure 5)
Fatigue Tests: Bearings were subjected to 50% shear strain for 20,000 cycles (a 55-year service life equivalent) in shear tests. In compression fatigue tests bearings were subjected to 500,000 cycles at 1Hz with a mean stress of 6.9 MPa (1000 psi) and a stress range of 6.9 MPa (1000 psi).

Test specimen design and fabrication: - Specimens were ordered from 2 manufacturers at shear moduli of 0.7 and 1.4 MPa (100 and 200 psi). Hardness was not specified. Specimens were

Table 1 - As Delivered Hardness and Shear Moduli as Determined From Testing

Specified Shear Modulus MPa (psi)	Manufacturer 1 Hardness	Manufacturer 1 Shear Mod. MPa (psi)	Manufacturer 2 Hardness	Manufacturer 2 Shear Mod. MPa (psi)
0.7 (100)	57.0	0.604(87.5)	53.9	0.602(87.3)
1.4 (200)	68.7	0.910 (132)	69.5	0.847 (123)

ordered in 229mm (9") lengths and 711 mm (28") widths and sectioned for testing. An elastomer thickness of 44.5 mm (1.75") was specified. Tapered specimen thickness was taken at the bearing's mid-length. Three bearing tapers were studied: flat to establish a "baseline" performance; nominal 4% and 6% tapers. Neither tapering nor number of shims influenced manufacturing precision.

Bearings with 3 and 6 shims were provided. Reinforcing steel was A570, Grade 40. Although AASHTO would have allowed thinner shims, manufacturers recommended 2.66 mm (12 gage) shims to preclude damage in vulcanization. Shims in flat bearings were parallel to one another. Shims in tapered specimens were equally spaced at each point along the length (cross-section). One 3 shim, 6% taper specimen was ordered with parallel shims to study the effect of shim orientation.

Expected Products

Based upon experimental results and a finite element study (1), a simplified, standard design procedure will be recommended to the Texas DOT. Additionally, changes to the 1992 AASHTO Specification will be recommended which will allow for greater flexibility in the use of elastomeric bearings especially with regard to the employment of tapered pads.

Preliminary Results

Displacement under dead load (Figure 5): Tests show unrestrained sloped bearing/girder systems displace horizontally as if acted on by a force of $H=0.4\theta W$ (plus P-Δ effects (2)). The ratio was not influenced by shape factor, shear modulus or girder/bearing slope mismatch. Tapered bearings with parallel shims deflected 30-40% less than comparable bearings with radially spaced shims.

Shear modulus tests, Matched Slope Tests: Doubling compressive stress from 3.85 to 7.69 MPa resulted in an average decrease in shear modulus of 5.75% (compared to 3% in the finite element study(1)). The calculated shear moduli of tapered and comparable flat bearings was within 5%, which is not significant. Increasing shape factor from 6.26 to 11.0 resulted in an overall 6.9% increase in shear modulus compared to 5% in the finite element study (1).

Shear modulus tests, Non-uniform Loading Tests (1.5 to 2.0% mismatch): Figure 6 shows how non-uniform loads cause a gap between the top of the bearing and the girder. Results showed that calculated shear moduli were 6.2% lower than in matched slope tests. Greater compressive stress (Figure 7) increases contact with the girder yielding only a 2% decrease in shear modulus.

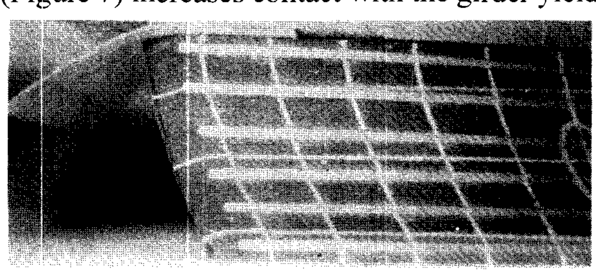

Figure 6 - Bearing at 3.85 MPa (550 psi)

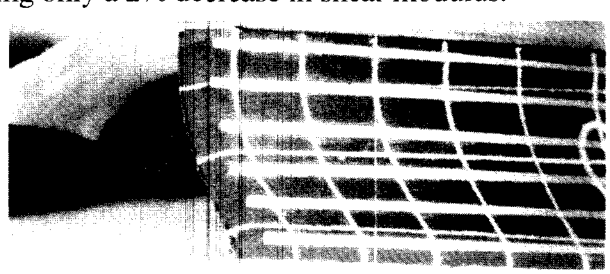

Figure 7 - Same Bearing - 7.69 MPa (1100 psi)

Compressive stiffness/modulus tests: Tests showed that increasing elastomer hardness increased compressive stiffness an average of 25% and increasing the shape factor from 6.26 to 11.0 resulted in a greater than 150% increase. Tapering has only a small effect on the compressive stiffness of the lower shape factor bearings but the effect on bearings with 6 shims is to lower the stiffness by 11.3% over a flat design and increase deflection by 25% due to the entire bearing bulging in the direction of the thick end. Comparison of experimental results with calculations based on Equation 5 shows that Equation 5 is more accurate for shape factors over 5, which includes most all bridge bearings in service. Equation 5 is not very good for predicting actual deformations at a given stress.
Rotational stiffness tests: Doubling the compressive stress increased rotational stiffness by 10.8% and rotation capacity (point where moment-rotation curve became non-linear) by 120%. Increasing elastomer hardness from 54 to 69 durometer increased stiffness by 21.9% and decreased rotation capacity 38.2% for 6 shim bearings, but had less effect on 3 shim bearings. Increasing the shape factor from 6.26 to 11.0 increased rotational stiffness 105% and decreased rotation capacity 60%. Tapered specimens show slightly less rotational stiffness, but greater rotation capacity than flat bearings. There was minimal lift-off under significant rotations. No uplift was ever noted.
Shear Fatigue Tests: Harder bearings showed an average 10.8% decrease in shear stiffness while lower durometer bearings showed a 3.4% decrease. Taper did not appear to influence test results.
Compression Fatigue Tests: Equations 3 and 4 showed that the lower durometer 3 and 6 shim 6% bearings were subjected to maximum shear strains of 5.33 and 3.86 respectively. Specimens with 3 shims averaged 13.5% compressive stiffness loss and showed extensive delamination as early as 150,000 cycles. Six shim specimens lost an average of 3.8% stiffness and showed no delamination at up to 500,000 cycles. Hardness, taper and non-uniform loading did not to influence test results.
Conclusions/Recommendations:
- Published tables can help estimate modulus, but only full-scale tests of reinforced specimens in direct shear under design compressive stress provide an accurate value.
- Based upon the results of static rotation tests and compression fatigue tests, elastomeric bearings can sustain more shear strain without failure than allowed by AASHTO.
- Higher compressive stress reduced the effect of non-uniform loading. Compressive stress was not observed to damage even the most lightly reinforced specimens at up to 34.5 MPa (5000 psi).
- If a total shear strain of 5.0 is allowed as in some foreign codes (4) and lift-off is disregarded, much greater rotations could be allowed.
- A taper of 5.5% (the maximum tested) will result in a great deal of horizontal displacement under dead load alone. The use of a higher shear modulus bearing may be advisable in such geometries.
- Mismatches up to 1% had no detrimental effect upon the bearings' performance.
- Shim orientation does not seem to influence behavior. Analytical studies support this conclusion (1). Parallel shim bearings had less horizontal deflection under dead load and are easier to fabricate.

References

1. Hamzeh, Osama, "Finite Element Analysis of Elastomeric Bridge Bearings," Ph.D. Dissertation, The University of Texas at Austin, August 1995.
2. Roeder, C. W., Stanton, J. F. and Taylor, A. W., "Performance of Elastomeric Bearings," *NCHRP Report #298*, Transportation Research Board, NRC, Washington, D.C., Oct 1987, 100 pp.
3. Standard Specifications for Highway Bridges, 15th Ed., AASHTO, Washington., D.C., 1992.
4. Stanton, J. F. and Roeder, C. W., "Elastomeric Bearings Design, Construction, and Materials," *NCHRP Report #248*, Transportation Research Board, NRC, Washington, D.C., Aug 1982, 82 pp.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
RESEARCH ABSTRACT

Title: Investigation of Two Bridge Alternatives for Low Volume Roads

Author(s) and Affiliation(s):

Terry J. Wipf, F. Wayne Klaiber, Brent M. Phares and Jeff W. Reid
Bridge Engineering Center, Department of Civil and Construction Engineering
Iowa State University, Ames, IA

Principal Investigator: Terry J. Wipf and F. Wayne Klaiber

Sponsor(s): Highway Research Board and the Highway Division of the Iowa Department of Transportation

Research Start Date: June 12, 1995
Expected Completion Date: December 31, 1996

Research Objectives:

Iowa county engineers are responsible for over 20,000 bridges on the secondary road system. As their budgets are limited, many county engineers design and construct short span bridges with their own labor forces. With the assistance of several county engineers, two bridge alternatives have been developed for use on low volume roads. The primary objective of this investigation is to test the alternatives in the laboratory to obtain strength and structural behavior data.

Expected Products or Deliverables:

Assuming successful completion of this investigation, it is envisioned that demonstration projects utilizing the two concepts will be undertaken. Construction of the demonstration bridges would be documented, videotaped, etc., for use in training county crews for such bridge construction.

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
BIOGRAPHICAL INFORMATION

Biographical Sketch(es) of Author(s):

Terry J. Wipf has been a Professor of Civil Engineering at the Iowa State University since 1983. He received his BSCE from the University of Nebraska in 1974; his MSCE and Ph.D. from the same university in 1979 and 1983, respectively. He is author of over 40 papers and research reports and a member of many bridge related technical committees.

F. Wayne Klaiber has been a Professor of Civil Engineering since 1968 at the Iowa State University. He received his BSCE in 1962, his MSCE and Ph.D. in 1964 and 1968, respectively, all from Purdue University. He is author of over 75 papers and research reports and a member of numerous bridge engineering technical committees.

Brent M. Phares is a Ph.D. candidate in Structural Engineering at the Iowa State University. He received his BSCE in 1994 and will receive his MSCE in May, 1996 both from the Iowa State University.

Jeff W. Reid is a Master of Science candidate in Structural Engineering at the Iowa State University. He received his BSCE in 1994 from the Iowa State University.

Principal Investigator's Telephone Number:	(515) 294-6979
Facsimile Number:	(515) 294-8216
E-mail Address:	wipf@iastate.edu

Investigation of Two Bridge Alternatives for Low Volume Roads

T. J. Wipf[1], F. W. Klaiber[1], B. M. Phares[2] and J. W. Reid[2]

Bridge Engineering Center
Dept. of Civil and Construction Engineering
([1]Professor, [2]Graduate Student)
Iowa State University
Ames, Iowa 50011

Research Objective

Iowa county engineers are responsible for over 20,000 bridges on the secondary road system. As their budgets are limited, many county engineers design and construct short span bridges with their own labor forces. With the assistance of several county engineers, the research team has developed two bridge alternatives for use on low volume roads. The primary objective of this investigation is to test the alternatives in the laboratory to obtain strength and structural behavior data.

Research Approach

General: As was previously noted, this investigation involves the laboratory testing of two bridge alternatives which have been developed for use on low volume roads. For clarity, these alternatives are presented separately in the following paragraphs as Concept 1 and Concept 2.

Concept 1: This alternative involves the fabrication of double-T precast units consisting of steel beams (new or used) connected compositely with a concrete deck. The deck thickness is limited so that the units can be easily lifted and transported to the field where several units can be placed side by side to obtain the desired width of bridge. The individual units will then be connected and the remaining portion of the concrete deck placed. Laboratory tests are being performed to determine the following:

- the best procedure for field connecting the individual units.
- the amount and best arrangement of reinforcing steel in the cast-in-place portion of the deck to prevent reflective cracking in the deck.

Several small scale flexural connector tests will be conducted to determine the best procedure for connecting the precast units. In these tests, various types of connections will be investigated; specimens will be tested with one and two layers of concrete to simulate the precast deck and the precast plus cast-in-place deck conditions. Specimens will be instrumented so that strains and deflections at critical locations can be monitored.

Results from the small scale connection tests will be used in the model bridge test which will consist of three double-T units as shown in Fig. 1. This model (Width = 21 ft; Length = 32 ft) will be tested as shown in Fig. 1a with connectors of various stiffnesses and spacings. The model bridge will be instrumented so that strains throughout the bridge and in the connectors can be measured. Deflection instrumentation will also be used to determine the response of the bridge to loads applied at critical locations. Data from these tests will be used to calibrate the finite element model (FEM) currently being developed. With the FEM and laboratory data, it will be possible to determine the best arrangement of connectors for strength and proper load distribution. The model bridge will also be tested with diaphragms at various locations to determine their effect on load distribution. Final tests on the bridge model will include the cast-in-place portion of the deck. The load tests will determine the load distribution and ultimate strength of the system.

Concept 2: This alternative involves the system shown in Fig. 2 that has been used by several Iowa counties as low water stream crossings for over 20 years. There is no reinforcing in this system or physical connection between the steel beams and the concrete. Field tests will be completed on one of these bridges this summer to obtain load distribution data. Laboratory flexural tests will be completed on two beam specimens (Length=30 ft) (cross-section shown in Fig. 3), to obtain bond and ultimate strength data. These specimens will be instrumented to obtain strains, deflections, and slip data.

Laboratory tests will also be performed on modifications of this system. Modification 1 involves drilling holes in the web of the steel beam and placing reinforcing steel through these holes to create composite action between the concrete and steel. This type of shear connector has been investigated by Roberts and Heywood (2) and is similar to Perfobond strips tested by Oguejiofor and Hosain (1). The top flange of the beams may (as shown in Fig. 4a) or may not be removed. Modification 2 involves eliminating concrete from the tension side of the cross-section; one way of doing this is illustrated in Fig. 4b. Individual beams of Modification 1 will be tested as well as a model bridge utilizing Modification 2. Service load data as well as ultimate strength data will be obtained in both the beam and bridge tests. Eleven series of push out tests (see Fig. 5) have been completed on the shear connector between the steel beams and concrete; variables included hole diameter, hole spacing, hole alignment, addition of reinforcing bar through a hole, reinforcement size, and cored vs. torched holes.

Expected Products

Assuming successful completion of this investigation, it is envisioned that demonstration projects utilizing the two concepts could be undertaken. Construction of the demonstration bridges would be documented, videotaped, etc. for use in training county crews for such bridge construction.

Preliminary Results

As previously noted, the 11 series of push-out tests have been completed. Load slip data is shown in Fig. 6 for four of the series tested; a description of these four series is given in Table 1. The curves shown in Fig. 6 provide the reader some information on the effect of adding holes, adding reinforcement through holes, and number of holes.

References

1. Oguejiofor, E. C., and Hosain, M. U. Behavior of Perfobond Rib Shear Connectors in Composite Beams: Full Size Tests. Canadian Journal of Civil Engineering, Vol. 19, No. 2, April 1992, pp. 224-235.

2. Roberts, W. S., and Heywood, R. J. An Innovation to Increase the Competitiveness of Short Span Steel Concrete Composite Bridges. Proc., Developments in Short and Medium Span Bridge Engineering '94, Halifax, Nova Scotia, Canada, 1994, pp. 1161-1171.

Table 1. Push-out Tests

Series	**No. of Specimens**	**Description**
1	6	1 1/4 in. holes on 3 in. centers, in line
7	3	No holes
10	3	1 1/4 in. holes on 3 in. centers, #4 rein. bar in middle raised hole
11	3	One 1 1/4 in. hole with #4 rein. bar

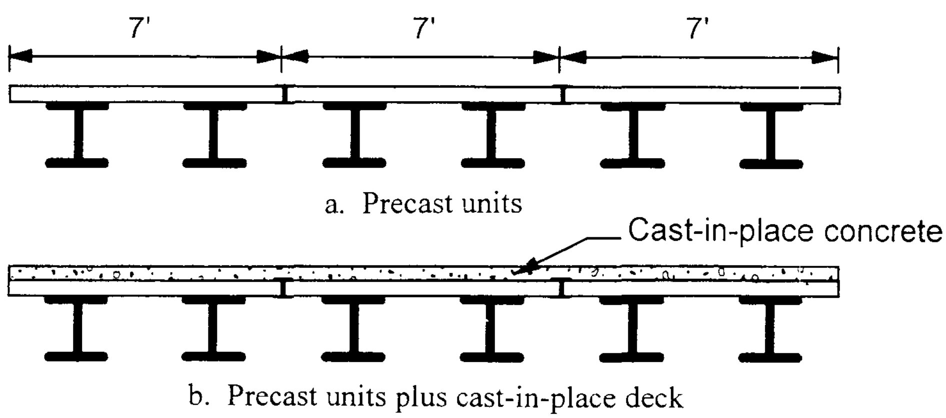

Fig. 1. Cross-section of bridge model - Concept 1.

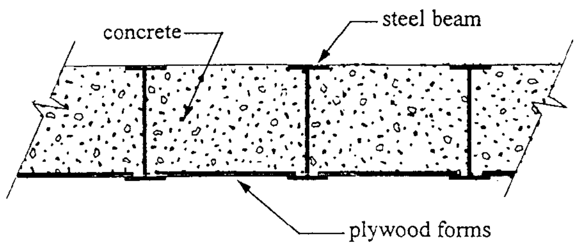

Fig. 2. Low water stream crossing - Concept 2.

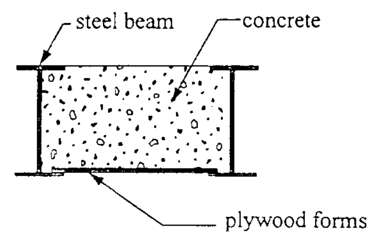

Fig. 3. Beam-cross-section of Concept 2.

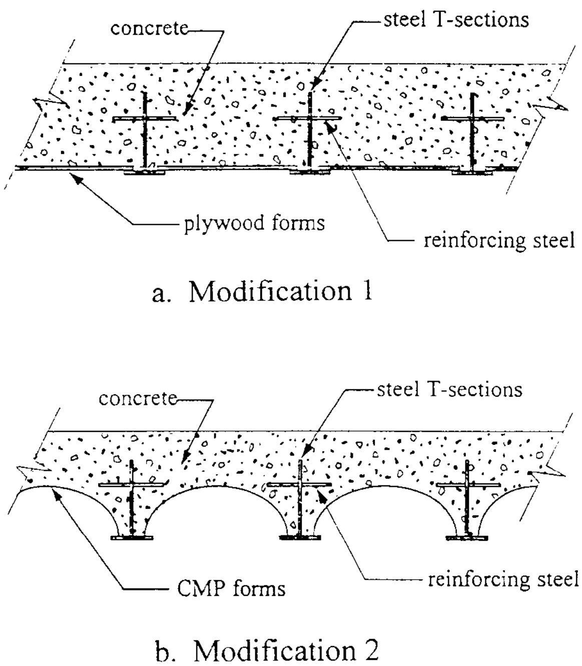

Fig. 4. Modifications to Concept 2.

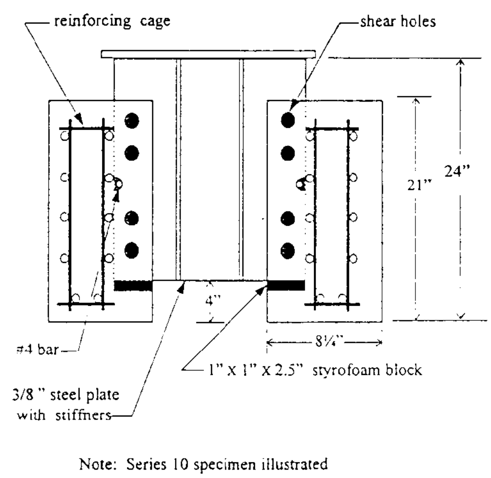

Fig. 5. Push-out specimen.

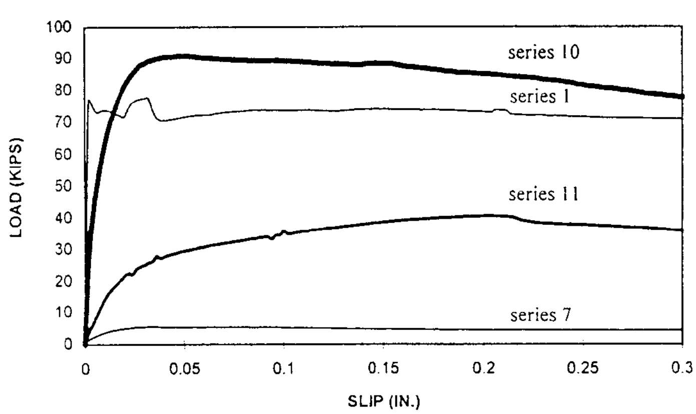

Fig. 6. Load-slip curves of push-out specimens.

POSTER SESSION

POSTER PRESENTATIONS

In addition to the oral presentations of the papers in the preceding section, the workshop also included two poster sessions. The following is the list of poster papers which were confirmed for presentation as of the time these proceedings went to press.

Subject Area - General

"Wheel Load Distribution in a 28' Span Slab Bridge," by Gerald R. Frederick, University of Nevada, Las Vegas

"A Laboratory Study of Elastomeric Bridge Bearing Slip and Friction Characteristics," by Joseph V. Muscarella, United States Military Academy

"Structural Integrity of a Three-sided Arch Span Bridge," by M. Zoghi, the University of Dayton; and Tim Beach, Conspan Bridge Systems

"Nonlinear Finite Element Analysis of Prestressed Concrete Bridge Beams," by William X. Zhang, Vanasse Hangen Brustlin, Inc.; and Richard A. Miller, University of Cincinnati

Subject Area - Earthquake Engineering

"Innovative Ductile Retrofit of Steel Deck-Truss Bridges," by Majid Sarraf and Michel Bruneau, University of Ottawa

"Ductility of Corroded Steel Members and Components," by Sayed M. Zahrai and Michel Bruneau, University of Ottawa

"Foundation Stiffness for Seismic Design of Bridges," by J.P. Singh, Mansour Tabatabaie, Zia Fafir, and Amir Tabatabaie, Geospectra, a Division of Kleinfelder, Inc.

"Noncontact Lap Splices in Bridge Column-Shaft Connections," by Carol Smith and David I. McLean, Washington State University

"Analytical Evaluations of Retrofit Strategies for Multi-column Bridges," by William F. Cofer, David I. McLean, and Yi Zhang, Washington State University

"Sliding Isolation Bearing Characteristics and Implementations," by Ronald J. Watson and Paul Bradford, R.J. Watson, Inc.

Subject Area - Codes and Specifications

"Development of Seismic Design and Evaluation Provisions for Railroad Bridges," by Zolan Prucz, Modjeski and Masters, Inc.; Kenneth L. Wammel, Southern Pacific Transportation Corporation; and James R. Beran, Union Pacific Railroad

"Evaluation of Fatigue Specifications Based on Case Histories and Fracture Mechanics," by George Tsiatas and Shane Palmquist, University of Rhode Island

"Development of Bridge Design Provisions for South Carolina and the Eastern United States," by Charles Lindbergh, Lindbergh & Associates; James E. Roberts, California Department of Transportation; and Rocque L. Kneece, South Carolina Department of Transportation

"A LSD Code for Bridge Substructures," by Roger Green, University of Waterloo

Subject Area - Instrumentation

"Real-time Remote Condition Monitoring of a Cable-stayed Bridge in Bangkok, Thailand," by Robert Nigbor and John Diehl, Agbabian Associates

"GPS as a Bridge Monitoring Sensor: New Developments in Performance, Cost Effectiveness, and System Intelligence," by Michael D. Hyzak, Keith A. Duff, and Mark P. Leach, the University of Texas at Austin

APPENDIX A

Summaries of Research Presented at the Third Workshop on Bridge Research in Progress

INTRODUCTION

This report documents the proceedings of the Fourth National Workshop on Bridge Research in Progress. In this section, authors from the third bridge research workshop, which was held at the University of California, San Diego in 1992, provide follow-up information on the research that was in progress at the time of the third workshop. Specifically, authors were asked to provide the following information: (a) the status of research (completed, still in progress); (b) specific objectives of the study; (c) a brief description of the conclusions or results; (d) an abbreviated list of published papers or presentations resulting from the study; and (e) a description on where and/or how the results of the study may have been implemented into practice.

The following pages summarize the results of some of these studies. Since not all of the authors from the third workshop were able to respond to this request for information, this section provides a sample of completed research rather than an exhaustive list of research accomplishments since 1992. Please note that the "Proceedings Ref." entry for each summary is the page on which the published paper in the proceedings of the third workshop appears (see Preface for full proceedings reference).

THIRD WORKSHOP RESEARCH SUMMARIES

Project:	**Applications of High Strength Concrete to the Prestressed Industry**
Researcher(s):	Mokhtarzadeh, A., Ahlborn, T., French, C., Leon, R.
Proceedings Ref:	Page 23
Status:	To be completed in September 1996. Mechanical properties and durability of high strength concrete phases of the study were completed in January of 1996.
Objectives:	To investigate the mechanical properties and durability of high-strength concrete. Nearly 7000 specimens were tested to investigate compressive strength with time, modulus of elasticity, modulus of rupture, splitting tensile strength, creep, shrinkage, indirect permeability, and freeze-thaw durability of high-strength concrete. Variables included the curing condition, cementitious materials composition, aggregate type, specimen size, etc. In addition, structural tests were conducted on two HSC prestressed bridge girders to obtain information on transfer length, prestress losses, serviceability, fatigue life, ultimate flexural and shear strength.
Conclusions:	It is possible to attain 10,000 psi concrete within 24 hours using conventional materials. The strength of HSC is controlled by the aggregate type and the aggregate type is very dependent on the local geological conditions. The use of microsilica in the mix helps to increase the strength of concrete made with round gravel because it improves the bond between the aggregate in the paste; whereas it is less effective in limestone concrete, in which case the failure occurs in the aggregate. It is possible to use large diameter strand (0.6 in.) on 2 in. centers in high-strength concrete. The concrete strength improves the transfer of forces between the prestressing strand and the concrete. Data is currently being investigated regarding the tests of the HSC girders.

References:
(partial list)

[1] Shield, C., French, C., and Ahlborn, T., "Effects of Initial Shrinkage Cracks on Prestressed Bridge Girders," submitted to ASCE Structural Engineering Journal, January 1996.

[2] Mokhtarzadeh, A., Kriesel, R., French, C. and Snyder, M., "Mechanical Properties and Durability of High-Strength Concrete for Prestressed Bridge Girders," Transportation Research Record, No. 1478, *Materials and Construction: Concrete and Concrete Pavement Construction*, National Academy Press, Washington, D.C., 1995, pps. 20-29.

[3] Ahlborn, T., French, C. and Leon, R., "Applications of High-Strength Concrete to Long-Span Prestressed Bridge Girders," Transportation Research Record, No. 1476, *Bridges, Other Structures, and Hydraulics and Hydrology: Steel, Concrete, and Wood Bridges*, National Academy Press, Washington, D.C., 1995, pp. 22-30.

Implementation: Based on the materials study, high strength concrete has been implemented in the bridge girder industry in the State of Minnesota. The results of these tests complement other studies underway at the FHWA and in Texas.

THIRD WORKSHOP RESEARCH SUMMARIES

Project:	**Bond Strength and Durability of Coated and Uncoated Rebar**
Researcher(s):	Grundhoffer, T., Lorentz, T., French, C., Leon, R.
Proceedings Ref:	Page 31
Status:	Completed March 1992

Objectives:

This project was divided into two studies: (1) the bond strength of coated and uncoated reinforcement in concrete and (2) the durability of coated and uncoated reinforcement in concrete. Variables included bar size and concrete compressive strength. The objectives of the second project were to investigate the durability of coated and uncoated reinforcement in concrete. Accelerated corrosion tests were conducted on coated, uncoated, coated with damage, coated with grit, reinforcing bars in concrete prisms. Additional variables investigated were the effects of the presence of silica fume on the rate of corrosion, and the effect of cracking in slab specimens.

Conclusions:

For the first study, it was concluded that epoxy coatings reduce bond strength due to lack of adhesion and reduction in effective bearing area. The strain distribution in epoxy-coated bars is spread over a much greater length. The addition of micro silica improved the bond strength. In the corrosion study, it was determined that the addition of micro silica improved the corrosion resistance; however, its effectiveness was reduced with increased percentages of microsilica. This indicated that there may be an optimum amount of microsilica for durability. With epoxy coating on both mats of reinforcement (top and bottom) the macrocell corrosion rate was significantly reduced even for epoxy bars which had significant imposed holidays. Cracking in concrete slabs greatly affected the corrosion activity; large amounts of corrosion deposits were observed on the reinforcement in the vicinity of the cracks.

References:

[1] Lorentz, T. and French, C., "Material Effects on the Corrosion of Reinforcing Steel in Concrete," ACI Materials Journal, Vol. 92, No. 2, March-April 1995, pp. 181-190.

[2] Grundhoffer, T., Lorentz, T., French, C. and Leon, R., "Bond Strength and Durability of Coated and Uncoated Rebar," Proceedings, NSF Bridge Research in Progress, San Diego, CA, November 16-17, 1992.

[3] Leon, R., Grundhoffer, T. and French, C., "Bond of Coated and Uncoated Reinforcement in Normal and High Strength Concrete," Proceedings, International Conference on Bond and Anchorage of Reinforcement in Concrete, Riga, Latvia, October 1992.

Implementation:

This study complemented a field investigation conducted by the Minnesota Department of Transportation on the in-situ performance of epoxy coated bars. Bridges were cored in the vicinity of cracks where it was confirmed that there was little corrosion activity on the epoxy coated bars while the uncoated bars in the vicinity of the cracks exhibited corrosion products. Use of epoxy-coated reinforcement continues in the State of Minnesota.

THIRD WORKSHOP RESEARCH SUMMARIES

Project:	**Concrete Arch Bridges Built Up Without Centering**
Researcher(s):	Wechsler, M.B.
Proceedings Ref:	Page 35
Status:	Continuing in order to determine conditions and possibilities of applying technology to actual loading and construction materials.
Objectives:	To find an economical procedure for building concrete arch bridges without the aid of centerings.
Conclusions:	At this time, there are not sufficient data to focus on conclusions or results.
References:	Submitted to the international design competition "An Image of the Bridge of the Future," sponsored by the planning committee for the Great Seto Bridge Memorial Center, Japan, March 1988.
Implementation:	None at this time.

Project:	**Tensile Pile Test**
Researcher(s):	Mason, J.A.
Proceedings Ref:	Page 67
Status:	Completed December 1992
Objectives:	(1) To determine cohesive strength of San Francisco Bay mud; (2) To introduce alternative pile types to Caltrans; and (3) Compare performance characteristics of various driven pile types with drilled and grouted piles.
Conclusions:	(1) The cohesive resistance of San Francisco Bay mud is now included in the tension capacity calculations in deep foundations founded through those soft clays; (2) Alternative piles are now being used on seismic retrofit projects.
References: *(partial list)*	[1] ASCE Structures Congress '93 - Structural Engineering in Natural Hazards Mitigation, "Tension Pile Test," pp. 385-390. [2] Deep Foundations Institute, (D.F.I.) and Caltrans Specialty Seminar, Sept. 1993; "Caltrans Pile Load Test Results at a Deep Bay Mud Site Using Various Pile Types, I-280/San Francisco Pile Load Test - Results and Seismic Design Guidelines." [3] Fifth International Conference and Exhibition on Piling and Deep Foundations, The Deep Foundations Institute (DFI), June 1994, "Use of High Capacity Piles in Bridge Foundations with Full Scale Test Results."
Implementation:	Caltrans now utilizes "alternative pile types" in bridge projects. The results from these tests are also used by foundation design engineers worldwide, for the design of deep foundation elements founded through soft clays.

THIRD WORKSHOP RESEARCH SUMMARIES

Project:	**Ultimate Load Testing of a 1:3 Scale Segmental Concrete Shell Bridge**
Researcher(s):	Farmer, B., Fanous, F., Klaiber, F.W.
Proceedings Ref:	Page 71
Status:	Completed in 1992
Objectives:	(1) To determine the effects of reducing the dimensions of the diagonal members used to connect the curbs to the shell on the structural behavior of the bridge model. (2) To determine the ultimate strength and the variables that influence the overall behavior of the shell-bridge model.
Conclusions:	(1) Reducing the size of gusset plates and the diagonal members resulted in an increase in the deflection and changed the longitudinal strain distribution. (2) Failure response of the bridge model was non-catastrophic and slowly provided sufficient warning signals. The failure was the result of the radial cracks that were formed in the shell. Near the ultimate load, large particles of concrete began spalling and the gages monitoring the loading system showed a loss of load indicating a loss of the section's structural integrity.
References:	[1] Farmer, B.W., "Ultimate Load Testing of a 1:3 Scale Segmental Concrete Shell Bridge," M.S. Thesis, Iowa State University, 1992. [2] Fanous F., Klaiber, F.W., and Farmer, B.W., "Ultimate Load Test of a 1:3 Scale Shell-Bridge Model," submitted to CSCE, January, 1996.
Implementation:	

THIRD WORKSHOP RESEARCH SUMMARIES

Project:	**Sensitivity of On-Line Modal Property Identification for Bridge Inspection**
Researcher(s):	Alampalli, S., Fu, G.
Proceedings Ref:	Page 79
Status:	Completed December 1995.
Objectives:	Current bridge inspection methods rely largely on visual examination to evaluate bridge condition. Remote Bridge monitoring systems (RBMS) using measured structural vibration are perceived to assist in bridge inspection. Sensitivity of measured modal properties for RBMS is critical for practical application. The purpose of this study was to examine sensitivity of modal parameters in detecting fatigue cracks, including frequencies, damping ratios, mode shapes, and their derivatives.
Conclusions:	(1) Modal frequencies may be used to detect the existence of damage or deterioration of highway bridges. Cross-diagnosis using multiple signatures such as mode shapes, MAC, and COMAC is warranted for such detection, because a single signature may not be conclusive due to the inevitable variation of measured data. (2) Based on a 95-percent confidence, presence of a crack of 6 cm (2.4 in) or longer can be detected using modal frequencies supplemented by MAC, both obtainable using commercially available instrumentation, in a 21.95 m (72 ft) long bridge. However, the crack location may be hard to find using mode shapes and their derivatives. (3) A limited number of modes may be inadequate for damage diagnosis with high confidence, because certain modes may be less sensitive to the damage or may include higher noise, leading to false diagnosis. (4) For complete damage diagnosis including existence and location, improved instrumentation may be needed for less noise and greater simultaneous coverage, with cost-effectiveness also considered.
References:	Alampalli, S., Fu, G., and Dillon, E.W., "Measuring Bridge Vibration for Detection of Structural Damage," Report FHWA/NY/RR-95/165, Transportation Research and Development Bureau, New York State Department of Transportation, December 1995.
Implementation:	

THIRD WORKSHOP RESEARCH SUMMARIES

Project:	**Residual Deformation Analysis for Bridges**
Researcher(s):	Dishongh, B.E.
Proceedings Ref:	Page 93
Status:	Redisual Deformation Analysis (RDA), as presented in La Jolla, has been refined into the shakedown analysis method known as Inelastic Moment Redistribution (IMR).
Objectives:	IMR is intended to serve as an easy-to-use shakedown analysis method. It will facilitate the work that is on-going with the Autostress research of Dr. Barker under a current NSF/AISC/AISI project.
Conclusions:	IMR is easy to use and aids in visualizing how shakedown occurs. If shakedown specifications are expanded in the LRFD code, IMR is a good method to reference.
References:	[1] "Inelastic Moment Redistribution for Bridges," ASCE Structures Journal, Feb. 1995. [2] "Shakedown Design" presentation to Steel Bridge Committee of TRB, Jan. 1995. [3] "Autostress Design", Seminar for Louisianna Department of Transportation and Development (La DOTD) bridge engineers, August, 1994.
Implementation:	La DOTD is using autostress to design La531 over I-10 overpass (130' - 130' continuous rolled beams). Autostress design uses 7 - W40 x 297 beams instead of 9 - w 40s by LFD.

THIRD WORKSHOP RESEARCH SUMMARIES

Project:	**The Relationship Between Geometric Form and Aeroelastic Characteristics of Bridge Decks**
Researcher(s):	Jones, N.P., Scanlan, R.H., Raggett, J.D.
Proceedings Ref:	Page 101
Status:	This paper represents a component of a broader research effort on the performance of long-span bridges in wind, which is continuing.
Objectives:	(1) To improve the understanding of interaction of wind flow with the complex cross sections of long-span bridges; (2) To improve modeling capability for long-span bridges in wind; (3) To acquire a better understanding and modeling of the structural dynamics of cable-supported bridges; and (4) To develop improved design concepts for large bridges.
Conclusions:	(1) Details of cross section (particularly upper leading edge) can strongly affect performance of structure in wind; (2) Aerodynamic and aeroelastic coupling among the modes of large bridges need to be considered; (3) The cable interactions with the bridge structure influence the structural dynamics and the wind performance; and (4) Interactions between decks in twin-deck configurations can be aerodynamically significant, and favorable.
References: *(partial list)*	[1] Sarkar, P.P., Jones, N.P., and Scanlan, R.H., "A Recursive Time-Domain System -Identification Procedure for Extraction of Aeroelastic Parameters," *Journal of Engineering Mechanics*, 120(8), ASCE, August, 1994, 1718-1742. [2] Jones, N.P., Scanlan, R.H., Sarkar, P.P., and Singh, L. (1995), "The Effect of Section Model Details on Aeroelastic Parameters," *Journal of Wind Engineering and Industrial Aerodynamics*, 54/55, pp. 45-53. [3] Jain, A., Jones, N.P., and Scanlan, R.H., "Coupled Flutter and Buffeting Analysis of Long-Span Bridges," accepted, *Journal of Structural Engineering*, ASCE, October, 1995.
Implementation:	Research in this program has been applied to the following structures, which are built or under construction: (1) Golden Gate Bridge retrofit; (2) Houston Ship Channel Bridge; (3) Port de Normandie, France; and (4) Kap Shui Mur Bridge, Hong Kong.

THIRD WORKSHOP RESEARCH SUMMARIES

Project:	**Transverse Load Distribution Factors for Permit Vehicles**
Researcher(s):	Puckett, J.A., Finch, T.R.
Proceedings Ref:	Page 105
Status:	Completed in 1993
Objectives:	To provide an automated procedure for the analysis of slab-on-girder bridges in order to determine distribution factors.
Conclusions:	Research resulted in a computer application that is simple to use and readily available. The program determines distribution factors along the span in each girder for shear and moment. The loading may be standard or very general. The program is distributed by the Wyoming Department of Transportation as part of their BRASS programs. The validation of the program included comparing the finite analysis results from NCHRP Project 12-26 which included over 300 bridges. The results were also used to compare to the LRFD distribution factors that were developed as part of the 12-26 project.
References:	The report is available from the Wyoming Department of Transportation.
Implementation:	The software is distributed through the Wyoming Department of Transportation (contact Mr. Watters at 307-777-4427).

Project:	**Calibration of LRFD-Based Highway Bridge Design Code**
Researcher(s):	Nowak, A.S.
Proceedings Ref:	Page 131
Status:	Completed in 1994
Objectives:	To develop load and resistance factors for the new probability-based AASHTO LRFD code.
Conclusions:	The new load and resistance factors were developed on the basis of state-of-the-art statistical data. They provide a good basis for rational design.
References:	[1] Nowak, A.S., 1995, "Calibration of LRFD Bridge Code," ASCE Journal of Structural Engineering, Vol. 121, No. 8, pp. 1245-1251. [2] Nowak, A.S., Yamani, A.S. and Tabsh, S.W., 1994, "Probabilistic Models for Resistance of Concrete Bridge Girders," ACI Structural Journal, Vol. 91, No. 3, pp. 269-276. [3] Nowak, A.S., 1993, "Love Load Model for Highway Bridges," Journal of Structural Safety, Vol. 13, Nos. 1+2, December, pp. 53-66.
Implementation:	The work was implemented in the new AASHTO LRFD code which was published in 1994.

THIRD WORKSHOP RESEARCH SUMMARIES

Project:	**Evaluation, Rehabilitation and Strengthening Manual for Low Volume Bridges**
Researcher(s):	LaViolette, M.D., Wipf, T.J., Klaiber, F.W., Besser, D.M.
Proceedings Ref:	Page 135
Status:	Completed February 1993
Objectives:	To develop a manual to assist county engineers in making cost-effective bridge strengthening or replacement decisions.
Conclusions:	(1) Steel stringer and timber stringer bridges have the greatest potential for cost-effective strengthening of all the bridge types on the secondary road system in Iowa; (2) Numerous procedures for strengthening and replacement have been provided, along with design aids.
References:	Wipf, T.J., Klaiber, F.W., Besser, D.M. and LaViolette, M.D., "Manual for Evaluation, Rehabilitation and Strengthening of Low Volume Bridges," Iowa DOT Project HR-323, Final Report ISU-ERI-Ames-93062, February 1993.
Implementation:	Iowa county engineers each received a design manual and several have used the results in choosing and evaluating bridge strengthening and replacement alternatives.

THIRD WORKSHOP RESEARCH SUMMARIES

Project:	**Reliability-Based Redundancy Measures for Bridge Evaluation and Design**
Researcher(s):	Frangopol, D.M., Iizuka, M.
Proceedings Ref:	Page 139
Status:	Completed in 1993
Objectives:	To develop system redundancy measures for bridge evaluation and design.
Conclusions:	(1) Several deterministic and probabilistic system redundancy measures were proposed and compared considering their reliability, generality, and ease of use; (2) Reliability-based definitions and criteria to incorporate bridge redundancy in bridge evaluation and design programs are now available.
References: *(partial list)*	[1] Hendawi, S., and Frangopol, D.M. (1995), "System Reliability and Redundancy in Structural Design and Evaluation," *Structural Safety*, Elsevier, Vol. 16, pp. 47-71. [2] Frangopol, D.M., Iizuka, M., and Yoshida, K. (1992), "Redundancy Measures for Design and Evaluation of Structural Systems," ASME Transactions, *J. of Offshore Mechanics and Arctic Engineering*, New York, Vol. 114, No. 4, pp. 285-290. [3] Frangopol, D.M., and Nakib, R. (1991). "Redundancy in Highway Bridges," *Engineering Journal*, AISC, 1st Quarter, Vol. 28, No. 1, 45-50.
Implementation:	The new AASHTO LRFD Bridge Design Specifications, First Edition, 1994, introduces redundancy factors and refers explicitly to the paper "Redundancy in Highway Bridges," Engineering Journal, AISC, 1st Quarter, Vol. 28, No. 1, pp. 45-50, by Frangopol and Nakib.

THIRD WORKSHOP RESEARCH SUMMARIES

Project:	**Bridge Nondestructive Evaluation by Structure Identification: Description of the Methodology**
Researcher(s):	Aktan, A.E., Chuntavan, C., Toksoy, T., Lee, K.L.
Proceedings Ref:	Page 143
Status:	Completed in 1993.
Objectives:	To classify existing NDE methodologies for constructed facilities. Develop quantitative NDE methodology for addressing the global health and reliability of a constructed facility.
Conclusions:	Modal testing is the principal experimental component of the NDE method. Multireference impact testing can serve as the main experimental component for comprehensive structural identification of large facilities. For earthquake and lateral overload vulnerability evaluation, forced-excitation modal testing may be required.
References:	[1] Aktan, A.E., Zwick, M., Miller, R., and Shahrooz, B. (1993). "Nondestructive and destructive testing of decommissioned reinforced concrete slab highway bridge and associated analytical studies." *Transportation Research Record*, 1371, TRB, National Research Council, 142-153. [2] Aktan, A.E., Chuntavan, C., Toksoy, T., and Lee, K.L. (1993). "Structural identification of a steel-stringer bridge for nondestructive evaluation." *Transportation Research Record*, 1393, TRB, National Research Council, 175-185.
Implementation:	Nondestructive and destructive evaluation tests on Ohio Bridge CLE-222-2622, a continuous three-span, skewed, reinforced concrete bridge constructed in 1953. Nondestructive evaluation and rating of Bridge HAM-42-0992, a continuous three-span, skewed, reinforced concrete slab on steel stringers bridge constructed in 1990.

THIRD WORKSHOP RESEARCH SUMMARIES

Project:	**Strengthening of Continuous Span Steel Stringer Bridges with Superimposed Trusses**
Researcher(s):	Klaiber, F.W., Wipf, T.J., Fanous, F., El-Arabaty, H.A.
Proceedings Ref:	Page 151
Status:	Completed September 1993.
Objectives:	To investigate the field use of superimposed trusses in combination with post-tensioning for bridge strengthening and to develop a strengthening manual that practicing engineers can use in the design of a strengthening system for a given bridge.
Conclusions:	Superimposed trusses can be used with post-tensioning to strengthen continuous span bridges. Significantly less post-tensioning forces are required in the cases where trusses are employed. Spreadsheets were developed for use in the design procedure that was developed.
References:	[1] Wipf, T.J., Klaiber, F.W., Fanous, F.S., and El-Arabaty, H., "Strengthening of Continuous-Span Composite Steel-Stringer Bridges," Fourth International TRB Bridge Engineering Conference, Vol. 2, Aug. 1995, pp. 33-44. [2] Klaiber, F.W., Wipf, T.J., Fanous, F.S., Bosch, T.E., and El-Arabaty,H., "Strengthening of an Existing Continuous-Span Steel-Stringer, Concrete-Deck Bridge," Final Report 94402, ISU, Ames, Iowa, Sept. 1993. [3] Klaiber, F.W., Fanous, F.S., Wipf, T.J., and El-Arabaty, H., "Design Manual for Strengthening of Continuous-Span Composite Bridges," Final Report 94404, ISU, Ames, Iowa, Sept. 1993.
Implementation:	To date the only bridge where trusses have been used in combination with post-tensioning is the demonstration bridge of this project.

THIRD WORKSHOP RESEARCH SUMMARIES

Project: **Bridge Nondestructive Evaluation by Structural Identification: Applications**

Researcher(s): Aktan, A.E., Toksoy, T., Chuntavan, C., Lee, K.L.

Proceedings Ref: Page 155

Status: Completed in 1993.

Objectives: Application of nondestructive and destructive evaluation tests to address the global health, damage detection, and reliability of Ohio Bridge CLE-222-2622, a continuous three-span, skewed, reinforced concrete bridge constructed in 1953. Also, application of nondestructive evaluation and rating of Bridge HAM-42-0992, a continuous three-span, skewed, reinforced conrete slab on steel stringers bridge constructed in 1990.

Conclusions: Finite element models, calibrated based on modal tests, provide analytical baseline for detection of future deterioration and damage. The sensitivity of modal flexibility to damage is demonstrated relative to frequencies and mode shapes.

References:

[1] Aktan, A.E., Chuntavan, C., Lee, K.L., and Farhey, D.N. (1994). "Nondestructive Testing & Identification for Bridge Rating - Phase 2: Steel-Stringer Bridges," *Report FHWA/OH-95/021, UC-CII 94/02*, Department of Transportation, Infrastructure Institute, University of Cincinnati, 254 pp.

[2] Aktan, A.E., Farhey, D.N., and Dalal, V. (1995). "Issues in Rating Steel-Stringer Bridges." *Transportation Research Record*, No. 1476, TRB, National Research Council, 129-138.

Implementation: Bridges CLE-222-2622 and HAM-42-0992.

Project:	**Effect of Hinge Restrainers on the Response of Highway Bridges During the Loma Prieta Earthquake**
Researcher(s):	Saiidi, M., Maragakis, E., Abdel-Ghaffar, S., O'Connor, D.
Proceedings Ref:	Page 171
Status:	Completed December 1993.
Objectives:	(1) To evaluate the performance of hinge restrainers during the 1989 earthquake in Loma Prieta, California, based on field observations and simulation studies; (2) To evaluate the restrainer design method.
Conclusions:	(1) Restrainers in many of the bridges affected by the Loma Prieta earthquake were activated by the earthquake, but generally performed well. (2) The restrainer connections and bridge diaphragms supporting these connections need to be designed so that they will not form a weak link in the bridge restrainer system. (3) The current Caltrans restrainer design method needs to be improved because the restrainers required based on this method are very sensitive to small variation in the assumed approximate variables.
References:	[1] Saiidi, M., E. Maragakis, and S. Feng, "Parameters in Bridge Restrainer Design for Seismic Retrofit," *Journal of Structural Engineering*, ASCE, Vol. 121, No. 8, January 1996. pp. 61-68. [2] Saiidi, M., and E. Maragakis, "Effectiveness of Hinge Restrainers as a Seismic Retrofit Measure," TRB, Fourth International Conference on Bridge Engineering, Vol. 2, August 1995, pp. 71-78. [3] Saiidi, M., E. Maragakis, and D. O'Connor, "Seismic Performance of the Madrone Bridge During the 1989 Loma Prieta Earthquake," (invited paper), *Structural Engineering Review Journal*, Vol. 7, No. 3, August 1995, pp. 219-230.
Implementation:	Some of the findings are being incorporated in ACI reports on seismic design of bridges, currently under preparation.

THIRD WORKSHOP RESEARCH SUMMARIES

Project:	**Study on the Dynamics of CFRP Cable-Stayed Bridges**
Researcher(s):	Khalifa, M.A., Tadros, M.K., and Hodhod, O.A.
Proceedings Ref:	Page 191
Status:	Theoretical and experimental investigations completed in March 1996.

Objectives:

To investigate the mechanical behavior of PFRP in tension and flexure, and beam tests on PFRP W-shapes. Also examined were stress concentrations around circular and square holes, creep behavior of coupon specimens in bending, and compression behavior of concrete reinforced with fiber reinforced grating.

Conclusions:

The feasibility study showed that FRP can be used to construct a proposed pedestrian cable-stayed bridge. The bridge should have two towers each of them is 24.4 m (80 ft) high above the deck level. The towers are diamond shaped offering aesthetically appealing, high torsional stiffness and ease of handling cable anchorages. The tower cross section is circular with a diameter of 0.9 m (3 ft). Each cable plane contains 40 cables.

The bridge deck may take one of two possible designs. The first is a box girder designed to offer modularity. Changes in deck depth or width can be accommodated easily by adding or removing one or more of the structural panels that constitute the cross section. Although this type of box girder might not be the most suitable aerodynamically, its aerodynamic characteristics can be easily modified by the addition of fairings. A second alternative for the deck is a flat honeycomb plate with or without longitudinal girders. Cross girders can be provided at each cable anchor.

The study showed that the proposed bridge will cost $1.2 million with no expected maintenance charges. A concrete girder bridge will cost $1.1 million (including maintenance cost for 50 years) and a steel-concrete bridge will cost $1.8 million (including maintenance).

References:
(partial list)

[1] Khalifa, M.A., Hodhod, O.A., and Zaki, M., "Analysis and Design Methodology for FRP Bridges," *The International Journal of Composites Engineering*, Oct. 1994. (Approved).

[2] Khalifa, M.A. and Hodhod, O.A., "In Plane Behavior of Concrete Reinforced with Glass Fiber Reinforced Grating," Proceedings of the Second International Conference on Composite Engineering ICCE/2, New Orleans, Louisiana, Aug. 1995.

[3] Hodhod, O.A. and Khalifa, M. A., "Statistical Inference on Strength of Pultruded Glass Fiber Reinforced Plastic," Proceedings of the Second International Conference on Composite Engineering ICCE/2, New Orleans, Louisiana, Aug. 1995.

Implementation:

The Lincoln composite bridge is supposed to be the world's longest composite pedestrian bridge. It is a cable-stayed bridge with a main span of 400 ft and total length of 900 ft. This bridge is proposed to be constructed at the intersection of 27th Street and Nebraska Highway 2 to connect the north and south parts of the Rock Island commuter trail.

THIRD WORKSHOP RESEARCH SUMMARIES

Project:	**University at Buffalo-Taisei Corporation Research Project on Bridge Seismic Isolation Systems**
Researcher(s):	Constantinou, M.C., Fujii, S., Tsopelas, P., Okamoto, S.
Proceedings Ref:	Page 235
Status:	Completed September 1993.
Objectives:	(1) Production and experimental verification of a class of bridge seismic isolation system by modifying and/or adapting existing technology; (2) Experimental study of sliding isolation systems which, while implemented, have not been previously tested on a shake table.
Conclusions:	Isolation systems consisting of sliding bearings, rubber restoring force devices, fluid dampers, or fluid restoring force and damping devices have been configured and tested with a flexible pier bridge model. Moreover, elastoplastic systems and various configurations of the FPS system were tested. Most of the tested systems developed substructure force at less than 1/3 of weight and bearing displacements below 200 mm under severe ground motions, which included all levels and soil types of Japanese bridge design motions. Elastoplastic systems developed significant permanent displacements.
References:	[1] National Center for Earthquake Engineering Research Reports 93-0020, 94-0002, 94-0014 and 94-0022. [2] Papers in *Earthquake Engineering & Structural Dynamics*, Vol. 25, No. 1, 1996, and *Engineering Structures*, Vol. 17. [3] Eight conference papers (two in Japanese).
Implementation:	(1) The demonstration of effectiveness of fluid dampers in isolation systems was helpful in the consideration of these devices in a number of projects, including the implementation at the San Bernardino Hospital project. (2) PWRI in Japan conducted verification tests for one of the tested systems. (3) Test results on the FPS system were instrumental in the specification of the system for a number of bridge projects in California.

Project: **Hybrid Control of Absolute Motion in Aseismically Isolated Bridges**

Researcher(s): Riley, M.A., Nagarajaiah, S., Reinhorn, A.M.

Proceedings Ref: Page 239

Status: Completed March 1996

Objectives: To explore the feasibility of reducing the movement of bridge decks with respect to piers and abutments without increasing forces in seismically isolated bridges by means of active control. The immediate objective was to explore an optimization scheme to control sliding or spring isolated systems using hydraulic servocontrol systems. Verify approach experimentally and theoretically.

Conclusions: (1) The control scheme is feasible to implement; (2) The improvement of performance of an actively controlled system versus a passively controlled system (isolators alone) is marginal. Displacement can be reduced up to 50% with large forces and force increase. (3) The hardware required to control the system, that includes the electrohydraulic system, provides a complex structural set-up which requires maintenance. (4) The solution explored may be cost-effective in extremely sensitive cases or when retrofit cannot be achieved otherwise in faulty passive systems. (5) Analytical solutions, including fuzzy sets controllers developed herein, can be used in other applications and provide state-of-the-art advances in the area of structural control.

References:

[1] Riley, M.A., Reinhorn, A.M., and Constantinou, M.C., "Active Control of Absolute Motion in Study Systems," in Dynamic and Control of Large Structures (L. Meirovitch, Editor), Virginia Polytechnic Institute Publication, Blacksburg, Virginia, pp. 243-252., May 1991.

[2] Nagarajaiah, S., Riley, M.A. and Reinhorn, A.M., "Hybrid Control of Sliding Isolated Bridges," *ASCE/Journal of Engineering Mechanics*, Vol. 119, No. 11, pp. 2317-2332, 1993.

[3] Reinhorn, A.M., Nagarajaiah, S., Riley, M.A., and Subramaniam, R.S., "Hybrid Control of Sliding Isolated Structures," in Structural Engineering and Natural Hazard Mitigation, [Ang AHS & Villaverde, (Eds.)], Vol. 1, ASCE, NY, pp. 766-791, 1993.

Implementation: The proposed control technique was not implemented directly; however sliding control devices are being developed by industry in the U.S. and abroad.

THIRD WORKSHOP RESEARCH SUMMARIES

Project:	**Seismic Retrofitting of Beam-Column Joints in Reinforced Concrete Box Girder Bridges**
Researcher(s):	Sanders, D.H., Douglas, B.M., Cornell, K.
Proceedings Ref:	Page 321
Status:	Pilot project completed Spring 1994. Currently, further research projects are under consideration by Nevada Department of Transportation and the California Department of Transportation.
Objectives:	Beam-column joints in older reinforced concrete box-girder bridges in seismic regions are often plagued with insufficient column bar development length and column ductility. The objective of the study was to investigate the shifting of the hinge down into the column away from the bent cap, so that there will be sufficient bar development length and ductility. A steel hinge shifter was being investigated which would provide easier construction in the field.
Conclusions:	The retrofit was successful in shifting the hinge location away from the column-bent cap interface and locating the hinge in a 2" gap left between the hinge shifter and the column jacket. The shifted hinge was able to reach a displacement ductility level of 6 before bar fracture occurred. The multiple bars fracturing proves the ability of the bars to be fully developed. This also brought concerns over the performance of lightly reinforced highly confined columns. The column bars reached higher strains than anticipated but only for a couple of cycles. After being cycled, the bars would slip. The loads were distributed out to the bolts. In both tests, bolt yielding was observed. The bolts should be dimensioned to prevent bolt yielding.
References:	[1] Sanders, D.H., Goudlas, B. and Cornell, K., "Bridge Seismic Retrofitting of Reinforced Concrete Beam-Column Joints by Hinge Shifting," Proceedings of 10th European Conference on Earthquake Engineering, Vienna, Austria, August 28-Sept. 2, 1994, Vol. 3, pp. 2241-2248. (Paper and Presentation). [2] Sanders, D.H., Douglas, B.M., and Cornell, K., "Retrofit Solutions for Inadequate Anchorage of Columns Bars into Pier Caps," Proceedings of the Ninth U.S.-Japan Workshop on Bridge Engineering, Japan, May 1993, pp. 401-414. (Paper and Presentation).
Implementation:	Results have been implemented from the pilot project but the Nevada and California DOTs are considering further research so that more refined design details can be developed.

Workshop Program

APPENDIX B

FOURTH NATIONAL WORKSHOP ON BRIDGE RESEARCH IN PROGRESS
JUNE 17-19, 1996
BUFFALO MARRIOTT HOTEL
BUFFALO, NEW YORK

FINAL PROGRAM

SUNDAY, JUNE 16

10:00 am - 4:00 pm	Pre-Workshop Sightseeing: Tour of Niagara Falls (optional)
5:00 pm - 8:00 pm	Registration
7:00 pm - 8:00 pm	Welcoming Reception

MONDAY, JUNE 17

7:30 am - 8:30 am	Registration
8:30 am - 10:00 am	**OPENING SESSION - WELCOME AND KEYNOTE PRESENTATIONS** *Session Chair - Ian G. Buckle, State University of New York at Buffalo* John B. Scalzi, National Science Foundation James D. Cooper, Federal Highway Administration Scott A. Sabol, Transportation Research Board James E. Siebels, American Association of State Highway and Transportation Officials
10:00 am - 10:30 am	Break
10:30 am - Noon	**SESSION I - BRIDGE MANAGEMENT AND CONDITION ASSESSMENT** *Session Chair - Mary Lou Ralls, Texas Department of Transportation* **Design of Bridge Inspection Programs for Structural Reliability**, G. Hearn, D. Frangopol and S. Marshall, University of Colorado **Life Cycle Cost Analysis for Bridges**, H. Hawk, National Engineering Technology Corporation

Life-Cycle Engineering for Bridge Decks, D. Veshosky, Lafayette College; S. Wagaman, C. Beidleman and J. L. Wilson, Lehigh University

Establishing Project Level Maintenance Policies for a Network Level BMS, T. Adams, J. Barut and P. Sianipar, University of Wisconsin-Madison

Fatigue Impacts on Bridge Cost Allocation and Truck Size and Weight Restrictions, J. Laman and T. Boothby, The Pennsylvania State University

Segment-Based Reporting for Element-Level Bridge Inspections, G. Hearn and D. Frangopol, University of Colorado

Implementation of an Optical Fiber Monitoring System in a Full Scale Bridge, R. Idriss, M. Kodindouma and K. White, New Mexico State University; M. Davis, D. Bellemore, A. Kersey, M. Putnam and E. Friebele, Naval Research Laboratory

Nondestructive Testing of Bridges, V. Saraf, A. Nowak and S. Kim, University of Michigan

Bridge Research in Progress at The University of Cincinnati, E. Aktan, et. al., University of Cincinnati

Noon - 1:30 pm

Lunch
Speaker: Commissioner John B. Daly
New York State Department of Transportation

1:30 pm - 3:30 pm

SESSION II - SEISMIC PERFORMANCE, DESIGN, AND RETROFITTING (part 1)
Session Chair - Zolan Prucz, Modjeski and Masters, Inc.

Seismic Analysis of Bridges Including Soil-Structure Interaction Effects, R. Betti and K. Huang, Columbia University

Practical Nonlinear Time-History Analysis for the Design of Bridge Structures Subjected to Strong Seismic Motion, R. Dowell, F. Seible and N. Priestley, University of California, San Diego

On Near-Field Earthquake Ground Motion Simulation and Its Effects on Long-Period Structures, R. Zhang and M. Shinozuka, University of Southern California

Evaluation of Bridge Abutment Stiffness During Earthquakes, R. Goel, Syracuse University

Characterization of Nonlinear Abutment Stiffnesses for Seismic Design and Retrofit, R. Siddharthan and M. El-Gamal, University of Nevada, Reno

Three-Dimensional Modeling of Stiffness Characteristics for Highway Bridge Abutments, J. C. Wilson, McMaster University; M. Justason, Berminghammer Foundation Equipment, Ltd.

Behavior of Laterally Loaded Piles Supporting Bridge Abutments, J. Deatherage, E. Burdette, and D. Goodpasture, University of Tennessee

The Use of Reticulated Micropile Groups for Bridge Foundations, F. Kulhawy, Cornell University; J. Mason, California Department of Transportation

Repair of Damaged Pier Walls, M. Haroun, G. Pardoen, H. Bhatia, S. Shahi, and R. Kazanjy, University of California, Irvine

Modeling Seismic Damage of Circular Reinforced Concrete Bridge Columns, A. Taylor and W. Stone, National Institute of Standards and Technology; A. El-Bahy and S. Kunnath, University of Central Florida

Seismic Fragility Analysis of Conventional Reinforced-Concrete Highway Bridges, C. Mullen and A. Cakmak, Princeton University

3:30 pm - 5:00 pm **POSTER SESSION**

6:30 pm - 7:30 pm Reception/Cash Bar

7:30 pm - 9:00 pm Dinner

Speaker: *Dr. William J. Rae*
Professor, Mechanical and Aerospace Engineering
State University of New York at Buffalo
"The Flight Dynamics of a Football"

TUESDAY, JUNE 18

7:30 am - 8:30 am Registration

8:30 am - 10:00 am **SESSION III - CONCRETE, MASONRY, AND COMPOSITE CONSTRUCTION**

Session Chair - Frieder Seible, University of California, San Diego

Release Methodology of Prestressing Strands, J. Kannel, C. French, and H. Stolarski, University of Minnesota

Durability of Bridges with Full Span Prestressed Concrete Panels, R. Peterman and J. Ramirez, Purdue University; R. Poston, Whitlock Dalrymple Poston & Associates

High Performance Concrete in Texas Bridges, N. Burns, S. Gross and M. Braun, The University of Texas at Austin

Field Measurement and Evaluation of Time-Dependent Losses in Prestressed Concrete Bridges, O. Onyemelukwe and C. Mills, University of Central Florida; M. Issa, Florida Department of Transportation

Field Monitoring of Prestress Forces in Four Box Girder Bridges Subjected to High Variation of Humidity, M. Saiidi and N. Mangoba, University of Nevada, Reno

Load Rating of Masonry Arch Bridges and Culverts, T. Boothby, The Pennsylvania State University; V. Dalal, Ohio Department of Transportation

Innovative Prestressed Steel Composite Short to Medium Span Bridges, D. Burroughs and J. Lockwood, J. Muller International; J. Hartmann, American Iron and Steel Institute

Slab Participation for Axial Forces in Composite Cable-Stayed Bridges, D. Byers, HNTB Corporation; S. McCabe, The University of Kansas

Nonlinear Finite Element Analysis of Composite Bridges, K. Fu and F. Lu, The University of Toledo

10:00 am - 10:30 am Break

10:30 am - Noon **SESSION IV - STEEL BRIDGES**

Session Chair - Arun Shirole, National Steel Bridge Alliance

FHWA Curved Steel Bridge Research Project, M. Grubb and J. Yadlosky, HDR Engineering, Inc.; A. Zureick, R. Leon and J. Burrell, Georgia Institute of Technology; D. Hall, BSDI, Ltd.; C. Yoo, Auburn University; S. Duwadi, Federal Highway Administration

High Performance Steels for Highway Bridges, W. Wright, Federal Highway Administration

Structural Systems for High Performance Steel Bridges, J. Kulicki, W. Wassef and P. Ritchie, Modjeski and Masters, Inc.

Characterization of the Environment for Weathering Steel Design Considerations, C. Farschon and J. Ault, Ocean City Research Corporation; R. Kogler, Federal Highway Administration

Expansion Joint Elimination for Steel Bridges, G. Tsiatas, E. McEwen and W. Boardman, University of Rhode Island

Bridge Girders with Corrugated Webs, M. Elgaaly, Drexel University

A Passive Fatigue Life Indicator for Highway Bridges, D. Thomson and G. Samavedam, Foster Miller, Inc.; W. Wright, Federal Highway Administration

Vincent Thomas Bridge Monitoring Tests, A. Abdel-Ghaffar, S. Masri and R. Nigbor, University of Southern California

Research in Progress on Steel Bridges at University of Washington, C. Roeder and G. MacRae, University of Washington

Noon - 1:30 pm — Lunch

1:30 pm - 3:30 pm — **SESSION V - SEISMIC PERFORMANCE, DESIGN, AND RETROFITTING (part 2)**
Session Chair - Bruce Douglas, University of Nevada, Reno

Bridge Damage and Its Consequences in the 1994 Northridge Earthquake, A. Kiremidjian, N. Basöz, S. King and K. Law, Stanford University

Seismic Retrofitting Experience and Experiments in Illinois, W. Gamble and N. Hawkins, University of Illinois at Urbana-Champaign; I. Kaspar, Illinois Department of Transportation

Strength Degradation of Existing Bridge Columns, O. Jaradat and D. McLean, Washington State University; M. Marsh, BERGER/ABAM Engineers, Inc.

Analysis and Design of Bridge Columns with Lap-Spliced Longitudinal Reinforcement, Y. Xiao and R. Ma, University of Southern California

Carbon Fiber Seismic Retrofit of Poorly Confined Square Reinforced Concrete Columns Subjected to Large Axial Forces, J. Stanton, G. MacRae and K. Nosho, University of Washington

Pile to Pile Cap Connection Test Series, F. Seible, N. Priestley, and P. Silva, University of California, San Diego

	Substructure Protection by Ductile End-Diaphragms in Steel Bridges, S. Zahrai and M. Bruneau, University of Ottawa
	Design of Seismic Restrainers, M. Eberhard, J. Stanton and P. Trochalakis, University of Washington
	Field and Laboratory Studies on the Seismic Performance of Bridge Bearings, J. Mander, S. Chen, D-K. Kim and D. Wendichansky, State University of New York at Buffalo
	Seismic Design of Bridges Using Spring-Viscous Damper Isolation System, A. Parvin, The University of Toledo
	Longevity and Reliability of Sliding Isolation Systems, M. Constantinou, P. Tsopelas and A. Kasalanati, State University of New York at Buffalo
3:30 pm - 5:00 pm	**POSTER SESSION**
6:30 pm - 7:30 pm	Reception/Cash Bar
7:30 pm - 9:00 pm	Dinner *Speaker: Dr. Jack Quinan* *Professor and Chair, Art History Department* *State University of New York at Buffalo* *"Frank Lloyd Wright: The Architect as Artist and Engineer"*

WEDNESDAY, JUNE 19

7:30 am - 8:30 am	Registration
8:30 - 10:00 am	**SESSION VI - COMPOSITE AND OTHER MATERIALS** *Session Chair - F. Wayne Klaiber, Iowa State University*
	Feasibility of GFRP/CFRP Prestressed Concrete "Demonstration" Bridge in the USA, N. Grace, Lawrence Technological University; G. Abdel-Sayed, University of Windsor
	Development of the Carbon Shell System Construction Concept for New Bridge Structures, F. Seible, R. Burgueño, and A. Davol, University of California, San Diego
	Fabrication and Testing of Fiber Reinforced Composite Bridge Decks, V. Karbhari, L. Zhao and Y. Gao, University of California, San Diego

A New FRP-Encased Bridge Pier Column, M. Shahawy, Florida Department of Transportation; A. Mirmiran, University of Central Florida

FRP Rebar with Enhanced Ductility and Sensing Capability, A. Belarbi, K. Chandrashekhara and S. Watkins, University of Missouri-Rolla

Advanced Composite Stay Cables, M. Wernli and F. Seible, University of California, San Diego

Optimization of Pultruded GFRP Composite Bridge Railing System, S. Seangatith and R. Yuan, The University of Texas at Arlington

Accelerated Test Methods for FRP/Concrete Systems in Highway Structures, T. Boothby, A. Nanni and C. Bakis, The Pennsylvania State University

An In-Situ Aluminum Girder Highway Bridge Test and Laboratory Fatigue Tests, R. Abendroth and W. Sanders, Iowa State University

10:00 am - 10:30 am   Break

10:30 am - Noon   **SESSION VII - ANALYSIS, LOADS, AND BEARINGS**
Session Chair - Catherine French, University of Minnesota

Continuing Research in the Analysis and Design of Long-Span Bridges for Wind Loads, N. Jones and R. Scanlan, The Johns Hopkins University

Aerodynamic Investigations of the Deer Isle-Sedgwick Bridge, H. Bosch, Federal Highway Administration

Bridge Pier Analysis for Ship Impact and Other Loads, M. Hoit, M. McVay and C. Hays, University of Florida

Dynamics of a Cable-Stayed Bridge During Construction, N. Jones and C. Pettit, The Johns Hopkins University; J. C. Wilson and A. ElDamatty, McMaster University

Longitudinal Loads in Railway Bridges, D. Foutch and D. Tobias, University of Illinois at Urbana-Champaign

Tests of Kevlar Prestressed Timber Beams for Bridge Girder Applications, T. Galloway, C. Fagstad, C. Dolan and J. Puckett, University of Wyoming; M. Ritter, USDA Forest Products Laboratory

Performance of Timber Bridge Railing Under Vehicle Impact, R. Wollyung, A. Scanlon, M. Carpino and B. Gilmore, The Pennsylvania State University

An Experimental Study of Flat and Tapered Elastomeric Bridge Bearings with Design Recommendations, J. Mascarella, United States Military Academy; J. Yura, University of Texas at Austin

Investigation of Two Bridge Alternatives for Low Volume Roads, T. Wipf, F. Klaiber, B. Phares, and J. Reid, Iowa State University

Noon - 12:30 pm

CLOSURE
Chair: Ian G. Buckle, State University of New York at Buffalo

1:00 pm - 3:00 pm

Post-Workshop Field Trip: NCEER Seismic Simulator Laboratory Tour and Luncheon (optional)

Author Index

APPENDIX C

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AUTHOR INDEX

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