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BRIDGE RESEARCH IN PROGRESS

Proceedings

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BRIDGE
ENGINEERING
IOWA STATE UNIVERSITY
CENTER

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Proceedings of a symposium funded by the National Science Foundation

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held in
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Symposium Co-Directors:
F. Wayne Klaiber and Wallace W. Sanders, Jr.
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BRIDGE ENGINEERING CENTER
IOWA STATE UNIVERSITY

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PREFACE

Iowa State University, through a grant from the National Science Foundation, sponsored this Symposium on Bridge Research in Progress. The symposium provided an opportunity for researchers to briefly outline current bridge research and discuss recent developments.

These proceedings contain summaries of papers presented at the symposium on September 26–27, 1988, in Des Moines, Iowa. The sponsors wish to express their appreciation to the authors for their efforts.

The symposium was organized by staff of the Bridge Engineering Center in ISU's Department of Civil and Construction Engineering with help from the Engineering Research Institute. ISU gratefully acknowledges the support of the National Science Foundation (Systems Engineering for Large Structures: John B. Scalzi, Program Director) and the guidance of the steering committee.

Symposium Co-Directors
F. W. Klaiber and W. W. Sanders, Jr.

Steering Committee

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B. M. Douglas	J. B. Scalzi
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KEYNOTE PRESENTATIONS

BRIDGE RESEARCH IN PROGRESS SPONSORED BY FEDERAL HIGHWAY ADMINISTRATION

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SYNOPSIS

The FHWA bridge research program is structured to address five High Priority National Problem Areas (HPNPA's) identified by field personnel, operations engineers and research engineers throughout the country. The five problem areas relate to the conduct of research on live load effects on bridges; prestressed concrete protection; geotechnology; steel bridge coatings and bridge substructure evaluation. Each of these areas of research and research in progress is summarized below.

INTRODUCTION

The Federal Highway Administration is funding five focused areas of research related to highway bridge design, construction, rehabilitation, maintenance and management. The scope of each of these High Priority National Problem Areas (HPNPA's) is intentionally kept narrow to allow for a full solution to problems identified by leading researchers and practioners.

The first of the five problem areas identified is Live Load Effects. The goal of this research effort is to provide better analytical tools, verified by field testing, to allow the States' bridge engineers to design new bridges and rate (for posting purposes) existing structures. This will provide guidance to management and allow the highest truck weights (axle and gross) economically feasible, without jeopardizing the safety or reducing the service life of the bridges.
Mr. John O'Fallon serves as the FHWA Program Manager.

The second area is Prestressed Concrete (PS/C) Protection. Much effort has been expended to solve the problem of concrete deterioration due to corrosion of reinforcing steel. With PS/C, corrosion of the steel may cause failure before concrete deterioration is apparent. This focused program is intended to provide specific guidelines for new designs which will be corrosion-resistant and to develop procedures to provide protection of the many existing PS/C members. Dr. Paul Virmani is the FHWA Program Manager.

The third area is Geotechnology. Much attention and effort has been devoted to this area over the past few years with great success as evidenced by the wide acceptance of mechanically stabilized earth-retaining structures over the older traditional reinforced concrete retaining walls. The cost savings to the highway industry in this area alone is impressive. This research program is devoted to completing work on remaining geotechnical problem areas, such as providing better guidelines for the design of drilled shafts to more fully use their load-carrying capacity, and putting all of the various design techniques together into a comprehensive manual that will gain ready acceptance by the State geotechnical engineers. Mr. Al DiMillo is FHWA Program Manager for this area of research.

The fourth high priority area, Steel Bridge Coatings, is intended to provide solutions to one of the bigger problems facing bridge owners today. The EPA regulations are seriously affecting industry's abilities to use existing paint systems that have been proven over the years to give good cost-effective protection of structural steels. One part of this program is to develop a means of accelerated testing of new paints being developed by industry to meet the EPA requirements. Another environmental problem is how to remove and dispose of lead-base paints on existing structures when repainting is required. In at least one instance, a bridge was abandoned because removal/disposal of the painted system was so costly. This effort will be devoted to developing acceptable cost-effective means of paint removal. In addition to the paint removal problem, existing bridge steels, with or without (weathering steel) initial paint systems, are generally contaminated with salt (roadway or marine) which seriously affects the service life of field-applied paints. Procedures to adequately clean these contaminated steels will be assessed and refined to provide more cost-effective repainting activities.
Mr. John Peart serves as the FHWA Program Manager.

The last of the high priority areas, Bridge Substructure Evaluation, will result in the development of procedures and criteria to systematically evaluate the condition of critical substructure elements of existing bridges and the development of strategies for the best use of available inspection and evaluation resources. Since more bridges are damaged or destroyed by scour action during floods, it is imperative to include the development of an efficient and effective underwater bridge substructure and scour inspection and evaluation system in this program. Dr. Roy Trent is the FHWA Program Manager.

LIVE LOAD EFFECTS

At present, bridges are designed using the AASHTO H and H-S truck loadings for short to medium span lengths, an alternative lane load for longer spans, and a heavy two axle military load for very short spans. The most common load, the HS-20 truck, features a 36 ton three-axle tractor semi-trailer, two of which are 16 tons each. Today many trucks on the road are heavier than the AASHTO design loads. Legal live loads are theoretically controlled by the so-called "bridge formula." This formula is intended to limit overstress of H-15 bridges to 30% of design and overstress of HS-20 bridges to 5%.

Until now, a simplified analysis and two-dimensional load distribution have compensated for the relatively light design loading. Unfortunately, with experimental stress measurements and three dimensional analysis indicating much lower live load stresses, unrealistic expectations for reserve strength have been raised. On the other hand, additional capacity does exist and will certainly reduce the need for posting, rehabilitation or replacement of some bridges. Either way, it is clear that some improvement in the handling of live loads in the design or rating of bridges is needed.

Any revision to the live load models must consider dynamic and fatigue effects. With revision of the AASHTO Bridge Specifications and the AASHTO Manual for Maintenance Inspection of Bridges beginning, it is imperative to study live load effects now.

To achieve this goal requires a realistic determination of truck loads currently applied to bridges, a projection of their likely growth and an evaluation of their affect on bridges.

Research in Progress

A summary of current FHWA contract staff research and research sponsored by State highway agencies follows. There are significant research efforts being conducted under the auspicous of NCHRP and TRB which are listed but not synopsised in this summary.

Variable amplitude load effects. This study by the University of Pittsburgh will be completed in 1991. It is investigating the current fatigue design rules and the possibility of adopting a variable amplitude, bi-modal, stress range spectrum. This new load spectrum model is to be tested experimentally to verify its performance.

Structural modelling for autostress by loading thru various deck systems (innovative bridge). This joint study with AISI and its contractor, Wiss, Janney, Elstner Assoc. is testing the application of autostress theory to a 0.4 scale model of a steel, two span continuous, three girder bridge with a modular PS/C deck at FHWA's new structures laboratory. A great deal of information about load distribution, structural response before and after shakedown, and performance of modular deck panels will be gained when the study is completed in 1989.

Load prediction and structural response. This is the major study within this HPNPA. It is a dual-purpose effort that will gather detailed truck traffic and experimental bridge response information using weigh-in-motion technology while making an analytic determination of whole bridge behavior under live load.

This study will identify the specific details of traffic, truck configurations, axle loads, bridge response, and structure types which are needed for more realistic load evaluation and bridge design.

Truck load and bridge response information will be collected for a broad spectrum of site conditions. The database will be evaluated to establish an accurate and current representation of truck loadings and associated bridge response. This work will be paralleled by an analytical study of bridge response. The response of each component to truck loads will be determined through a three-dimensional analysis. The study will use the results of the current studies cited above. Information on actual vs. predicted stress levels in bridge members will be developed and verified by comparison with the experimental data. As a minimum, steel, prestressed and reinforced concrete multi-beam bridges will be investigated. It is also intended to determine if an improved empirical design analysis which is truly representative of a structure's behavior can be developed. This study will be completed in 1991.

Bridge overstress criteria. This two-year study will review the rationale for the level of allowable overstress and evaluate alternative criteria for bridges. The effect of fatigue on the overstress criterion for bridges will also be considered. In addition, the "Bridge Formula" and proposed "Texas Formula" will be reviewed with an assessment of the impact of increased or decreased legal load limits based on different levels of enforcement and overload permitting practices. The impact of various levels of maintenance activity (including retrofitting fatigue-sensitive details to raise capacity) will be assessed to provide guidance in areas where maintenance or retrofitting may provide substantial benefits by increasing load capacity and allowing higher truck loads. The study will be completed in 1990.

Maryland HP&R study, utilization of WIM to determine the effect of truck traffic on bridges. This HP&R study is being conducted by the University of Maryland for the Maryland SHA with a 1989 finish date. It is aimed at developing a better understanding of bridge response with emphasis on skew effects, location of neutral-axis, effective composite flange width, load distribution and impact factors. Results of this study will supplement research planned for this HPNPA.

NCHRP research studies relating directly to this problem area include: Distribution of wheel levels on highway bridges, inelastic rating procedures for steel beam and girder bridges, nondestructive load testing for bridge evaluation and rating, development of site specific load models for bridge ratings and development of a comprehensive bridge specification and commentary.

PRESTRESSED CONCRETE PROTECTION

Use of PS/C members in bridge construction is relatively new. Hence, corrosion problems and concrete deterioration are just beginning to surface. Although PS/C members are generally manufactured with higher strength concretes, time has shown that they are subject to the effects of corrosion in the same way that reinforced concrete is. Documented cases of strands breaking as a result of corrosion, make this a most pressing problem. Since PS/C members rely on the tensile strength of the strands to resist loads, loss of even a few strands per member could prove catastrophic. In addition, because of the high stress in the strands, corrosion effects are accelerated. Even small corrosion pits could cause fracture of a strand, as compared to mild steel reinforcing that will literally rust away before breaking.

Of particular concern are segmental bridges where concrete cover over strands and anchorages is minimal. The strands may be in paper wrappings, plastic tubes or steel conduits. These strands may be protected with a rustproofing grease, grouted or left unprotected. Canadian research on post-tensioned members removed from existing bridges shows that there is reason for concern. Recent collapses of two post-tensioned bridge deck panels in England and India raise significant concern about the degree of protection provided by the grout covering the prestressing steel in ducts. Detailed investigations on these two recent collapses are underway. In addition to the direct penetration through concrete cover, chloride ions from either deicer salts or marine environments invariably find their way into the ducts and grouted strands some distances away from the anchorage locations. Also pre-tensioned beams in a few bridges have exhibited corrosion of embedded strands. The corrosion in these strands is attributed to the chloride ions from deck deicers which penetrated the beam's concrete cover.

Research in Progress

Following are summaries of FHWA contract studies and State HP&R studies:

Salt penetration and corrosion in prestressed concrete members. This field study by Construction Technology Laboratories, Inc., is investigating the corrosion of prestressing steel caused by chloride ions in selected PS/C bridge components. Based on the analysis of the collected field data, useable methods for assessing the condition of prestressing steel will be identified while rehabilitation with a suitable protective system is still feasible.

Cathodic protection (CP) developments for prestressed components. A contract is being awarded to develop and apply effective cathodic protection systems to PS/C bridge components and develop durable cost-effective anode materials for their use on these PS/C bridge components. The development of the anode materials will be for all vertical and overhead concrete surfaces;but will exclude bridge decks.

Protecting prestressing steel in ducts. A contract is being awarded to develop and test new mix designs for grout used in ducts so that the grout may provide long term (more than 50 years) corrosion protection for the prestressing steel in post-tensioned concrete bridge structures. In addition, accelerated test methods will be developed to test these grout mixtures to predict their performance over the design life of the bridge.

Florida HP&R: The mitigation of corrosion of reinforcing and prestressed steel in marine environment. The objective of this study is to estimate time to corrosion, corrosion rate and life expectancy of both plain and prestressed steel in concrete. Particular emphasis will be placed on marine piling and similar submerged structures. The study is scheduled to be completed by June, 1989.

Florida HP&R: Cathodic protection and environmental cracking of steel tendons in prestressed concrete. The objective of this research is to investigate the tendency of high strength pretensioning steel tendons to undergo brittle cracking due to (accelerated) cathodic protection currents under simulated conditions. The results will be used for design and application of cathodic protection on corroding prestressing tendons in chloride contaminated concrete.

The following NCHRP studies are providing important results required to complete objectives of this problem area: Non-destructive methods for field inspection of embedded or encased high strength steel rods and cables, corrosion protection of prestressing systems and durability of prestressed concrete highway structures.

GEOTECHNOLOGY

This program area addresses the problem of developing a comprehensive design and construction guideline manual or a series of manuals for foundations of highway bridges and various ground improvement techniques. The guidelines manuals will be used by State, Federal, and local highway agencies to design and construct more efficient and cost-effective bridge foundations, retaining walls, cut-slopes and embankments which account for over 50 percent of the cost of most highway projects.

Geotechnical engineering for foundations and earth structures has lagged behind other highway engineering disciplines in evolving from an art to a science. Many of the commonly used design techniques suffer from a lack of precise definitions and an imperfect understanding of fundamental mechanisms that govern the behavior of geotechnical structures.

Much of the difficulty and expense of defining soil behavior involves the inconsistencies and uncertainties associated with applying engineering principles to nonhomogeneous ground materials. Predicting the response of soils to load transferred by various piles in a pile group or tensile elements in a reinforced soil are special cases where more precise definitions would lead to more economical design.

Research in Progress

Pile load test data base. An FHWA study is underway to develop a high quality data base for pile load tests. The data base will serve as a "standard" against which new and existing design procedures can be compared.

Model testing of piles and pile groups. An FHWA staff study on model piles is underway to develop additional load test data for verifying and refining pile design procedures.

Behavior of reinforced soil. A series of laboratory model tests and full-scale field tests were recently conducted on several types of soil reinforcement systems. The experimental test data will be used to evaluate soil and reinforcement parameters required for verification and refinement of existing design and construction guidelines.

Corrosion/durability of soil reinforcing elements. A research study is currently underway to develop new techniques and equipment to determine corrosion potential of metallic reinforcing elements and durability of non-metallic elements as related to the design life of the reinforced soil structures.

Soil nailing for highway cut slopes. A series of laboratory model tests and full-scale field tests are currently underway to evaluate soil and reinforcement parameters. Data analysis and evaluation of existing procedures will be used to develop a comprehensive design and construction guidelines manual.

Three NCHRP studies relating to the geotechnology area are currently in progress. They are: Bituminus coatings to reduce drag loads on piles, recommended revisions to AASHTO foundation specifications, and load factor design criteria for bridge foundations.

STEEL BRIDGE COATINGS

The National Bridge Inventory indicates there are approximately 224,000 steel bridges in the United States. Of these, 134,000 are listed as deficient, many of which will require major refurbishment or replacement. this is the result of deferred maintenance practices and low prioritization of maintenance budget dollars for painting and corrosion protection. The problem is exacerbated by the fact the great majority of the structures requiring maintenance have been protected by materials now considered hazardous to the environment, thus requiring expensive methods of removal, containment and disposal. The exclusion of the use of potentially hazardous materials in paint formulations and tight limits on their volatile organic solvent content has eliminated use of many effective coatings. These environmental constraints have introduced the critical need for innovative technological advances in corrosion control material and methods. The industry is facing drastically increased maintenance cost and steel replacement rates if timely and effective development and application of new technology is not initiated.

The program research elements are organized under three groupings; 1) Environmentally Acceptable Corrosion Materials and Methods, 2) Improved Performance and Testing, and 3) New Approaches to Long-Term Corrosion Control.

Research in Progress

Maintenance coating of weathering steel. The Steel Structures Painting Council has completed the study's laboratory testing phase to verify proposed substrate cleaning methods and the evaluation of coating systems for the protection of field corroded weathering steel. The field exposure phase of the study is in progress.

Performance of alternate coatings in the environment (PACE), phase II. The laboratory accelerated testing and the comprehensive field exposures, including bridge exposures, in various field environments has been completed. The data is being evaluated and analyzed statistically to determine viable conclusions.

A preliminary report on the Alternate Pigment portion of the study concluded that the formulations containing corrosion inhibiting pigments designed to replace lead did not perform as well as the lead containing materials. Zinc oxide's performance in the tests has identified it as a possible alternate pigment.

Other elements investigated in the study were: 1) Alternate Surface Preparation, 2) Waterborne Coatings, 3) Waterborne Epoxsies, and 4) Alternate Systems.

Development of performance test criteria and specifications for coatings. The objective of the study is to develop an accelerated testing program for bridge coatings. The results obtained with the developed test regimen should reliably predict coating life in bridge service. This will provide the coating specifier a valuable tool in determining life cycle cost of the available corrosion control options.

The effect of substrate contaminants on coating life. The cleanliness of the substrate to be painted is critical to the performance life of the applied coating. Critical concentration levels of the different contaminants associated with the service environment has not been identified previously. Contaminant species resulting from both environmental sources and transfer by the abrasive used in the cleaning process will be identified and threshold limits determined. The information obtained will be used to develop a bridge cleaning standard to insure the maximum coating performance is achieved.

Environmentally acceptable materials for the corrosion protection of steel bridges. The objective of the study is to identify cost effective environmentally acceptable corrosion protection materials and methods for steel bridges. Both shop applied and field maintenance systems are being evaluated. The bridge structure and components will be characterized for the severity of exposure and rate of corrosion. A cost comparison of the available corrosion control options for the identified corrosion environment will be made based upon projected life cycle costs. Types of materials and methods being evaluated include; high solids, waterborne, powder, and metal sprayed coatings.

BRIDGE SUBSTRUCTURE EVALUATION

The present procedures used by bridge owners to evaluate the performance capability and relative safety of existing bridges are based upon generalized criteria and guidelines which are not adequate for many of the critical bridge substructure elements that may result in failure and sudden collapse of the bridge.

It is imperative for the bridge owner to provide an efficient and effective underwater bridge substructure and scour inspection and evaluation program since approximately 86 percent of the 576,000 ridges in the National Bridge Inventory are built over water. More bridges are damaged or destroyed by scour action during floods than from any other cause. As flood waters recede, waterborne sediments re-deposit and scour holes are refilled. Conventional scour inspection, and measurement methods can provide misleading information, and contribute to erroneous conclusions on scour vulnerability. Improved engineering scour evaluation techniques are required for credible scour inspection ratings.

Where bridge substructure units are underwater and diving or probing are not feasible, conventional inspection procedures do not give reliable assessment of the integrity of inservice bridges. In many cases the actual substructure configuration may be unknown. While advanced inspection techniques are known to exist, the suitability and practicality for use underwater is not well established. Practical and reasonable inspection methods and engineering evaluation techniques for substructure integrity and scour vulnerability are needed to avert catastrophic bridge failures.

Research in Progress

Riprap sizes for protection of bridge piers from scour. This is an FHWA laboratory study to determine riprap sizes needed to protect bridge piers from scour. Lab experiments are being conducted to get an accurate velocity profile around piers of different shapes and to test stability of model riprap of different sizes.

Evaluation of design practices for riprap used in protection of highway crossings. this study by the U.S. Geological Survey and the Sutron Corporation is near completion. Two reports were published by USGS in 1986. FHWA awarded a follow up implementation contract to the Sutron Corporation to revise the FHWA hydraulic engineering circular (HEC-11) on riprap design procedures based on the USGS results. A draft of that circular is currently being updated.

Performance of bridges during flooding. This study by the U.S. Geological Survey features a team of investigators who will go to flood sites to monitor stream bed movement during floods. This study is generally referred to as the National Scour Study because it serves as a focal point for the State sponsored scour investigations. This is a 4-year study that is scheduled for completion in 1991.

Subbottom profiling of refilled scour holes. This study by the U.S. Geological Survey is to assess the capability of selected geophysical sensing methods in determining subsurface riverbed elevations and geologic characteristics of sub-bed materials near bridges. Geophysical techniques being explored include ground penetrating radar, continuous seismic reflection profiling, black and white fathometer, and color video echo sounder.

HP&R field scour studies. These studies are being sponsored by Arkansas, Arizona, Louisiana, Oklahoma, Delaware, Virginia, Maryland and Washington State using either State or HP&R funds. These studies are aimed at reconnaissance prior to flooding and scour monitoring during flooding to document field data. Data from these studies will be fed into the national study described above.

Virginia HP&R: Major factors affecting the performance of bridges during flooding. This study investigates the feasibility of analyzing soil borings and other site data after a flood to reconstruct the amount of scour that may have occurred during a flood.

An NCHRP study, Hydraulic Analysis of Bridges on Streams with Moveable Beds and Banks is being conducted by Simons and Associates.

SUMMARY

This paper summarizes the FHWA sponsored highway bridge research program. Only those research studies in progress have been discussed. Additional detailed information on research completed and future research can be obtained directly from the program managers listed in the introduction to this paper. Four page summaries which document each HPNPA will be available shortly from the Federal Highway Administration. They provide more detail on the research in progress. Proposed research studies are outlined and the FHWA Technology Transfer program relating to each HPNPA is summarized.

HIGHWAY BRIDGE RESEARCH IN CANADA-AN OVERVIEW

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SYNOPSIS

This paper contains a brief summary of some of the recent and ongoing research in highway bridge engineering conducted in Canada. This research is carried out mainly at universities and at research departments of some Provincial Ministries of Transport. Much of Canadian bridge engineering research has direct relevance to bridge design codes, in particular to the Ontario Highway Bridge Design Code, which has gained wide acceptance in recent years. The more extensively researched topics include bridge analysis, live loads and their effects on bridges, soil-steel structures and timber bridges.

INTRODUCTION

The design, construction and maintenance of highways in Canada fall under Provincial jurisdictions so far as the ten provinces are concerned, and under Federal Government jurisdiction in the Yukon and Northwest Territories. Accordingly, highway bridge research in Canada is conducted not only in some twenty of its universities but also in research departments of some of the Provincial Ministries of Transport. Of these the most active research group is in the Ministry of Transportation of Ontario (MTO). The university research is supported by a combination of research grants from the Natural Sciences and Engineering Research Council of Canada and research contracts from various sources, notably the Provincial Ministries of Transport. Indeed, the MTO is committed to spending a certain portion of its research budget on contract research carried out at the various universities (9 for bridge research) in the Province of Ontario.

This paper presents an overview of the state-of-the-art in Canadian bridge research. The overview can be presented in several ways; for example it would be possible to arrange the material by geographical location in which the research is being performed; however, in this paper it is preferred to arrange a presentation in terms of subject areas.

CODES

In universities, there is a long standing tradition of research that is undertaken because of the wish of the researcher to undertake an investigation that is of interest to that person. In bridges research, such 'curiosity motivated' research certainly does occur in Canada as well, but it is not often that the end product of such research gets used in bridge design or evaluation. Another source of motivation for bridge research comes, however, from familiarity with existing codes of practice and recognition of aspects of those codes that could bear improvement. Much recent research on bridge engineering in Canada has come about by this second kind of motivation; this has two distinct advantages, the one being that the fruits of such research are more likely to be immediately applicable to bridge design or evaluation and the second that the various research projects undertaken in different locations are more likely to be complementary to one another.

In Canada, the adoption by the MTO of the limit states design philosophy for the Ontario Highway Bridge Design Code (OHBDC) has had a noticeable mark on bridge research carried out during the past ten or twelve years.

It is worth recalling that the probability-based limit states design method is not different in format from that of the load and resistance factor design method, and that the limits states design philosophy does not relate only to the failure limit state; it also deals with serviceability aspects of the structure under loads well below the collapse levels.

Loads and Load Factors

The formulation of the design loading for OHBDC was based on vehicle weight surveys conducted in the Province of

Ontario. The results of these surveys are reported in (27). Ref. (2) deals with the accuracy of weigh-in-motion systems which were used for these surveys. The forthcoming CSA bridge design code is also in limit states format; as for the OHBDC, its design loading and related load factors are based on vehicle surveys which were conducted across Canada. The details of the development of the CSA design loading are presented in (3). It is noted that the OHBDC design truck has five axles with the base length of 18 m and the total weight of 700 kN. The CSA truck, on the other hand, has four axles with a base length of 16 m and variable total load. The total weight for the CSA design vehicle which is recommended to be used for bridges on interprovincial roads, is 600 kN. It is worth noting that the live load factors for OHBD and CSA codes are 1.40 and 1.60, respectively.

The work on the determination of the dynamic load allowance (DLA), or the impact factor, was conducted in Ontario through a fairly large number of dynamic tests on in-service bridges (13, 14). This work confirmed the validity of the OHBDC provisions for DLA which relate its value to the first natural frequency of the bridge. The OHBDC provisions for DLA have also been adopted in toto by the forthcoming CSA code for highway bridge design.

Another aspect of the application of probabilistic methods on the load side of the design equation lies in the determination of reduction factors for multiple presence of vehicles. The work reported in (29), has led to the conclusion that for evaluation, different reduction factors can be used for bridges carrying different volumes of traffic.

Turning to the resistance side of the design equation it may be noted that adequate statistical data has been available for some time on the strengths of some structural materials, whilst there has been a significant lack of information about others, especially timber. This lack is being addressed by work at the University of British Columbia (e.g. 35). Work is also underway in Ontario to determine the failure load of timber bridges (e.g. 28).

In reporting research that has originated in Canada it is important to emphasize the degree of collaboration that has been forthcoming from a number of well known American research engineers who have responded to Canadian invitations for their input and participation. For example, in the calibration studies relating to the OHBDC the work reported in (25) and (45) is recognized as being important. Part of this work was carried out in the U.S.A.

ANALYSIS

As might be expected, a very large amount of work has been done and is going on in the development of methods of analysis of bridges. It is interesting and important to note that several methods have been developed wholly or mainly in Canada and that refinements to these methods so as to increase their power and range of applicability are well underway. Recent and ongoing research on these methods is discussed immediately below.

Simplified Methods

Unlike their European counterparts, bridge engineers in North America are fond of using simplified methods of transverse distribution of live loads. In conformity with this tradition, a large number of simplified methods has been developed from the results of rigorous analyses. Initially these methods were developed for the OHBDC but later their applicability was enhanced to include AASHTO and CSA codes (5, 8 and 10). It is noteworthy that some of these methods have also been incorporated in the forthcoming CSA bridge design code.

Semi-Continuum Method

The method of analysis in which a bridge superstructure is idealized as a series of longitudinal beams and a transverse continuum was developed in the 1950's. This method, which has been used extensively in several countries, is called the semi-continuum method. In its earlier form it was limited to bridges with negligible torsional stiffness. Recently the semi-continuum method has been generalized to include any degree of torsional stiffness and adapted a to implementation on micro-computer (30).

Finite Strip Methods

Much of the early work on the finite strip method was carried out in the late 1960's and early 1970's in Canada, notably at the University of Calgary.

In recent years the power and applicability of the finite strip method has been greatly increased so as to render it readily applicable to a) bridges having girders of varying flexural and torsional rigidities (18); b) continuous bridges (17); c) non-linearity of structural responses arising from the geometry of the structure; and d) non-linearity of structural responses arising from material properties.

As a result of advances (c) given above the finite strip method can now be applied advantageously in the analysis of cable-stayed bridges (19). As a result of advances (d) the finite strip method is being applied to develop simplified methods of analysing slab-on-girder bridges as the ultimate limit state is approached.

Research is currently underway (e.g.1) to use the finite strip method for the analysis of soil-steel structures. For this application, the strips are curved in elevation.

Finite Element Methods

In recent years, research at two Canadian universities, namely McGill and McMaster, has been ongoing for the finite element analysis of reinforced concrete and prestressed concrete components as loading approaches the ultimate limit state. The resulting non-linear behaviour of the structure is usually being represented by a deteriorating but peacewise linear formulation. Examples of the application of this method to bridges are given in (41) and (42).

Non-Linear Methods of Load Distribution

Considerable research is being done in different parts of Canada to investigate the transverse load distribution behaviour of bridges at the ultimate limit state (see e.g. 16, 36, 41 and 44). Because of this work, it has been proposed that in the forthcoming third edition of the OHBDC, an 8% redistribution of live load longitudinal moments in the transverse direction be permitted for slab-on-girder bridges with girders of compact steel sections.

BRIDGE TYPES AND COMPONENTS

Much research in bridge engineering is best described in relation to the type of bridge or bridge component concerned. There now follows an account of certain Canadian activities in bridge research, categorized in this way.

Deck Slabs

Concrete deck slabs of slab-on-girder bridges subjected to concentrated wheel loads have traditionally been designed for pure bending. Research conducted in Ontario, particularly at Queen's University and MTO, has conclusively shown that these components should be designed for the punching shear mode of failure. Designing these components for this mode of failure not only leads to a considerably saving of reinforcement but also to enhancement of their durability. Whilst initially there was a great deal of reluctance in accepting the method of design that resulted from this research, subsequent experience including the construction of several bridges designed in this manner, has now led to wide acceptance of its validity. More or less upto date research on deck slabs has been summarized in (9). Current research in deck slabs includes work on partially prestressed deck slabs which is currently underway at Queen's University.

Soil-Steel Structures

Considerable research has been conducted in Canada in recent years on soil-steel structures which are also referred to as buried corrugated metal plate pipes. The research consists of analytical and laboratory studies as well as monitoring and testing of in-service structures. The most intense research activity on soil-steel structures is at the University of Windsor(e.g. see 1, 20, 21, 24, 26, 31 and 32). Much of this work has helped formulate the OHBDC provisions for the design of soil-steel structures. Worthy of note in this work are the laboratory and analytical studies for developing a soil-steel structure which incorporates reinforced earth in its back-fill. Other work includes the determination of live load effects on the conduit walls, and their buckling capacities.

Some research has also been conducted on soil-steel structures at MTO (e.g. 7), at the Technical University of Nova Scotia (e.g. 4), and at Lakehead University (e.g. 22). Research has also been conducted at the University of Alberta on the strength of the bolted joints of corrugated plates. From the research it has been concluded that longitudinal joints become ductile if the bolts are staggered in two rows and if the bolts closer to the visible edge of the mating plates are placed in the valleys of the corrugations. In the other case, i.e. when the bolts closer to the visible edge are placed on the ridges, the joint becomes quite brittle when subjected to moments and thrusts.

Timber Bridges

Fundamental research on the properties of timber and the design values on strengths of different species of wood is being done at the University of British Columbia (e.g. see 35) Considerable research has been conducted in Ontario on the development of the means of rehabilitating existing deteriorated timber bridges and of new types of timber bridges. This research has led to the technique of transversely prestressing laminated wood decks. As described in (43), this technique has been used not only for the rehabilitation of existing deteriorated nailed laminated decks, but also for new construction. The concept of steel-wood composite bridges has also been developed recently in Ontario. It is shown in (11) that the live load carrying capacity of existing slab-on-girder bridges with steel girders and deteriorated concrete decks can be enhanced considerably by replacing the existing deck with a composite prestressed laminated wood deck in which the laminates run parallel to the girders. In this system, the composite action between the prestressed deck and steel girders is achieved by drilling large enough holes in the deck to accommodate clusters of shear connectors attached to the flanges of girders and then filling the holes with expansive concrete.

Work is also underway in Ontario to reappraise the philosophy of designing timber bridges. The reappraisal relates mainly to designing bridges as systems of components as distinct from designing them on a component by component basis.

Prestressed Steel Girder Bridges

Considerable work has been done in recent years in Iowa on the rehabilitation of steel girder bridges by prestressing applied to the girders. Independent of this work, a similar and simultaneous research activity has been underway for some time at Concordia University. This activity includes both experimental and analytical investigation of the prestressing technique for simply supported as well as continuous span steel girder bridges. It has been demonstrated through work reported in (47, 48, 49, 50 and others) that prestressing of steel girders of existing bridges is an economical means of rehabilitating structurally deficient bridges with steel girders. Although the studies conducted at Concordia University have not so far dealt with truss bridges, it has been suggested that the prestressing techniques can also be applied with advantage, to these bridges. The research work has led to a book entitled 'Prestressed Steel Structures' written by M.S. Troitsky; this book is currently in the press.

Steel Box Girders

A study of field measurements was undertaken at McMaster University to establish out-of-plane imperfections in the webs and bottom flanges of several steel box girders bridges in Canada (33). It was found that measured imperfections in the plates of Canadian manufactured box girders bridges, were not significantly different from those observed in several European countries. Work is also underway at McMaster University to study the influence of these imperfections on the ultimate strength of box girders (34, 46).

Work of similar nature is also ongoing at Concordia University. In this work, the buckling strength of steel box girders is being investigated both analytically and experimentally by taking account of initial geometric imperfections and residual welding stresses.

In connection with multi-spine steel box girders, mention should also be made of research at Waterloo University which relates to bracing between the spines both during and after construction.

Inverted T-Girders

Reinforced concrete girders with inverted T-girders are sometimes used as bentcap girders to support precast stringers of bridges. Experimental studies on determining optimum reinforcement for these components were conducted at the University of Texas at Austin. However, extensive reanalysis of the experimental data was done recently at Lakehead University. This work which is reported in (39) has led to a series of recommendations for the detailing of reinforcement in inverted T-girders.

PARTIALLY PRESTRESSED CONCRETE

Although partially prestressed concrete has been used for bridges, in some European countries for a long time, it has not been accepted for bridges in North America until recently. Ontario has taken the lead by permitting in the forthcoming third edition of OHBDC the use of partially prestressed concrete in bridges. This permission to use partially prestressed concrete came about after an extensive study of the state-of-the-art conducted at Queen's University (12). It is noted that research is ongoing currently at Queen's university on the subject of partially prestressed concrete; this research deals with cracking, prestress losses and fatigue behaviour of partially prestressed flexural components.

Following the decision to permit the use of partially prestressed concrete in bridges, the design provisions for reinforced and fully prestressed concrete components have been integrated with those for partially concrete structures. The resulting set of provisions of the forthcoming third edition of OHBDC correspond to a logical transition between reinforced and fully prestressed concrete.

Shear Capacity of Beams

The University of Toronto has a long standing tradition of extensive research on shear capacity of reinforced and fully prestressed concrete components. Research continues in that university to extend the truss analogy method of determining shear capacity to concrete beams with varying degrees of prestressing. A project entitled, 'Determination and Evaluation of Reinforced concrete, Partially Prestressed Concrete and Prestressed Concrete Bridges for Shear', sponsored by MTO, is in the final stages of completion at the University of Toronto. It is expected that the work of this project will be useful in formulating the shear design provisions for the forthcoming third edition of OHBDC.

Cracking and Prestress Losses

Considerable work has been done at the University of Calgary with respect to cracking and prestress losses in partially prestressed concrete structures. Recent work undertaken at this university includes the development of computer programs dealing with concrete bridges. One computer program is concerned with the analysis required for design check of serviceability of plane frames (23); it can be used for any structures, built in stages,such as segmental bridges. The second program performs design checks for serviceability and ultimate limit states for both grids and continuous beams(40).

BRIDGE TESTING

There is no better way for a bridge engineer to understand the short comings of the mathematical models used for design or evaluation of bridges than to investigate the behaviour of bridges through field testing. The MTO has for many years conducted a program of field testing of highway bridges which has included both static and dynamic tests. The dynamic tests were necessarily of the behaviour type, but static tests included not only behaviour test but also proof tests and a few ultimate load tests. More than 225 bridges have been tested in Ontario in recent years.

As a result of participation in this testing program, many lessons have been learnt and it has become obvious that design analysis customarily practiced can be in error in not one, but many different ways. Indeed, it is fair to say that virtually every bridge test has a surprise in store bringing to notice some significant aspect of bridge behaviour which had been more or less completely ignored in the evaluation analysis of the bridge.

Bridge testing serves several functions at the same time: (a) It is used to evaluate realistically the load carrying capacities of existing bridges; (b) it provides invaluable data for bridge research; and (c) it generates bridge research by identifying those areas in bridge behaviour which require investigation.

Results of bridge tests conducted in Ontario have been published extensively in the literature. In (6) it has been shown with the help of specific examples that the correlation between actual and predicted responses is made difficult by several factors in a bridge which, in practice, cannot be assessed realistically without a field test.

Many lessons have been learnt regarding the behaviour of bridges from the field testing program of Ontario, which had been at one time the only one of its kind in the world. A similar program has been started recently in the State of Florida.

COMPUTER GRAPHICS

It is usual to form an opinion regarding the appearance of a bridge from construction drawings or form physical models. The former approach gives quite subjective results and the latter is expensive and inflexible in that minor changes cannot be readily incorporated in the physical model. Research is currently underway at the Technical University of Nova Scotia (TUNS) to model bridges graphically on the computer from construction drawings. Such representations not only provide views of the bridge from different vantages points, but can also be a useful aid for construction. This aspect of computer graphics has been over-looked by researchers as it is considered to be in the realms of demonstration and not research; and practicing engineers have shunned it because its application has not been demonstrated. TUNS have taken the lead by applying it to several existing bridges and demonstrating that aesthetics of bridges can be studied realistically through computer graphics.

BEARINGS

An MTO sponsored research project is underway at Queen's University into bridge bearings. The first phase of this project dealing with the state-of-the art in TFE bridge bearings is reported in (15).

It should be mentioned here that results of bridge testing have shown that even apparently functioning elastomeric bearings have a considerable resistance to horizontal movement of the bottom flanges of girders. This resistance is large enough to reduce the moments due to live loads by as much as 10%.

CATHODIC PROTECTION OF BRIDGES

It is well known that deicing salts used on roads cause the embedded steel reinforcement in concrete to corrode. This corrosion leads to concrete spalling and general deterioration of concrete structures. New structures are protected against the effects of deicing salts by epoxy-coating the reinforcement. However, after the chloride ions have penetrated the concrete of older structures without epoxy-coated reinforcement, the process of deterioration cannot be arrested by simply replacing the deteriorated concrete with new concrete. For concrete structures saturated with chloride ions, the only feasible method for stopping the corrosion of steel is cathodic protection.

The technology for the cathodic protection of reinforcement in concrete is relatively new. Ontario is the only jurisdiction where this procedure is used as a matter of routine. This technique has been applied since 1978 to concrete deck slabs. Since 1981, Ontario has initiated a viable means of providing cathodic protection to even concrete substructures. Upto date details of research conducted in Ontario in the field of cathodic protection of concrete bridges is provided in (38).

CONDITION SURVEY OF DECKS

Due to the effect of deicing salts, the bridge components which exhibit the most rapid deterioration in bridges, are the concrete decks. A survey of the extent of corrosion in these components is made difficult by the fact that much of the deterioration can remain hidden, especially on asphalt-covered decks. Ontario has developed a system for rapid and automatic collection of data on the condition of bridge decks. This system is a combination of radar and thermography, and is known by the acronym of DART (Deck Assessment by Radar and Thermography). Details of DART are provided in (37). This system is now used extensively to provide condition survey data on bridge deck slabs for use in bridge management system and to establish priorities for rehabilitation.

CLOSING REMARKS

It is important to note that much of the research carried out in Canada has direct relevance to bridge design codes, in particular to the OHBDC which is gradually gaining wide acceptance not only as a code to design to, but also as a model for other codes. Because of direct relevance to a design code, the Canadian research can be seen to be immediately useful for the design and evaluation of bridges.

This paper was written at the invitation of Wayne Klaiber at very short notice, as a result of which not all research workers in the field of bridge engineering in Canada could be contacted. It is quite likely, therefore, that we may have inadvertently left out from the paper mention of some relevant research. We are grateful, nevertheless, to the following of our Canadian research colleagues for providing us with the details of their and their colleagues' work: G. Abdel-Sayed, B. deV. Batchelor, D. Beaulieu, M.S. Cheung, A.A. Mufti, A. Ghali, R. Green, R.M. Korol, D.G. Manning, S.A. Mirza, A.G.Razaqpur, P. Smith and M.S. Troitsky. It is noted that the research mentioned herein is limited to that which has been reported since 1982. Previous research has, indeed, helped to shape bridge engineering as it is today in Canada, but it is not being considered here as being current.

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THE NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM AND BRIDGE ENGINEERING RESEARCH

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SYNOPSIS

Sponsored by participating members of the American Association of State Highway and Transportation Officials (AASHTO), in cooperation with the Federal Highway Administration, the Transportation Research Board's National Cooperative Highway Research Program (NCHRP) was created in 1962 as a means to accelerate research on acute problem areas that affect highway planning, design, construction, operations, and maintenance nationwide. The following paper provides a brief overview of the program and a description of the wide-ranging NCHRP activities related to bridge engineering.

Introduction

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

In recognition of these needs, the American Association of State Highway Officials (AASHO)---which was reorganized in 1973 to become the American Association of State Highway and Transportation Officials (AASHTO)--initiated a national program highway research in 1962. This program, known as the National Cooperative Highway Research Program, is supported on a continuing basis by funds from the member states of AASHTO and receives the full cooperation and support of the Federal Highway Administration (FHWA).

Each year, AASHTO considers only those problems submitted through officially sanctioned sources which include the chief administrative officers of the member departments, chairmen of selected AASHTO committees, and the Federal Highway Administrator. Research problems to be conducted within the Program are selected on a yearly basis by the AASHTO Standing Committee on Research.

The Program is administered by the Transportation Research Board (TRB). The Board is a unit of the National Research Council (NRC), which serves both the National Academy of Sciences and the National Academy of Engineering. Research projects addressed to the AASHTO-selected problems are defined and developed by an NRC-appointed NCHRP Project Panel assembled specifically for a given project. The Project Panel is composed of experts in the field of the subject research and includes a balanced representation from State transportation departments, universities, consulting firms, private research organizations, and industry. The panel reviews all proposals accepted in response to a project statement (equivalent to a request for proposals) developed by the panel and selects the proposal that demonstrates the best understanding of the problem and chance for successful completion of the project. The panel monitors all technical activities of the agency through the end of the project.

The NCHRP is a program of contract research--it does not operate on a grant basis. Proposals can be received only in response to announced project statements because each year's funds are earmarked in their entirety for research problems specified by the program sponsor--AASHTO.

The needs for highway research are many, and the National Cooperative Highway Research Program makes significant contributions to the solution of transportation problems of mutual concern to many responsible groups. In doing so, the Program operates complementary to, rather than as a substitute for or duplicate of, other highway research programs.

NCHRP Bridge Engineering Research

NCHRP research activities cover a wide range of transportation-related problem areas. The Program handles problems related to administration, transportation planning, design, construction, materials, maintenance, traffic operations and traffic planning.

NCHRP research funding averaged more than $3 million annually for the first 20 years of the program and increased to approximately $5-1/2 million starting with the 1985 fiscal year (due to the five-cents-per-gallon federal gasoline tax increase). Since 1980, approximately one-third of these funds have been allocated for studies of problems in the area of bridge engineering due primarily to the concerns about the state of our nation's aging bridge inventory.

Within the field of bridge engineering, NCHRP research has also been broad-ranging. Historically, research projects have covered many areas related to bridge design, materials, construction, rehabilitation, and repair. Problems associated with bridge decks, elastomeric bearings, bridge management, wheel-load distribution, fatigue, traffic barriers and bridge rails, deicing salts, underwater inspection, and paint removal have all been researched in the program at various times. In the mid-1980s, the focus of the NCHRP's bridge engineering research was on evaluation of existing structures.

Several methods are available for reporting on the progress and outcome of NCHRP research. The two major report series, NCHRP Reports and NCHRP Syntheses, provide documentation on the results of completed projects and have had a widespread impact on highway practice. The NCHRP Research Results Digest provides a means for communicating the preliminary results of ongoing research and for summarizing the major accomplishments of completed research.

Research Results Digest 167, dated May 1988, is reprinted on the following pages and provides a summary of the status of all NCHRP bridge engineering research since the inception of the Program. This bridge engineering Digest is updated approximately once every two years and can be referred to, in the interim, for its information on completed research and the availability of reports.

Since its inception in 1962, the National Cooperative Highway Research Program (NCHRP) has included numerous studies of interest to bridge engineers. In recent years, there has been a growing national awareness of bridge problems, and a substantial number of bridge research projects have been referred to the NCHRP by the program sponsors, the American Association of State Highway and Transportation Officials (AASHTO). In fact, since 1980, more than one-third of the NCHRP's funds have been allocated for studies of problems in the area of bridge engineering.

Many of these studies have been directed toward development of improved methods of design and construction, with the ultimate goal of modifying the AASHTO *Standard Specifications for Highway Bridges*. In the early- to mid-1980's, the focus of NCHRP research shifted toward solving problems in evaluation, repair, or rehabilitation of existing bridges. More recently, NCHRP's bridge research has included numerous projects that will result in recommended revisions to major sections of the AASHTO bridge specifications, incorporating the results of the bridge research completed during the last decade.

AASHTO DESIGN SPECIFICATIONS

Since initial adoption more than 50 years ago, the AASHTO *Standard Specifications for Highway Bridges* has been modified annually by the AASHTO Highway Subcommittee on Bridges and Structures. These specifications are relied upon by engineers in state highway agencies, consulting firms, and other organizations responsible for design, construction, and maintenance of bridges. Because of the piecemeal development of the current specifications, extra care is required to avoid inconsistencies, fragmentation, and internal conflicts as individual sections of the specifications are revised each year. This problem is compounded by the fact that a comprehensive commentary is not available to record the origin and clarify the intent of key provisions of the specifications.

At the request of the AASHTO Highway Subcommittee on Bridges and Structures, research was initiated on NCHRP Project 12-33 in mid-1988 with the objective of developing recommended bridge design specifications and commentary that can be considered for adoption by the Subcommittee. The new specifications, based on the load and resistance factor design (LRFD) philosophy, are expected to draw heavily from recent developments in bridge design practice throughout the world as well as from recently completed and current bridge research. It is estimated that completely new LRFD-based bridge specifications and a commentary can be developed in 42 months at an estimated cost of $1.6 million.

CONSTRUCTION

Several other major specification-related studies were also requested by the AASHTO Highway Subcommittee on

Bridges and Structures. NCHRP Project 12-34, "Update of AASHTO *Standard Specifications for Highway Bridges*: Division II-- Construction," was initiated in late 1987 with the objective of providing a complete overhaul to the present Division II--Construction specification in the AASHTO *Standard Specifications*. These specifications should reflect the latest state of the art in proven construction practice. As technology changes, it is important to have these changes reflected in the specifications. The changes that have occurred in Division II over the years have also been made on a piecemeal basis and no longer reflect current bridge construction practice. As a result, less than one-half of the states presently use the current Division II specification. NCHRP Project 12-34 will provide the basis for the required updating.

INSPECTION AND EVALUATION

The AASHTO *Manual for Maintenance Inspection of Bridges* is intended as a guide to provide uniformity in inspection and evaluation techniques for all bridges on public roads. Since the *Manual* was initially adopted by AASHTO in 1970, only minor changes and additions have been made to it. Many subsequent advances in analytical and practical techniques are being used in bridge design, construction, and evaluation, but they have not been reflected in the *Manual*.

NCHRP Project 12-23, "Recommended Revisions to the AASHTO *Manual for Maintenance Inspection of Bridges*," will start near the end of 1988. The objective of the project is to develop a revised *Manual for Maintenance Inspection of Bridges* that can be recommended to AASHTO for consideration for adoption. A thorough review and revision of the inspection and evaluation criteria that is based on current technology and recently completed and ongoing research will result in a better assessment of the condition and load capacity of existing bridges.

THIS DIGEST

This Results Results Digest supersedes Research Results Digest 155 published in February 1986. The purpose of the Digest is to outline for easy reference (see Tables 1 through 5) the status of, and inter-relationships between, all NCHRP research related to bridges. Included are projects under development, in progress, and completed. A listing of all related research reports is also provided.

Table 1 lists all NCHRP bridge engineering projects currently in progress. At the present time, more than 30 bridge-related studies are underway with a total research funding of more than $10 million. Included in Table 1 are the previously described NCHRP Projects 12-33 and 12-34. Details on each of the projects in this table can be found in the NCHRP *Summary of Progress Through 1987*.

Research projects in the developmental stage or expected to start in the near future are covered in Table 2. NCHRP Project 12-23, described above, is included in this table.

Table 3 provides a cross-reference between pending, active, and recently completed NCHRP research projects and the previously described studies (NCHRP Projects 12-23, 12-33, and 12-34) relevant to bridge design, construction, and inspection and rating. In some cases, individual NCHRP projects will produce results related to more than one specification. For example, NCHRP Project 10-20 on elastomeric bearings will produce recommendations for changes to both the design specifications and construction specifications.

A chronological arrangement of all NCHRP publications on bridge research is provided in Table 4. Some 68 relevant publications in the NCHRP Report Series are contained in Table 4(a). Several of the earlier reports are included for the sake of completeness, and should no longer be considered to be thorough, up-to-date treatments of the subject matter. NCHRP Syntheses of Highway Practice that are concerned with bridge problems are listed in Table 4(b). These reports emanate from NCHRP Project 20-5, "Synthesis of Information Related to Highway Problems." Table 4(c) includes NCHRP Research Results Digests on studies of bridge problems.

Table 5 summarizes the availability of uncorrected agency final reports which have not been published by the NCHRP.

OBTAINING PUBLICATIONS

Copies of the publications in Table 4 and microfiche of the uncorrected agency reports listed in Table 5 can be obtained from the Publications Office, Transportation Research Board, 2101 Constitution Avenue, NW, Washington, DC 20418. A check or money order payable to the *Transportation Research Board* must accompany orders totaling $20.00 or less.

NCHRP research covers a wide range of problem areas related to design, construction, and maintenance of bridges. Nevertheless, the studies included in Tables 1 through 5 comprise only a small portion of all bridge research carried out in the United States in recent years. A more comprehensive listing of current and planned research, including FHWA-sponsored contracts and state Highway Planning and Research (HP&R) studies, can be found in the documentation for the FHWA's Nationally Coordinated Program of Highway Research, Development, and Technology (NCP), which may be obtained from the office of Mr. Charles F. Galambos, Chief, Structures Division, Office of Research, Development & Technology, HNR-10, Federal Highway Administration, 6300 Georgetown Pike, McLean, Va 22101 (703/285-2087).

TABLE 1 - RESEARCH IN PROGRESS

Project Number	Title	Research Agency	Completion Date
2-16	Relationships Between Vehicle Configurations and Highway Design	Transportation Research Board	3/90
3-36	Development of a Low-Cost Bridge Weigh-In-Motion System	Bridge Weighing Systems, Inc.	8/89
3-39	Evaluation and Calibration Procedures for Weigh-In-Motion Systems	Texas A & M University	12/90
4-15	Corrosion Protection of Prestressing Systems in Concrete Bridges	Wiss, Janney, Elstner Assoc.	*
10-13/1	Ultrasonic Measurement of Weld Flaw Size	The Welding Institute (UK)	*
10-20/2	Elastomeric Bearings Design, Construction, and Materials	University of Washington	6/89
10-22/1	The Performance of Weathering Steel in Bridges	Sheladia Associates, Inc.	*
10-29	Anchorage Zone Reinforcement for Post-Tensioned Concrete Girders	University of Texas at Austin	10/89
10-30(3)	Nondestructive Methods for Field Inspection of Embedded or Encased High Strength Steel Rods and Cables	University of Manchester (UK)	10/89
10-31	Acceptance Criteria for Steel Bridge Welds	Materials Research Laboratory	1/89
10-35	Fatigue Behavior of Welded and Mechanical Splices in Reinforcing Steel	Wiss, Janney, Elstner Assoc.	5/90
10-36	Evaluation of Weldments Incorporating Backing Materials	Arctec Canada Limited	5/91
12-15(5)	Fatigue Behavior of Variable Loaded Bridge Details Near the Fatigue Limit	Lehigh University	1/90
12-18A	Assessment of an Integrated Bridge Design System	Engineering Computer Corp.	*
12-26	Distribution of Wheel Loads on Highway Bridges	Imbsen & Associates, Inc.	*
12-27	Welded Repair of Cracks in Steel Bridge Members	The Welding Institute (UK)	8/88
12-28(1)	Load Capacity Evaluation of Existing Bridges	Case Western Reserve Univ.	9/89
12-28(2)/1	Bridge Management Systems	ARE, Inc.	11/89
12-28(6)	Distortion-Induced Fatigue Cracking in Steel Bridges	Lehigh University	10/88
12-28(7)	Guidelines for Evaluating Corrosion Effects in Existing Steel Bridges	Modjeski and Masters	2/89
12-28(10)	Guidelines for Determining Redundancy in Steel Bridges	Lehigh University	9/88
12-28(11)	Development of Site-Specific Load Models for Bridge Rating	Imbsen & Associates, Inc.	2/89
12-28(12)	Inelastic Rating Procedures for Steel Beam and Girder Bridges	University of Minnesota	12/89
12-28(13)	Nondestructive Load Testing for Bridge Evaluation and Rating	Raths, Raths, & Johnson, Inc.	4/89
12-29	Design of Simple-Span Precast Prestressed Bridge Girders Made Continuous	Construction Technology Laboratories/PCA	*
12-30	Fatigue of Cables in Cable-Stayed Bridges	Acer/Freeman Fox Ltd.	8/88
12-31	Notch Toughness Variability in Bridge Steel Plates	University of Texas at Austin	3/90
12-33	Development of a Comprehensive Bridge Specification and Commentary	Modjeski and Masters	1/92
12-34	Update of AASHTO Standard Specifications for Highway Bridges: Division II - Construction	Imbsen & Associates, Inc.	10/89
12-35	Recommended Specifications for the Design of Foundations, Retaining Walls, and Substructures	D'Appolonia	7/89
15-11	Computer-Aided Analysis of Highway Encroachments on Mobile Boundary Streams	Simons & Associates, Inc.	4/90
17-8	Traffic Barrier and Control Treatments for Restricted Work Zones	Texas A & M University	6/91
20-5	Synthesis of Information Related to Highway Problems Topic 16-01, Bridge Inspection Practices - Equipment, Staffing, and Safety Topic 18-03, Bridge Approach Design and Construction Practices Topic 18-04, Treatment of Problem Foundations for Highway Embankments Topic 19-01, Computer-Aided Design and Drafting Systems	Transportation Research Board	Varies
24-3	Laboratory Evaluation of Piles Installed with Vibratory Drivers	University of Houston	9/88
24-4	Load Factor Design Criteria for Highway Structure Foundations	Virginia Tech	6/90

* Final report in review.

TABLE 2 - PENDING RESEARCH

Project Number	Title	Funds Available	Expected Start
10-20	Elastomeric Bearings Design, Construction, and Materials (Phase IV)	250,000	Early 1989
10-29	Anchorage Zone Reinforcement for Post-Tensioned Concrete Girders (Phase II)	250,000	Late 1988
12-23	Recommended Revisions to the AASHTO Manual for Maintenance Inspection of Bridges	200,000	Early 1989
12-26	Distribution of Wheel Loads on Highway Bridges (Phase II)	200,000	Late 1988
22-7	Update of "Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances"	200,000	Early 1989
22-8	Evaluation of Performance Level Selection Criteria for Bridge Railings	200,000	Early 1989
20-5	Synthesis of Information Related to Highway Problems		
	Topic 20-07 - Concrete Bridge-Deck Removal Procedures		
	Topic 20-09 - Removal of Toxic Paints from Bridges		
	Topic 20-10 - Repair and Replacement of Highway Culverts		
	Topic 20-12 - Latex-Modified Concrete		
	Topic 20-19 - Surface Preparation for Concrete Repairs		
20-7	Research for AASHTO Standing Committee on Highways		
	Task 34 - AASHTO/AWS Bridge Welding Code		
24-5	Downdrag on Bitumen-Coated Piles	200,000	Mid 1988

TABLE 3 - PENDING, ACTIVE, AND RECENTLY COMPLETED NCHRP RESEARCH PROJECTS AND THEIR EFFECTS ON AASHTO BRIDGE SPECIFICATIONS

Project	Description	Design[a]	Construction[b]	Inspection/Rating[c]
10-20	Elastomeric bearings (unconfined and pot)	X	X	
10-22	Weathering steel	X		
10-29	Post-tensioned girder anchorage zone	X		
10-35	Fatigue strength of rebar splices	X		
10-36	Fatigue evaluation of backing bar weldments	X		
12-15(5)	High-cycle variable amplitude fatigue	X		
12-25	Fatigue and fracture of riveted steel bridges			X
12-26	Wheel load distribution	X		X
12-28(1)	Load capacity evaluation of existing bridges			X
12-28(3)	Fatigue evaluation and design procedures	X		X
12-28(5)	Inspection of reinforced concrete bridge components			X
12-28(6)	Distortion induced fatigue evaluation and repair	X	X	X
12-28(7)	Evaluation of corrosion			X
12-28(10)	Redundancy	X		X
12-28(11)	Site-specific load models for bridge evaluation			X
12-28(12)	Inelastic rating for steel girder bridges			X
12-28(13)	Nondestructive load testing			X
12-29	Simple-span precast girder connections	X	X	
12-30	Cable fatigue in cable-stayed bridges	X		
12-31	Notch toughness variability in bridge steels	X		X
12-32	Bridge deck protection		X	
12-35	Update of existing foundation and substructure specifications	X		
20-7/32	Segmental bridge specifications	X	X	
22-8	Bridge railing	X		
24-3	Piles installed with vibratory drivers		X	
24-4	Load factor design for foundations, substructures, and retaining walls	X		
24-5	Downdrag on piles	X		

[a] Projects that will provide input into NCHRP Project 12-33 (LRFD-based specification and commentary)
[b] Projects that will provide input into NCHRP Project 12-34 (Update construction specifications)
[c] Projects that will provide input into NCHRP Project 12-23 (Revision of Manual for Maintenance Inspection)

TABLE 4 - AVAILABLE NCHRP SERIES PUBLICATIONS

No.	Title	Proj.	Research Agency	Pgs.	Cost	Year
	(a) NCHRP Reports					
1	Evaluation of Methods of Replacement of Deterioriated Concrete in Structures	6-8	Bertram D. Tallamy Associates	56	*	1964
4	Non-Chemical Methods of Snow and Ice Control on Highway Structures	6-2	Roy Jorgensen and Associates	74	*	1964
16	Protective Coatings to Prevent Deterioration of Concrete by Deicing Chemicals	6-3	Battelle Memorial Institute	21	*	1965
23	Methods for Reducing Corrosion of Reinforcing Steel	6-4	Battelle Memorial Institute	22	*	1966
74	Protective Coatings for Highway Structural Steel	4-6	Steel Struc. Painting Council	64	2.80	1969
74A	Protective Coatings for Highway Structural Steel--Literature Survey	4-6	Steel Struc. Painting Council	275	*	1969
74B	Protective Coatings for Highway Structural Steel--Current Highway Practices	4-6	Steel Struc. Painting Council	102	*	1969
80	Oversize-Overweight Permit Operation on State Highways	2-10	Roy Jorgensen and Associates	120	*	1969
83	Distribution of Wheel Loads on Highway Bridges	12-2	Iowa State University	56	*	1970
86	Tentative Service Requirements for Bridge Rail Systems	12-8	Texas A & M University	62	*	1970
90	Protection of Steel in Prestressed Concrete Bridges	12-5	University of Denver	86	4.00	1970
101	Effect of Stress on Freeze-Thaw Durability of Concrete Bridge Decks	6-9	University of Illinois	70	*	1970
102	Effect of Weldments on the Fatigue Strength of Steel Beams	12-7	Lehigh University	114	5.40	1970
105	Dynamic Pavement Loads of Heavy Highway Vehicles	15-5	General Motors Corporation	94	*	1970
106	Revibration of Retarded Concrete for Continuous Bridge Decks	18-1	University of Illinois	67	*	1970
109	Elastomeric Bearing Research	12-9	Battelle Memorial Institute	53	*	1970
116	Structural Analysis and Design of Pipe Culverts	15-3	Northwestern University	155	*	1971
141	Changes in Legal Vehicles Weights and Dimensions: Some Economic Effects on Highways	19-3	Wilbur Smith and Associates	184	*	1973
147	Fatigue Strength of Steel Beams with Welded Stiffeners and Attachments	12-7	Lehigh University	85	4.80	1974
149	Bridge Rail Design--Factors, Trends, and Guidelines	12-8	Texas A & M University	49	4.00	1974
153	Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances	22-2	Southwest Research Institute	19	3.20	1974
163	Design of Bent Caps for Concrete Box-Girder Bridges	12-10	Portland Cement Association	124	6.80	1976
164	Fatigue Strength of High-Yield Reinforcing Bars	4-7	Portland Cement Association	90	5.60	1976
165	Waterproof Membranes for Protection of Concrete Bridge Decks--Laboratory Phase	12-11	Materials Research and Development	70	4.80	1976
180	Cathodic Protection for Reinforced Concrete Bridge Decks	12-13	USS Engineers and Consultants	135	*	1977
181	Subcritical Crack Growth in Steel Bridge Members	12-14	U. S. Steel Corporation	82	5.60	1977
182	Economic Evaluation of Ice and Frost	6-11	Midwest Research Institute	73	4.80	1978
188	Fatigue of Welded Steel Bridge Members Under Variable Amplitude Loadings	12-12	U. S. Steel Corporation	113	6.40	1978
190	Use of Polymers in Highway Concrete	18-2	Lehigh University	77	*	1978
198	State Laws and Regulations on Truck Size and Weight	20-16	R. J. Hansen Associates	117	7.20	1979
201	Acceptance Criteria for Electroslag Weldments in Bridges	10-10	U. S. Steel Corporation	44	5.20	1979
203	Safety at Narrow Bridge Sites	20-7 (Task 7)	Texas A&M University	63	6.00	1979
204	Bridge Deck Joint-Sealing Systems--Evaluation and Performance Specification	10-11	Howard Needles Tammen & Bergendoff	46	5.60	1979
206	Detection and Repair of Fatigue Damage in Welded Highway Bridges	12-15 & 12-15(2)	Lehigh University	85	6.80	1979
222	Bridges on Secondary Highways and Local Roads--Rehabilitation and Replacement	12-20	University of Virginia	132	9.20	1980
226	Damage Evaluation and Repair Methods for Prestressed Concrete Bridge Members	12-21	G. O. Shanafelt & W. B. Horn	66	7.20	1980
227	Fatigue Behavior of Full-Scale Welded Bridge Attachments	12-15(3)	Lehigh University	47	6.40	1980
230	Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances	22-2(4)	Southwest Research Institute	42	6.00	1981
234	Galvanic Cathodic Protection for Reinforced Concrete Bridge Decks--Field Evaluation	12-13A	Portland Cement Association	64	6.80	1981
239	Multiple-Service-Level Highway Bridge Railing Selection Procedures	22-2(3)	Southwest Research Institute	161	10.40	1981
240	A Manual to Determine Benefits of Separating Pedestrians and Vehicles	20-10(2)	SRI International	56	7.20	1981
242	Ultransonic Measurement of Weld Flaw Size	10-13	The Welding Institute (UK)	76	8.00	1981
243	Rehabilitation and Replacement of Bridges on Secondary Highways and Local Roads	12-20	University of Virginia	46	6.80	1981

No.	Title	Proj.	Research Agency	Pgs.	Cost	Year
	(a) NCHRP Reports, Continued					
244	Concrete Sealers for Protection of Bridge Structures	12-19A	Wiss, Janney, Elstner & Assoc.	138	10.00	1981
248	Elastomeric Bearings Design, Construction, and Materials	10-20	University of Washington	82	8.40	1982
251	Assessment of Deficiencies and Preservation of Bridge Substructures Below the Waterline	10-16	Byrd, Tallamy, MacDonald and Lewis	80	8.40	1982
257	Long-Term Rehabilitation of Salt-Contaminated Bridge Decks	18-2(3)	Lehigh University	32	6.40	1983
265	Removal of Lead-Based Bridge Paints	10-23	Midwest Research Institute	72	8.00	1983
267	Steel Bridge Members Under Variable Amplitude Long Life Fatigue Loading	12-15(4)	Lehigh University	26	6.40	1983
271	Guidelines for Evaluation and Repair of Damaged Steel Bridge Members	12-17A	G. O. Shanafelt and W. B. Horn	64	7.60	1984
272	Performance of Weathering Steel in Bridges	10-22	Sheladia Associates, Inc.	164	12.00	1984
276	Thermal Effects in Concrete Bridge Superstructures	12-22	Engineering Computer Corp.	99	9.60	1985
278	Cathodic Protection of Concrete Bridge Substructures	12-19B	Wiss, Janney, Elstner Assoc.	60	8.40	1985
280	Guidelines for Evaluation and Repair of Prestressed Concrete Bridge Members	12-21(1)	G. O. Shanafelt and W. B. Horn	84	9.20	1985
286	Evaluation of Fatigue Tests and Design Criteria on Welded Details	12-15(5)	Lehigh University	66	8.40	1986
287	Load Distribution and Connection Design for Precast Stemmed Multibeam Bridge Superstructures	12-24	University of Washington	137	11.80	1986
289	Performance of Longitudinal Traffic Barriers	22-4	Southwest Research Institute	169	13.20	1987
290	Reinforcement of Earth Slopes and Embankments	24-2	Dames & Moore	323	40.00	1987
292	Strength Evaluation of Existing Reinforced Concrete	10-15/1	Engineering Computer Corp.	133	14.00	1987
293	Methods of Strengthening Existing Highway Bridges	12-28(4)	Iowa State University	114	12.00	1987
297	Evaluation of Bridge Deck Protective Strategies	12-32	University of Washington	80	12.00	1987
298	Performance of Elastomeric Bearings	10-20/1	University of Washington	100	12.00	1987
299	Fatigue Evaluation Procedures for Steel Bridges	12-28(3)	Case Western Reserve Univ.	94	11.20	1987
300	Bridge Management Systems	12-28(2)	ARE, Inc.	74	10.40	1987
301	Load Capacity Evaluation of Existing Bridges	12-28(1)	Case Western Reserve Univ.	104	11.60	1987
302	Fatigue and Fracture Evaluation for Rating Riveted Bridges	12-25	Lehigh University	**	**	1987
304	Condition Surveys of Concrete Bridge Components	12-28(5)	New Mexico State University	**	**	1987
306	Correlation of Bridge Load Capacity Estimates with Test Data	12-28(8)	University of Tennessee	**	**	1988
	(b) NCHRP Synthesis of Highway Practice (Project 20-5)					
2	Bridge Approach Design and Construction Practices	Topic #2	Transportation Research Board	30	*	1969
4	Concrete Bridge Deck Durability	#3	Transportation Research Board	28	*	1970
5	Scour at Bridge Waterways	#5	Transportation Research Board	28	*	1970
33	Acquisition and Use of Geotechnical Information	#5-04	Transportation Research Board	40	4.00	1976
41	Bridge Bearings	#6-09	Transportation Research Board	62	*	1977
42	Design of Pile Foundations	#5-04	Transportation Research Board	68	4.80	1977
44	Consolidation of Concrete for Pavements, Bridge Decks, and Overlays	#7-01	Transportation Research Board	61	4.80	1977
50	Durability of Drainage Pipe	#5-09	Transportation Research Board	37	3.60	1978
53	Precast Concrete Elements for Transportation Facilities	#8-05	Transportation Research Board	48	5.60	1978
57	Durability of Concrete Bridge Decks	#9-01	Transportation Research Board	61	*	1979
67	Bridge Drainage Systems	#10-06	Transportation Research Board	44	5.60	1979
68	Motor Vehicle Size and Weight Regulations, Enforcement, and Permit Organizations	#10-04	Transportation Research Board	45	6.00	1980
78	Value Engineering in Preconstruction and Construction	#11-02 & #11-03	Transportation Research Board	23	6.40	1981
82	Criteria for Evaluation of Truck Weight Enforcement Programs	#12-02	Transportation Research Board	74	7.20	1981
86	Effects of Traffic-Induced Vibrations on Bridge-Deck Repairs	#10-21	Transportation Research Board	40	6.80	1981
88	Underwater Inspection and Repairs of Bridge Substructures	#10-08	Transportation Research Board	77	7.60	1981
101	Historic Bridges: Criteria for Decision Making	#13-11	Transportation Research Board	84	8.00	1983
107	Shallow Foundations for Highway Structures	#12-06	Transportation Research Board	38	6.80	1983
108	Bridge Weight-Limit Posting Practice	#13-08	Transportation Research Board	30	6.40	1984
111	Distribution of Wheel Loads on Highway Bridges	#14-22	Transportation Research Board	22	7.20	1984
112	Cost-Effectiveness of Hot-Dip Galvanizing for Exposed Steel	#15-19	Transportation Research Board	28	7.20	1984
118	Detecting Defects and Deterioration in Highway Structures	#15-03	Transportation Research Board	52	8.00	1985

TABLE 4 - Continued

No.	Title	Proj.	Research Agency	Pgs.	Cost	Year
	(b) NCHRP Synthesis of Highway Practice (Project 20-5), Continued					
119	Prefabricated Bridge Elements and Systems	#15-10	Transportation Research Board	75	8.80	1985
123	Bridge Designs to Reduce and Facilitate Maintenance and Repair	#12-11	Transportation Research Board	65	8.40	1985
124	Use of Weigh-In-Motion Systems for Data Collection and Enforcement	#16-02	Transportation Research Board	34	7.60	1986
127	Use of Fly Ash in Concrete	#16-07	Transportation Research Board	66	8.40	1986
129	Freezing and Thawing Resistance of High-Strength Concrete	#16-05	Transportation Research Board	31	7.60	1986
136	Protective Coatings for Bridge Steel	#15-09	Transportation Research Board	**	**	1987
140	Durability of Prestressed Concrete Highway Structures	#15-02	Transportation Research Board	**	**	1988
141	Bridge Deck Joints	#16-10	Transportation Research Board	**	**	1988
	(c) NCHRP Research Results Digest					
81	Crash Testing and Evaluation of Attenuating Bridge Railing Systems	22-1A	Texas A & M University	10	1.00	1976
85	Bridge Deck Repairs	12-16	Battelle Columbus Laboratory	22	1.00	1976
115	NCHRP Research on the Durability of Reinforced Concrete Bridge Components	Var.	Transportation Research Board	6	1.00	1979
141	Liability of State Highway Departments for Defects in Design, Construction, and Maintenance of Bridges	20-6	Transportation Research Board	30	3.00	1983

Copies of publications listed in Table 4 can be obtained from the Publications Office, Transportation Research Board, 2101 Constitution Avenue, NW, Washington, DC 20418. A check or money order payable to the Transportation Research Board must accompany orders totaling $20.00 or less.

* Out of print - Available in microfiche from the Transportation Research Board. The cost is $5.95 per publication.
** In publication - Available during 1988.

TABLE 5 - UNCORRECTED AGENCY FINAL REPORTS

Proj. No.	Title	Research Agency	Availability*
4-14	Coating Systems for Painting Old and New Structural Steel	Georgia Institute of Technology	B
10-15	Structural Strength Evaluation of Existing Reinforced Concrete Bridges	Engineering Computer Corporation	B
10-30(1)	Nondestructive Methods for Field Inspection of Embedded or Encased High Strength Steel Rods and Cables	University of Manchester (UK)	A & B
10-30(2)	Nondestructive Methods for Field Inspection of Embedded or Encased High Strength Steel Rods and Cables	Southwest Research Institute	A & B
12-1	Deformation of Steel Beams Related to Permitted Highway Bridge Overloads	University of Missouri	B
12-4	Thermal Characteristics of Highway Bridges	Southwest Research Institute	B
12-6	Prediction of Permanent Camber of Bridges	University of Missouri	B
12-11/1	Waterproof Membranes for Protection of Concrete Bridge Decks	Materials Research & Development	B
12-15	Detection and Repair of Fatigue Cracking in Highway Bridges	Lehigh University	B
12-15(2)	Retrofitting Procedures for Fatigue-Damaged Full-Scale Welded Bridge Beams	Lehigh University	B
12-16	Influence of Bridge Deck Repairs on Corrosion of Reinforcing Steel	Battelle Columbus Laboratories	A & B
12-17	Evaluation of Repair Techniques for Damaged Steel Bridge Members	Battelle Columbus Laboratories	B
12-18	Development of an Integrated Bridge Design System	Multisystems, Inc.	A & B
12-19	Corrosion Control and Repair of Concrete Bridge Structures	Corrosion Eng. & Research Co.	B
12-19	Cathodic Protection of Concrete Bridge Structures	Corrosion Eng. & Research Co.	B
18-2(2)	Polymer Concrete in Highway Bridge Decks	Lehigh University	A & B
20-5	Welding and Inspection Practices in Bridge Fabrication (NCHRP Synthesis of Highway Practice, Topic 9-12)	Carl E. Hartbower	A & B
22-1	Concepts for Improved Traffic Barrier Systems	Walter W. White	B
22-1A	Testing and Evaluation of Bridge Rail Concepts	Texas A&M University	B
22-2(2)	Multiple Service Level Highway Bridge Railings--Performance and Design Criteria (Phase 1)	Southwest Research Institute	B
22-2(2)	Multiple Service Level Highway Bridge Railings--Development and Evaluation of Low-Cost Railing Systems (Phase II)	Southwest Research Institute	B

* A--A copy of the uncorrected draft of the agency's report may be obtained on a loan basis by request to the Director, Cooperative Research Programs, Transportation Research Board.
B--Available in microfiche from the Transportation Research Board. The cost is $5.95 per publication.

RESEARCH IN EUROPE ON COMPOSITE BRIDGES

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SYNOPSIS

The following paper contains an overview on the development and research on composite bridges currently under study in Europe. Some explanations are given on the research relating to the behavior of composite bridge girders and of stud shear connectors. Also, information relating to the durability of composite bridges with the use of external prestressing as an innovative bridge concept is given. Based on a theoretical study and field measurements, aspects of typical Swiss connection between steel girder and concrete slab are discussed and conclusions drawn.

EUROPEAN CONCEPTION OF COMPOSITE BRIDGES

The typical European (particularly in Germany, France, Great-Britain and in Switzerland) composite bridges for highways or railways have two main steel girders supporting a concrete deck slab. The cross-section of the steel girders is of slender type and such girders are used for span length in the range of 25 m (82 ft.) to 90 m (295 ft.). For longer span lengths or for bridges with horizontal curvature, open-top or closed-top steel box girders are used. For highway bridges, the width of the concrete slab is up to 15 m (49 ft.) and its thickness normally varies from 25 cm (10 in.), between the main girders, to 30 cm (12 in.) or 40 cm (16 in.) on the supports. The composite girders are continuous over the piers, and sufficient stud shear connectors are provided in the negative moment regions to enable the composite action. Prestress is often used in the longitudinal direction for the composite section or for the slab only. If there is no prestress, sufficient reinforcement is provided in the negative moment region for crack control. A typical highway composite bridge is shown on Fig. 1.

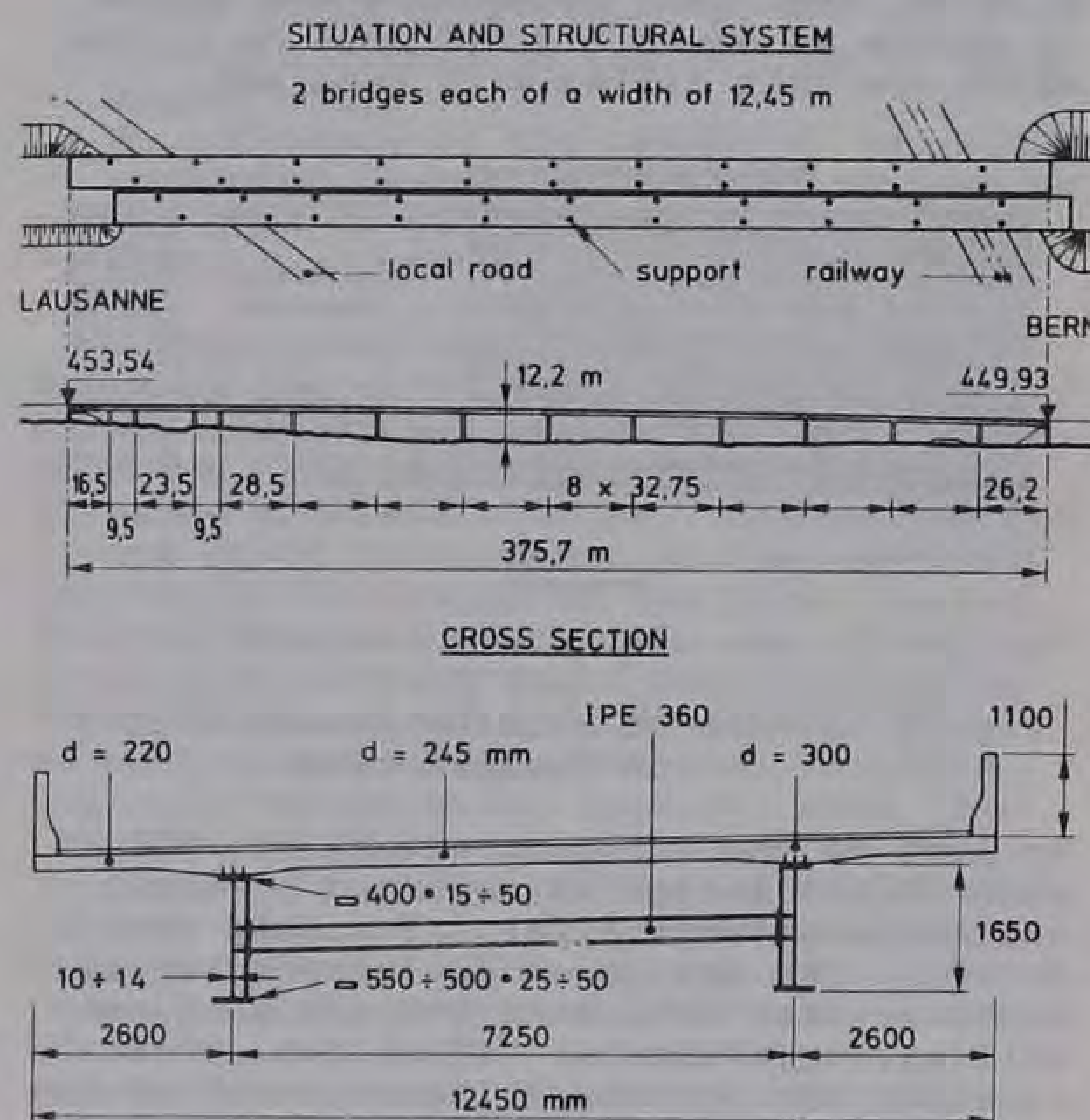

Fig. 1 Structural System and Cross-Section of the "Viaducs du Chene"

Steel beams of composite girders are in most cases completely welded. Transportable units are prefabricated in the shop and welded together on the site. Mobile cranes allow the most economical erection of the steel structure. Where it is not possible, and this is often the case, the steelwork is assembled behind one abutment and the complete steel structure is launched. The structure is usually unshored so that during erection and concreting, all loads are carried by the steel girders. Such loading cases as well as the different situations during the launching process need special attention during the design and the erection phases.

Three methods for the casting of the slab are available: cast in situ, precast slabs and sliding ribbon method (slip-decking method). The in situ slab provides the better slab from the point of view of the absence of transversal cracks due to the concreting procedure. The precast slabs and sliding ribbon methods are more rapid, and both require holes in the slab to create the composite action afterwards. The precast slabs, after fill-in joints, are normally prestressed longitudinally before the holes, into which the shear connectors are located, are filled. The sliding ribbon method of construction proceeds in the following manner: (1) Casting 20 m (66 ft.) to 30 m (98 ft.) length of slab on the steelwork near one end of the bridge, (2) Jacking it longitudinally to clear the formwork, (3) Casting another length again at its end, (4) Jacking again and so on until the whole deck is completed. Sliding friction is reduced by the method shown in Fig. 2. Where the slab is cast, the steel girders are provided with an additional top flange plate. During jacking, steel skate is placed in the shear connection hole so that the whole of the slab is supported on the skates which are lubricated with graphite. After completion of the sliding, the shear connectors are welded in groups at about 1 m (40 in.) on center, which are then filled with concrete.

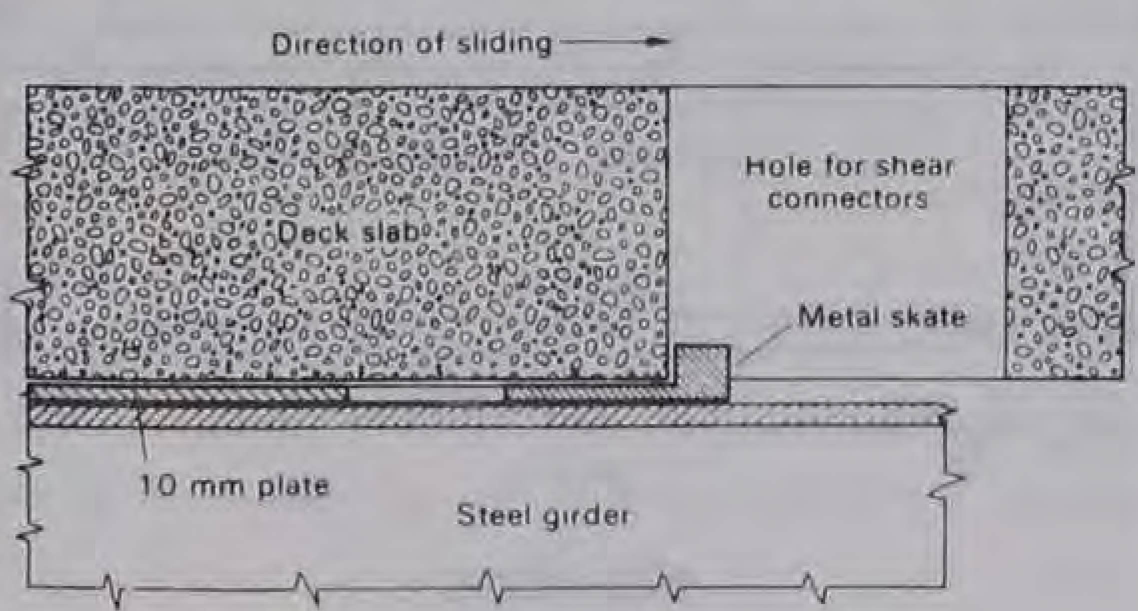

Fig. 2 Detail of Sliding Ribbon Method of Construction

In most of the European countries, allowable stress design is replaced by limit state design with a partial safety factor format. The design of any structural system must meet both strength and serviceability requirements. These new codes consider the post-buckling strength of the compressed plates. The ability of tension field action to enhance the capacity of slender web with transverse stiffeners is now recognized directly. Web capacity is a function of panel aspect ratio, boundary member flexural capacity and web slenderness. Where tension field action is utilized in transversally stiffened webs, the transverse stiffeners, which act as compressive posts in the simulated truss action, have to be designed directly for the ensuing compression. This requirement has influenced considerably the design of intermediate web stiffeners. Whereas stiffeners were previously designed to a simple limiting stiffness criterion, they are now required to be designed to have adequate strength and stiffness. With the new codes, the longitudinal stiffeners can often be suppressed, and we prefer now a thicker web with fewer transverse stiffeners. The steelwork becomes more simplified, thus the work in the shop is diminished. With progress of research in fatigue strength of steel structure, whose results are now in the codes, the details of construction have likewise changed. Figure 3 presents the development of the cross-section of composite bridges during the last twenty years(1).

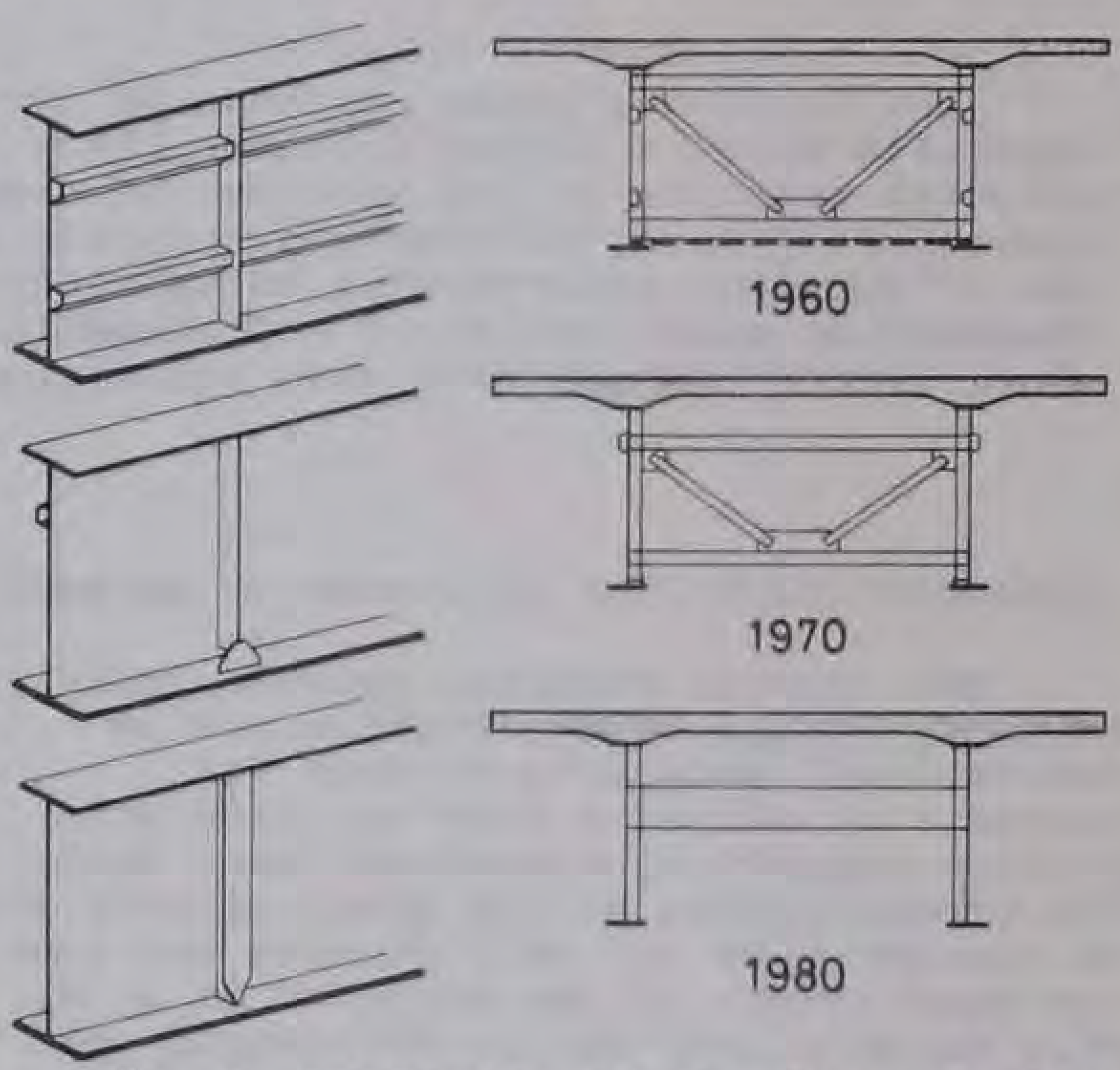

Fig. 3 Evolution of the Cross-Section of Composite Bridges

Other developments in composite bridges are actually going on. The composite cable-stayed bridge, whose competitive economy is again confirmed, and the composite truss bridges are examples of such new developments. Several composite truss bridges have been designed and built in recent years in Germany for the high speed railroad(2). These bridges must present high serviceability qualities regarding the comfort of the passengers. The maximum vertical acceleration does not exceed 2 m/s^2 (80 in/s^2), and no resonance effects occur up to speeds of 420 km/h (280 mph). These requirements limit the maximum deflection at midspan to L/2200 with L being the span length. Composite structures prove to be competitive under these circumstances. The composite truss bridge shown in Fig. 4 is a single span solution due to adverse foundation conditions. The truss is designed so that the neutral axis of the concrete slab and the intersection of the axes of the diagonals coincide to avoid eccentricities in the top flange during erection and in service. The top chord is embedded within the concrete top flange.

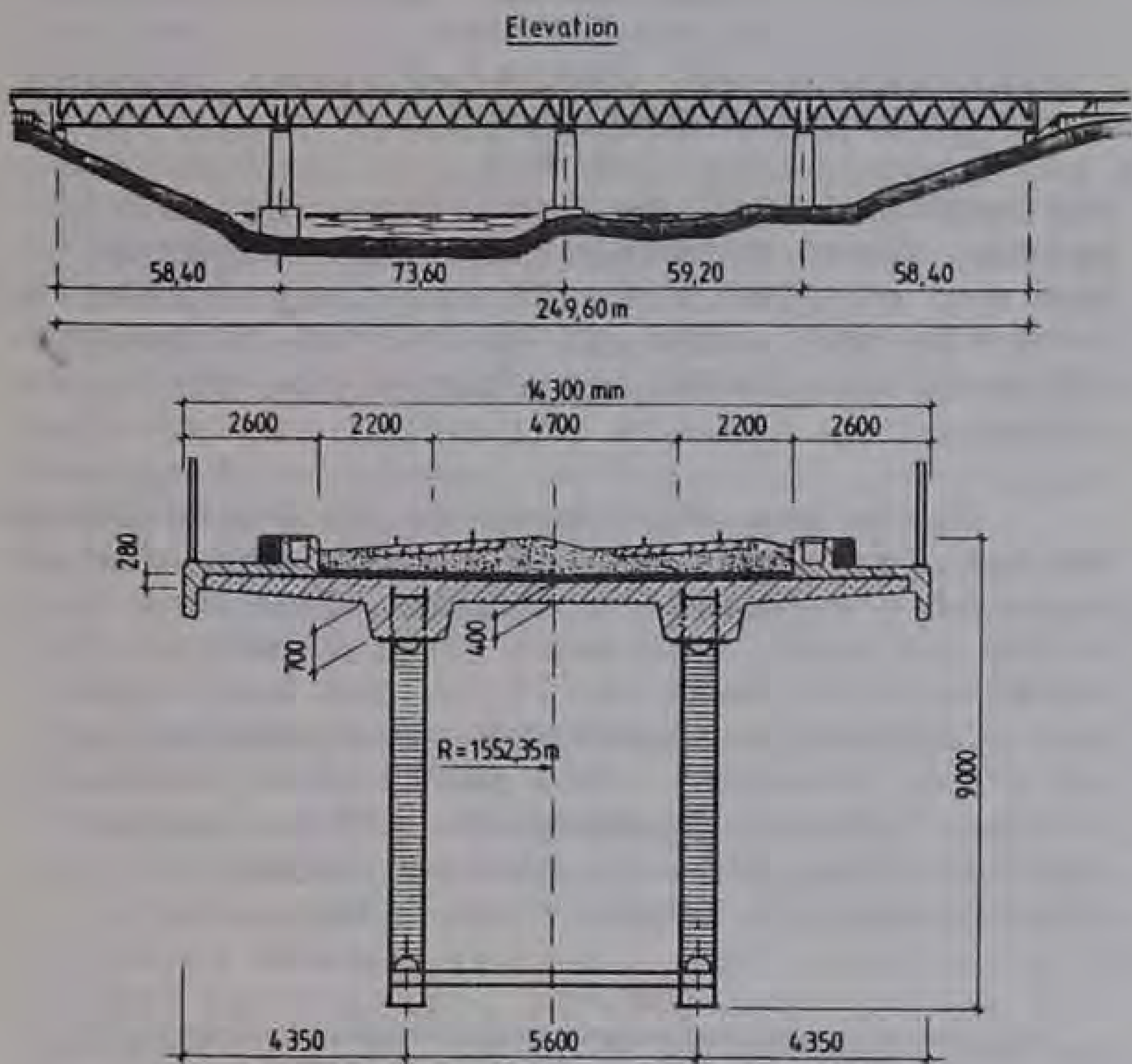

Fig. 4 Structural System and Cross-Section of a Composite Truss Bridge(2)

Composite structures have maintained a standard field of application where low structural weight is requested and restrictions to construction depth or adverse foundation conditions exist. The present economical conditions favor prestress concrete design, but it is closely challenged by composite structures, also with regard to maintenance, durability and appearance.

RESEARCH AND DEVELOPMENT

The sum of the experience of the individual European countries is brought in the draft of Eurocode 4 (3) which is the harmonized structural code for composite construction of the European community. The final draft of this document should be ready in 1989. The Eurocodes will provide an optional set of design rules which can be applied within the community as an alternative to the corresponding national rules covering the same technical matters.

For the ultimate limit state design, the Eurocodes distinguish four different types of steel beam cross-section dependent on their respective moment-curvature properties: (1) plastic, (2) compact, (3) semicompact, and (4) slender. Recent, as well as current, research are concentrated on analytical and experimental determination of shear strength of composite girders(4) and of rotational capacity of composite girders(5) to confirm the criteria determining the four types of cross-section. Work on these criteria is due to the lack of recognition, in the negative moment regions of continuous composite beams, of the following important effects which influence ductility:

- Moment gradient adjacent to maximum moment,
- Coincident axial compressive force balancing the tensile force in the reinforcement of the cracked concrete flange (tension stiffening)
- Interaction between three distinct types of buckling (web, flange, lateral)

Another field of research concerns the fatigue strength of stud shear connectors. This research is going on in Great-Britain(6) and in Germany, on both composite beams(7) and on large push-out specimens(8). The conclusions of these tests on large specimens are interesting:

- All test results range above the mean value obtained from earlier investigation,
- Tests with transverse prestressing of the slab range considerably higher than these without,
- The higher the concrete strength, the higher the fatigue strength,
- There is no significant effect of concrete cracks on the fatigue of stud connectors,
- An influence of the length of the stud connector cannot be observed as long as the stud length exceeds 4.2 times the shaft diameter.

Some other works are done in the domains of the serviceability and durability of the composite bridges. In recent years, problems have been encountered with prestressed concrete bridges. Unexpected and severe crack development in the concrete leading to corrosion of the prestressing tendons have occurred. The use of salt on roads as a deicing agent has accelerated corrosion damage. On composite bridges, no such severe damage has been observed, the main reason being that for some time the effects of creep and shrinkage in the concrete slab and differential temperature effects have been considered in design. However, thermal stresses can be significant when compared to dead or live load stresses in the slab and an intensive research program(9) has led to better regulation to control temperature effects.

It is generally agreed that wide cracks in the deck slab must be avoided, because they may contribute to rupturing of the waterproof membrane, which is normally used in Europe to protect the bridge slabs, and to corrosion of the slab reinforcement and the prestressing tendon, if they are, that is difficult both to detect and to repair. To avoid cracking and cracking effects, there are different ways used in Europe. One solution which is widely used in Great-Britain and more recently in Germany, is the control of crack widths by longitudinal reinforcement. Intensive research in the field of crack formation and crack widths are made in Germany(10). The width of the cracks in the negative moment region is minimized by an appropriate sequence of construction of the deck and controlled by the use of closely-spaced longitudinal reinforcement. This method is used in Switzerland, but for the highway bridges, the administration wants the slab to be prestressed for the long term use (that is for the permanent load acting after the composite action is created, beyond the creep and shrinkage effects, the temperature effects and a portion of the live load). The concrete slab only is prestressed longitudinally over the whole length of the bridge. This method is possible even in the case where the slab is cast-in-situ, because holes are created around groups of connectors during the concreting. With this method, the loss of prestress is less than in the case where the slab is prestressed after being attached to the steel girder or in the case where prestress is introduced by jacking over the supports during construction. A long term research on the behavior of two negative moment regions of composite girder, one with longitudinal prestress and one without, is now in progress(11).

Another method or prestressing composite bridges is now used, i.e., prestress with external tendons in truss action. This method is not really new, since it was used in Germany in 1957 and is used to reinforce old bridges. Figure 5 shows schematically the tendon's position along one girder of a composite bridge.

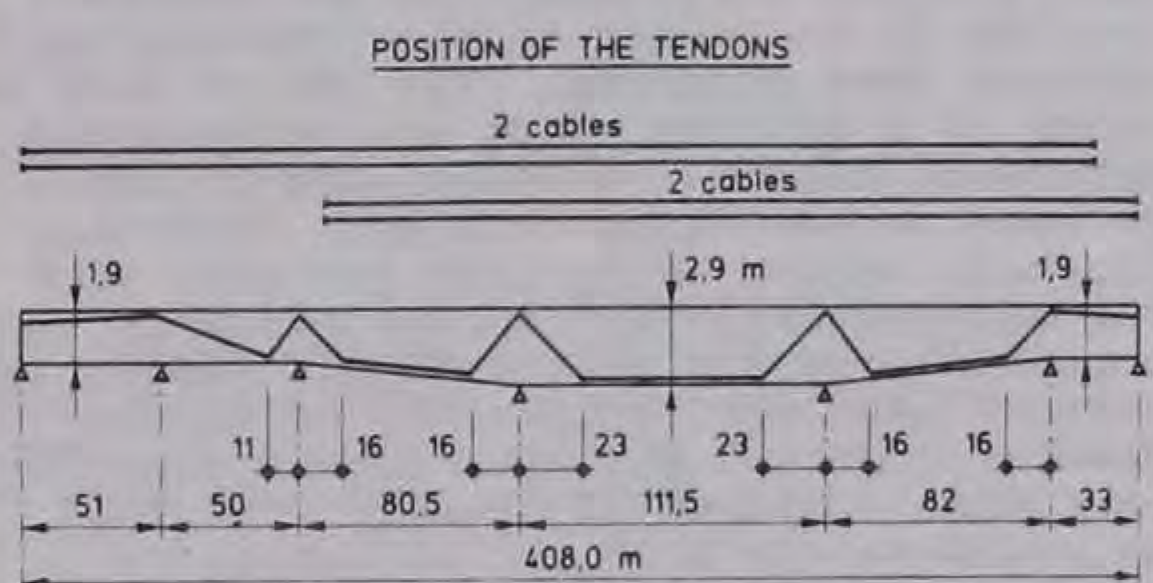

Fig. 5 Position of the External Tendons Along a Girder

The tendons are situated within the box girders without surrounding them with concrete. Their position and their prestressing forces are chosen to assure that the slab is under compression for dead loads as well as for a portion of the live load. The advantages of this system are:

- It enables the control and the maintenance of the tendons,
- It enables easy replacement and additions of tendons to be carried out after erection,
- The loss of prestress due to shrinkage and creep is minimized,
- Cracking of the slab near tendon anchorage is avoided.

This method will be used for a Swiss bridge whose construction will begin at the end of 1988. Tests are in progress to know the influence of curvature at points of tendon's direction change, on the cables.

These new developments in Switzerland do not form an isolated evolution. For example, in France, similar ideas are being pursued, and more than twenty bridges have been built in the last five years achieving concrete prestressing by external tendons. One particular example is the "Viaduc de Maupre". This bridge demonstrates clearly the principal development in France(12), (Fig. 6).

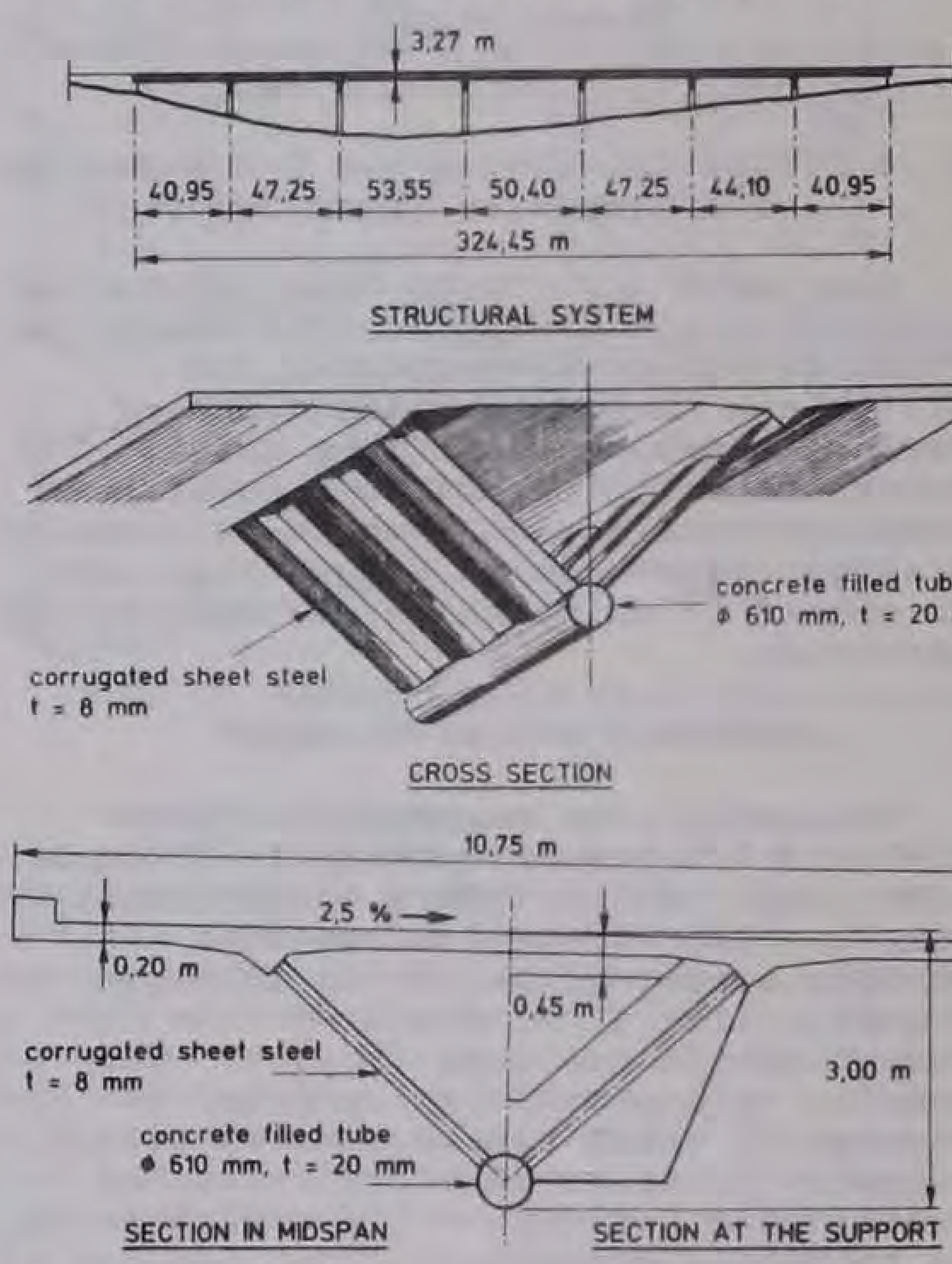

Fig. 6 Structural System and Cross-Section of the "Viaduc de Maupre"

There are three striking characteristics of this structure:

- The triangular shape of cross-section,
- The web consist of corrugated sheet steel,
- Concrete filled tube forms as part of the section.

The use of corrugated webs, capable of withstanding shear forces without absorbing unwanted axial stresses due to prestressing is a very attractive concept. Moreover, it is possible to deform or to bend a web panel transversally and longitudinally, and even to twist it, before placing it in its final position. This flexibility facilitates the assembly of webs without any special tolerances. As a result, the prestressing force is applied principally to the deck. The tendons are laid in a polygonal manner within the box. The concrete within the tube serves to stabilize the profile. This example demonstrates the following aspects of bridge design development in France: reduction of bridge weight, prestressing with external tendons, and the development of new forms of composite structures in steel and concrete.

SPECIFIC RESEARCH IN SWITZERLAND

One of the most complex aspects of the design of continuous composite girders of slender sections is the design of the negative moment region. The uncertainties caused by cracking of concrete on the redistribution of moments to midspan and on the forces acting on the shear connection necessitate more research. On the other hand, the influences of the grouping of the shear connectors at 1 m (40 in.) spacing on the flanges of the girders, which is typical of composite bridges in Switzerland, has not been studied. As stated in the draft of Eurocode 4: "Connectors may be placed in groups, with the spacing of groups greater than that specified for individual connectors, provided that consideration is given in design to the nonuniform flow of longitudinal shear, and to greater possibility of slip between the slab and the steel member." Does this possibility exist and what are the consequences on the load carrying behavior of the composite girder? At least, in all countries, the design strength of the stud shear connectors is chosen so that, at the serviceability limit state, the slip between the slab and the steel member is small enough to neglect its influence on the composite girder. If we calculate the force acting on a stud connector at this limit state, we find a great difference (50%) between the different European countries and with the Eurocode 4 who allows the highest value. Are these differences grounded?

In response to these questions, research has been undertaken which led to the development of a theoretical model (13). This model takes into account partial interaction between the concrete slab and the steel beam and is established to meet the requirements particular to composite bridges, which considers discontinuities at connection consisting of groups of shear connectors and the presence of cracking in the concrete slab in the regions of negative moment. The cracking is simulated in the model by one discontinuity of the slab, but not of the longitudinal reinforcement, at every edge of a group of connectors which represent the transverse cracking of the slab we can observe on the composite bridges. The theoretical model has been verified by comparing calculated values with results from tests performed on a composite bridge presently in service (Fig. 1). This comparison illustrates good agreement, and the theoretical model accepted as suitable for the purpose. A parametric analysis has then been carried out to define the effects of slip and concrete cracking on the state of stresses, deflections, and forces transmitted by the shear connections in composite bridges.

The results of the theoretical model have been compared to the results of the conventional design. In continuous composite bridges, cross-sections of steel members are of slender type, so elastic analysis of structure is normally required in conventional design. In the structural analysis, the concrete is considered uncracked over the support but in stress analysis, in the negative moment region, only the steel section and the reinforcement are taken into account. For the design for the number of connectors, the concrete is admitted as uncracked.

<u>Results</u>

As a first result, the slip calculated with groups of connectors and the slip calculated with the same number of connectors but distributed along the steel-concrete interface only differ by 3% (essentially negligible). Hence, the consequences of slip with groups of connectors on the behavior of the composite girder should be the same. This is true for the deformation of the composite beams and for the stresses in the bottom flange of the steel girder. But in the top flange, the shape of the stresses along the flange is quite different. With the groups of connectors, it resembles "saw teeth". The peaks of these "saw teeth" are more important in the region of zero moment than in the region of maximum moment. In the case of unshored construction, as it is usually, this phenomena, due to the grouping of connectors, is less important than with shored construction where it could be of importance, because the concrete load acts on the shear connection. With the connectors grouping at 1 m (40 in.) these peaks reach about ± 25 N/mm^2 (± 3.6 ksi) at the ultimate limit analysis which is not of consequences. Research is going on to fix some limits for the distance between groups of connectors.

Figures 7 and 8 show for the case of

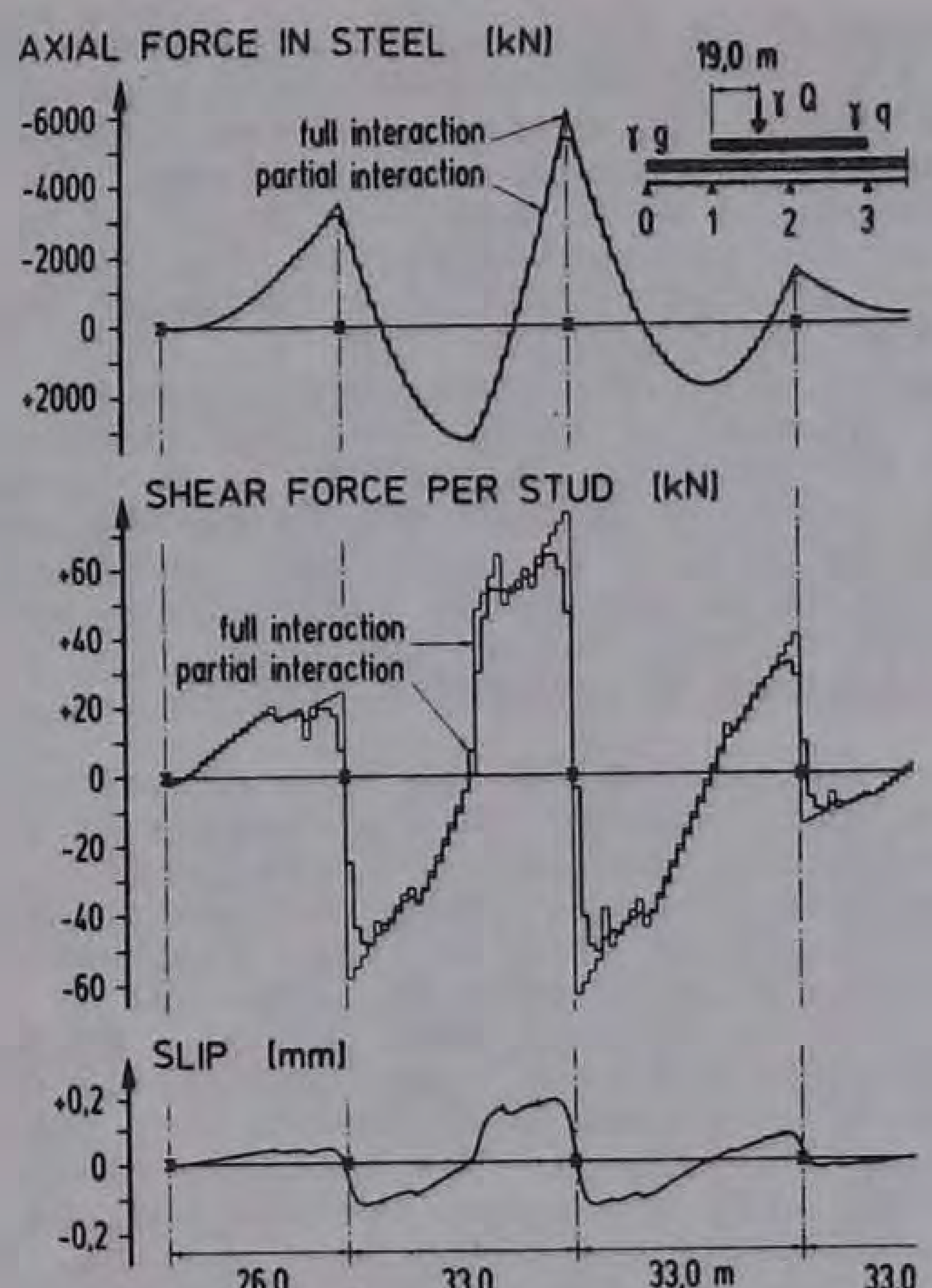

Fig. 7 Effects of the Slip at the Steel-Concrete Interface

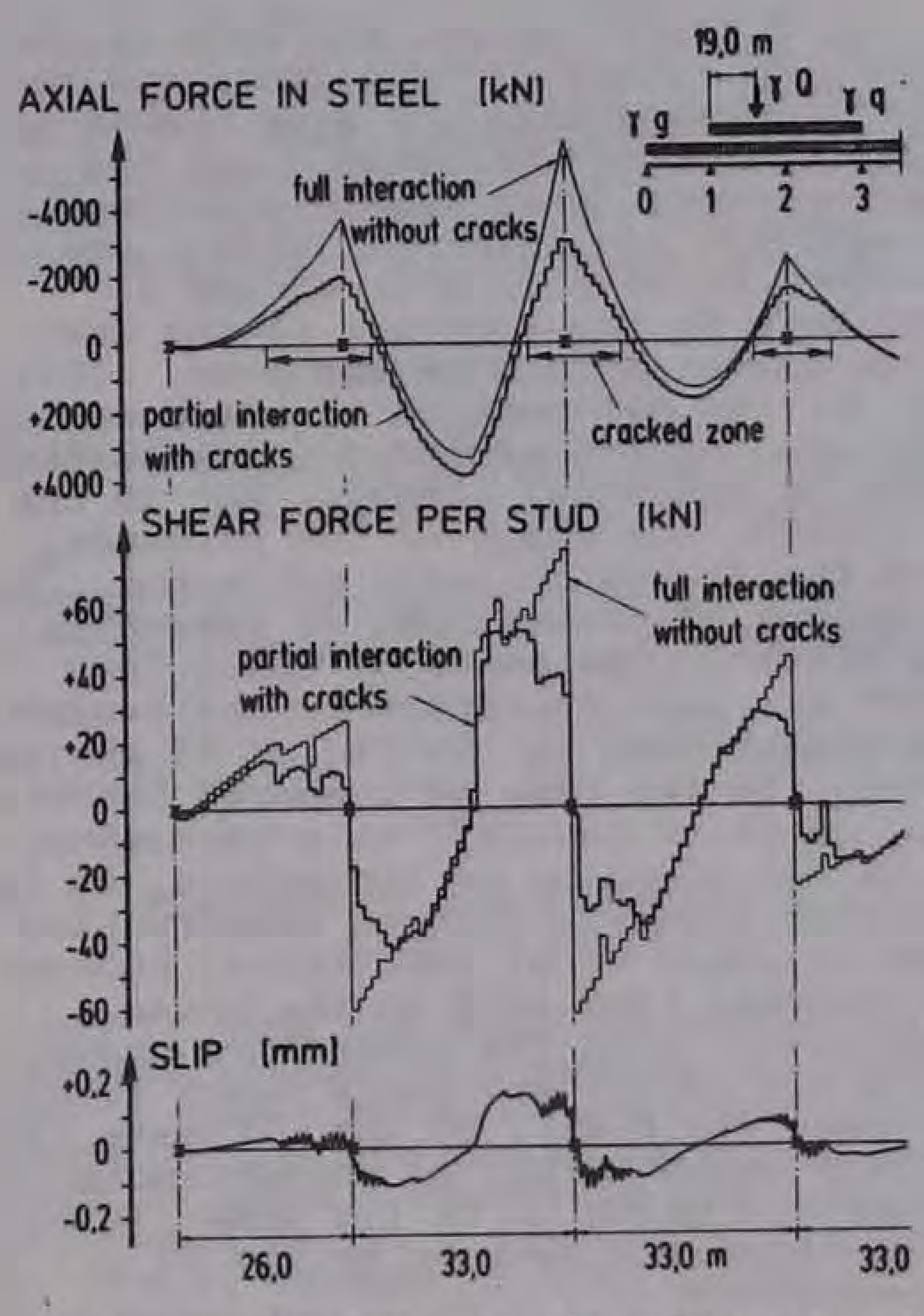

Fig. 8 Effects of the Slip and of the Concrete Cracking

the in-service bridge of Fig. 1, respectively the effects of slip and the effects of slip with cracking together for the loading case creating the maximum moment at the support 2.

As can be seen on the figures, the axial force in the steel, in the concrete slab, or in the reinforcement is reduced at the support region when considering partial interaction. This reduction is greater when considering the cracking of the slab, but in this case we have an increase of the axial force in the positive moment region mainly due to the redistribution of the bending moment. Of course, if the axial force is reduced, the shear force per stud is reduced too, as it is represented in the second part of the figures. The effect of the slip on the shear force is very distinct in the section where changes in the direction of the shear forces occur. Thus, the most stressed group of connectors is not located near the support, but at a distance corresponding to L/10, or L/5 when the concrete is cracked, of the intermediate supports, where L is the span length. For this loading case, the maximum slip is 0.19 mm (0.0075 in.). As a resume of the results:

- Effects of slip at steel-concrete interface:
  - . Reduction of the normal force: 15%
  - . Reduction of the force acting on the connectors: 20%
  - . Increased deflection: 2%
  - . Negligible change of stress.
- Effects of slip with concrete cracking:
  - . Reduction of the normal force: 50%
  - . Reduction of force acting on the connectors: 35%
  - . Increased deflection: 15%
  - . Moment redistribution: 10%
  - . Stress changes related to moment redistribution.

When the span length becomes greater, these effects diminish somewhat, but the maximum slip does not change significantly.

Another interesting effect of the flexibility of the connection is the reduction of the stress range, due to the fatigue loading case, in the shear connector shaft. This reduction is about 15% for a span length of 30 m (98 ft.) and is about 11% for a span length of 52 m (170 ft.).

The conclusions drawn may be summarized:

- Referring to the comparison of full and partial interaction, it can be said that even if connectors are welded in groups at distance of 1 m (40 in), slip has little influence and the effects of slip can be neglected when calculating the stresses and deformations,

- When calculating bending moments, the effects of cracking should be taken into account. As a simple method of accounting for cracking, one can consider a redistribution of bending moments of 10%,

- The positive effects of both flexibility of the connection and cracking leads to noticeable reduction of the shear force across the connectors that could be taken into account. For example, the rules given in Eurocode 4 for the design of the connection which are less conservative than those given in certain European codes could be applied.

CONCLUDING REMARKS

Currently, most European developments of different countries in composite bridge design follow parallel trends. Such unity of ideas and development may be due to similar requirements. These requirements include the need to build more economically by simplifying the structures but without risking their durability. Also, a certain conservatism or generosity when designing structures improves their durability and may finally prove to be a more economical design philosophy than the designing for minimum initial costs.

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RESEARCH NEEDS FOR SHORT- AND MEDIUM-SPAN BRIDGES

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SYNOPSIS

In November 1986, the National Science Foundation sponsored a Workshop on the identification of long range research needs for short- and medium-span bridges. State-of-the-art papers were presented in each of the following areas: bridge loads, materials, evaluation and strengthening, management systems, analysis and expert systems. These presentations were followed by workshop group sessions to identify and prioritize long range research needs in each subject area. In total, several hundred research needs were proposed and these are summarized in this paper. It is concluded that an extensive coordinated research effort is required if significant improvements are to be made in the design of new bridges and the rehabilitation of existing structures.

INTRODUCTION

Bridge building is an art which has been practiced for many thousands of years. Today it has evolved into a complex science as the demand for spanning greater distances increases. The art and science of bridge engineering spans both old and new structures, from early primitive materials to modern high strength steels. Rehabilitating existing structures and designing new bridges that satisfy a multiplicity of design constraints is the challenge facing today's engineer.

To meet this challenge, education, research and innovation are essential, and at least two Federal agencies in the United States (the Federal Highway Administration and the Transportation Research Board) have been active in this area, addressing the immediate (short-term) needs of the bridge profession. It is however necessary that a complementary research effort be directed towards the long-term, ill-defined, high risk/high cost projects to strengthen the present research programs. In recognition of this need, the National Science Foundation sponsored a Workshop in November 1986 to review and identify non-seismic research needs for bridges with special emphasis on long-term needs.

The workshop had the following objectives:

1) To determine the state-of-the-art of bridge design and construction;

2) To identify research needs, currently active research agencies and programs, and opportunities for long-range, high-risk, high-cost research investment; and

3) To identify priorities for these research needs.

Key personnel from both the private and public sectors participated in this Workshop. These included research and design engineers from Universities, State and Consultant offices. Further, representatives from the Federal Highway Administration (FHWA), the Transportation Research Board (TRB), and the American Association of State Highway and Transportation Officials (AASHTO) were also active participants in this Workshop.

Following two days of keynote lectures and special presentations at the Workshop, working groups met on the third day of the Workshop to discuss and identify research needs. These working groups and their respective Chairmen were as follows:

Bridge Loads:
- Professor Andrzej Nowak
  University of Michigan

Bridge Materials:
- Dr. John Kulicki,
  Modjeski and Masters,
  Pennsylvania
- Professor J.K. Rao, California State
  University at Long Beach

Bridge Evaluation and Strengthening:
- Professor James Baldwin
  University of Missouri, Columbia

Bridge Management Systems:
- Professor Celal Kostem, Lehigh
  University with
- Professor Richard McClure,
  Pennsylvania State Department
  of Transportation

Bridge Analysis:
Professor Frieder Seible
University of California, San Diego

Bridge Expert Systems:
Professor Celal Kostem, Lehigh University with
Professor Graham Powell, University of California, Berkeley

RESEARCH NEEDS

Several hundred research needs were submitted during the course of this meeting and in the final group sessions these were consolidated to a total of 94. Of these, about one-half were judged to be of high priority. Research needs are summarized below for each subject area noted above. Detailed descriptions and the text of the keynote presentations are contained in the Workshop Proceedings [1].

Priorities are identified as
H for high priority
M for medium priority
L for low priority

These identifiers are enclosed in parentheses in each subject title.

A. BRIDGE LOADS

Four subject areas were identified under this general title. These were

- live loads
- dynamic loads
- other loads
- load and resistance factors/load combinations.

Research needs in each area are listed below:

A1. Live Load

A1.1 (H) Live load model for bridge rating.
Develop live load (truck and/or lane load) for evaluation of existing bridges, to be used by AASHTO. Currently different states use different models.

A1.2 (H) Live load models for bridge design code (new bridges).
Develop live load (truck and/or lane load) to be used for the design of new bridges (AASHTO Specification). Current model (AASHTO) is not adequate.

A1.3 (L) Live load model for transit guideways.
Develop design live load model for future systems to be constructed in the United States. Recently completed transit systems clearly indicate to the need for such models.

A1.4 (M) Histograms of truck weights.
Gather data on truck weights, axle configurations, and axle weights; through truck surveys, weigh-in-motion, and other means.

A1.5 (M) Site-specific load spectra.
Develop live load spectra for typical bridges. Consider interstate highways, state roads, secondary and rural roads, live load spectra for posted bridges. This will serve as a basis to differentiate design criteria.
Currently secondary and primary road bridges are designed and evaluated using practically the same criteria.

A1.6 (H) Multiple presence models.
Develop models for headway distance, multiple presence of trucks on multilane bridges (in-lane and side-by-side), load spectra for bridge members (girders) due to multiple presence.

A1.7 (M) Load growth models.
Develop live load models for future bridges, establish load growth rate. This involves truck weight growth, changes in axle configurations, frequency of traffic, multiple presence.

A1.8 (L) Fatigue loading.
Develop load spectra for fatigue analysis, load spectra for members and connections, future changes.

A1.9 (M) Bridge load - damage accumulation.
Develop a relationship between live load level and bridge damage accumulation level, economic analysis (cost) of design, repair, and maintenance as a function of live load level (truck weights, frequencies, axle load and configurations), optimize the bridge formula.

A2. **Dynamic Load**

A2.1 (H) Develop a dynamic load model for design and evaluation of bridges.

The following considerations are recommended:

- effect of surface condition (roughness of the road), this is particularly important in short span bridges
- dynamic properties of the bridge (natural frequency of vibration, mass, span, material)
- vehicle properties (suspension system, speed)
- multiple presence, dynamic effect of multiple trucks or axles on the bridges
- live load vs. dynamic load, relationship between extreme live load and dynamic load, dynamic load as a function of truck weight and axle configuration
- dynamic load for timber bridges. In the current AASHTO specifications, no dynamic load is considered for timber bridges. Ministry of Transportation

(Ontario) tests indicate there is some dynamic effect. It is necessary to quantify the level of this effect.

- time effect vs. failure mechanism. Investigate the relationship between the truck crossing time and failure mechanism. Failure to timber structures extends in time and a short crossing time may justify higher load.

A3. Other Loads

A3.1 (M) Construction loads.
Develop load models to be used in evaluation of bridges during construction. This particularly applies to segmental bridges, but also to temporary structures (scaffolding, forms)

A3.2 (L) Temperature effects

- Develop temperature effect models for bridges without expansion joints. Elimination of expansion joints helps to reduce deterioration of bridges. However, temperature differentials are the major loading and the current state of knowledge is insufficient.

- Theoretically, nonlinear temperature gradients result in continuity stresses and self-equilibrating stresses. The self-equilibrating stresses acting on an unrestrained structure may produce high stresses within the member which are not clearly understood. Additional physical testing should be conducted to verify the existence and magnitude of these self-equilibrating stresses.

- More field testing is needed to calibrate the proposed temperature differentials for the United States.

A3.3 (L) Collision forces.
More data is needed to develop design criteria for collision forces. In particular the following should be considered:
- vehicle collision (pier, superstructure, railing)
- ship collision
- railway loads (derailing forces, direct impact)

A3.4 (L) Scour
Scouring is identified as the most frequent cause of bridge failure. More research effort should be directed to this problem to determine the effect of scouring on bridge performance, control of damage and prevention.

A3.5 (L) Impact load for bridge railing systems.
Develop design criteria for bridge railing systems.

A3.6 (L) Effects of severe environments on the safety of bridges.
Explore the effects of severe environments such as cold and moisture on bridge performance.

A3.7 (L) Braking forces.
Develop design criteria for braking forces due to single and multiple trucks.

A4. Load and Resistance Factors and Load Combinations

A4.1 (H) Load and resistance factors for the design of new bridges.
Develop load and resistance factors using state-of-the-art methodology in bridge engineering, structural analysis and probabilistic methods.

A4.2 (H) Load and resistance factors for evaluation of existing bridges.
Develop load and resistance factors using state-of-the-art methodology.

Other topics, which are related to A4.1 and A4.2 above, are listed below.

A4.3 (L) Verification of stochastic models.
New methods are now available which have been developed for either building structures or in other areas of engineering. They require verification and adjustment to bridge engineering applications.

A4.4 (M) Load combinations for calibration
Develop practical load combinations to be used in the development of load and resistance factors.

A4.5 (H) Target reliability level(s)
Establish the acceptable safety level(s) for bridges, taking into account age, type, costs and other parameters. This effort must involve a wide spectrum of bridge engineers representing bridge authorities, designers, researchers and users.

A4.6 (M) Reliability models for bridge structures
Develop reliability models for bridge structures using state-of-the-art methodology. Consideration should be given to members as well as structural systems (system reliability).

B. BRIDGE MATERIALS

Six subject areas were identified under this general title. These were

- mechanical properties
- damage and damage mechanisms
- new and advanced materials
- nondestructive evaluation
- reconstitution
- other topics.

Research needs in each area are listed below.

B1. Mechanical Properties

B1.1 (H) Materials and components (members) and connections

The following should be investigated:

- Environmental exposure effects

- Loading rate effects on material resistance

- In-situ measurement of properties

- Formulation of constitutive equations

- Failure criteria (response to stress fields across the section of members)
- Environmental fracture criteria (stress intensity in flawed/cracked regions) for propagation and energy absorption. Brittle-ductile fracture transitions.

- Long-term time-dependent properties

- Material variability characteristics quantification methodology in a uniform format between materials for characterization of resistance safety factors.

B2. Damage and Damage Mechanisms

B2.1 (H) Accumulation and Control Methods

To avoid or replace corroded metal members or components, the following should be investigated:

- Fatigue control and evaluation of remaining useful service life; prediction methods as a part of rehabilitation and replacement programs in bridge management systems. (For example, steel and composite members.)

- Environmental effects on materials (deicing salts on concrete, decks with protective mechanisms, acid rain, abrasion of wind and particles, biological attack).

- Relationship to serviceability (useability, reliability vis-a-vis repair mechanisms), ultimate strength (especially connections of metal structures, composite materials, members and connections)

B3. New Advanced Materials

New materials requiring application-related study include: fiber-reinforced, composite materials, structural adhesives, and joining techniques for steel. The following topics are therefore recommended:

B3.1 (H) Basic Research into New Materials

- Basic research to develop composite materials in bridge components and connections.

- Structural properties and behavior

- Non-structural issues (Fabrication Technology)

- Cost-effectiveness methodology, with life-cycle economics for load, environmental and durability effects

- Consistent safety factor design of bridge components made of composite materials for variation in material properties and load effects, to give consistent safety levels for bridge strength and serviceability, for corrosion resistance and fatigue.

B3.2 (H) Interaction with Conventional Materials

- FRP connections for metals and wood,
- cover plating

B3.3 (H) Methods to Maximize Structural Benefits

High-strength/lightweight materials should be studied together with the environmental benefits of composites.

B4. Non-destructive Evaluation

B4.1 (H) Material Assessment Techniques (MAT)

- Stress Analysis incorporating MAT output

- Survey of non-related technological fields for possible new material assessment techniques

B5. Reconstitution

B5.1 (H) Basic Materials Studies

- Dispersion of properties and flaws by processing

- Fundamental modification to new form, shape, and on predetermined residual state (autostress, post-tensioning, strengthening)

- Combination of materials

- In-situ modification of structural components

B6. Other Topics

B6.1 (M) Establish databases for correlation of in-situ field performance against laboratory test results.

B6.2 (M) Connection details (linked with B1. and B2.).

B6.3 (M) Characterization of resistance (as related to consistent safety), especially for new materials, composites, structural adhesives (linked with B1.).

B6.4 (M) Process control (linked with others).

C. BRIDGE EVALUATION AND STRENGTHENING

Five subject areas were identified under this general title. These were:

- improved rating methods using nondestructive methods

- estimation of load capacity

- correlation of the rate of deterioration with service conditions

- methods of repair, rehabilitation and strengthening

- nondestructive instrumentation.

Research needs in each area are described below.

C1. (H) **Implementation of Non Destructive Field Test Results in the Evaluation and Rating Process**

Field tests have shown that the "real" load carrying capacity of a bridge is almost always much greater than that predicted by conventional evaluation analyses. This discrepancy is due in large part to conservative modeling assumptions concerning unknown conditions. Non destructive field tests permit many of these assumptions to be eliminated, because the bridge itself provides an exact model. There is a need for more knowledge concerning appropriate measurements and interpretation of the results. Is it possible to identify the critical failure mode? What limit state should be considered in old bridges, yielding or collapse? Once the strength has been determined, what are the appropriate load factors for rating?

C2. (H) **Development of a Better Fundamental Understanding of the Real Load-Carrying Capacities of Bridges, using Results of Destructive Field Tests and Analysis**

If the differences between "real" and predicted ultimate strengths are as great as some field tests indicate, a great deal more knowledge is needed concerning modeling and analytical procedures. Test data on a wide variety of bridges are needed as a check on improved procedures as they are developed. Perhaps a center for bridge testing is needed. Such a center might serve as a clearing house for information on bridges that become available for testing, a repository for bridge test data, and a source of advice on what test information is needed. Such a center might also provide partial support for field tests and as a stimulus for sponsorship by other entities.

C3. (H) Correlation of Deterioration Rates with Service Loads and Conditions

It may be that far too much emphasis is being placed on evaluation of the current ultimate strength. In rating a bridge, what is really needed is a prediction of both the strength and serviceability of the bridge just before the next inspection. This obviously requires some prediction of deterioration under future service conditions. Except for fatigue, very little is known about deterioration rates of bridges under service conditions.

It may become possible to substantially increase estimates of current ultimate strength through incorporation of field test data in the evaluation process. If such a development were to result in substantially increased service loads, accompanying increases in deterioration rates would undoubtedly nullify at least part of the apparent gain.

C4. (M) **Repair, Rehabilitation and Strengthening**

If an old bridge is evaluated and found to be unsuitable in its existing condition to carry the traffic for which it is needed, decisions must be made concerning possible repair, rehabilitation, strengthening or replacement. Research is currently being conducted under several TRB projects in an effort to bring together available knowledge on techniques for accomplishing each of these. However, there is still a need for overall design criteria to be applied when working with old bridges. Since old bridges may not be expected to last as long as a new bridge, the design criteria for repair, rehabilitation and strengthening may not be the same as those for new bridges.

C5. (M) **Development of New Non-Destructive Instrumentation for Field Testing and Instrumentation**

Experimental measurements are always limited to some extent by the available instrumentation. Development of new instruments which are more economical, easier to use, more reliable, more precise, and would measure additional parameters, would be quite helpful in the evaluation process. Crack detection, measurement of corrosion deterioration and in-situ measurement of material properties such as fracture toughness are suggested for consideration.

D. BRIDGE MANAGEMENT SYSTEMS

Research needs in bridge management systems (BMS) are divided into two major categories:

- BMS programming
- Strategic long range planning

"BMS programming" is not limited to "computer programming" per se. The research recommended within this category includes both fundamental and applied research. The nature of the end product of BMS will inevitably be computer-based software systems, databases, and rules and guidelines. Within "BMS programming" the major subject headings for recommended research programs include:

- Optimization models for BMS

- Rating and Routing via BMS

- Decision making strategy and activity effectiveness.

Research needs in these areas are listed below.

D1. **BMS Programming**

D1.1 <u>Optimization Models</u>

Mathematical optimization models should be formulated and implemented in the prioritization and selection procedures.

Mathematical optimization models such as the linear programming procedure and the stochastic decision process should be formulated for use in the prioritization and selection procedure of an effective bridge management system. Models with funding constraints generally contain decision-making features and become quite complex but should be computationally feasible. The optimization models should be developed for implementation as the basic data quality and quantity increase during the later stages of bridge management development.

D1.2 Rating and Routing

Procedures must be developed to enable the state transportation departments to regulate the weights of licensed vehicles that can use the bridges and also to assess the bridges for safe passage of overweight vehicles operating with special hauling permits.

D1.3 Decision strategy and activity effectiveness

- Analytical tools are needed to accurately determine the reponse characteristics of repaired, retrofitted, or strengthened bridge structures.
- The feasibility of improved cost effective techniques for erecting, maintaining, repairing, testing and strengthening of bridges needs to be investigated.
- Collection and interpretation of data on the effects of specific maintenance, repair and rehabilitation on bridge performance is needed.
- The effect of maintenance, rehabilitation and replacement activities on bridge life must be determined.
- Determination of the cost-effectiveness of bridge maintenance, rehabilitation, and replacement.
- Development of benefit-cost analyses to determine bridge activity costs and user costs.

The feasibility of improved cost effective techniques for inspecting, posting, testing, maintaining, repairing, and replacing bridges must be explored. Effective bridge management requires reliable information on the additional service life that can be purchased with discretionary expenditures on existing bridges. Life-cycle costing using accurate data should be used to determine the optimum amount of funds to be spent on bridge activities. A systematic plan for collection of data on the effects of various bridge activities could be carried out at an NSF "Center of Excellence."

D2. Strategic Long Range Planning

D2.1 Develop accurate methods to predict future needs.

Since 1970, \$12 billion in Federal funding has been made available to States and local governments to improve bridges. Despite these unprecedented expenditures, 41 percent of the Nation's 574,000 highway bridges remain deficient. Each year as many bridges are added to the national list of deficient bridges as are removed from it. For the present, bridges are maintaining the status quo. However, because 40 percent of all existing highway bridges are between 15 and 35 years old, bridge needs are likely to increase substantially in the next two decades. Because current projections indicate a probable increase in the rate of bridge need growth, a comprehensive system is needed which will anticipate future needs and respond to changes in funding levels. The greatest potential benefits of a comprehensive bridge management system are the ability to explore a wide range of "what if" questions and predict what is going to happen in the future.

E. BRIDGE ANALYSIS

The identification of research needs in the bridge analysis area is complicated by the fact that all bridge research areas have some kind of analytical component. This makes it difficult to separate out individual analytical needs without direct reference to the overall research needs.

Therefore an attempt is made to identify analytical research needs by application rather than by more traditional methods in order to establish a clear objective.

A total of six application categories were identified as follows:

- Analytical Tools for Limit State Design
- Analytical Tools to Assess the Effects of Structural Rehabilitation
- Time History Models
- Systems Identification Methods
- Experimental Verification Analytical Models
- Special Topics

In addition to the identification of individual research needs within the above categories, the following general concerns are expressed regarding the development of new complex analytical tools:

a) There exists a large gap between the state-of-the-art in bridge analysis and the analytical tools most frequently employed in the practicing engineering community. Every effort should be made to disseminate advances in the analytical field in order to overcome the "recipe" oriented bridge design approach.

b) The analysis of a bridge structure is only as good as the model used to represent the actual conditions. Frequently obtained large discrepancies between analytical predictions and field load test results are often attributed to inadequate analytical methods whereas in fact the modelling of boundary conditions and secondary effects is in error.

c) Complex analytical methods have to be validated by experimental verification tests. In the nonlinear and failure range, large- or full-scale experimental models are needed to properly identify prototype behavior. Analytical models have to be calibrated against experimental tests under controlled laboratory conditions first before any reliable field applications can be made. The importance of, and necessity for, experimental validation of analytical models prompted the listing of a separate research category in that area even though experimental testing was not explicitly addressed in this workshop.

E1. Analytical Tools for Limit State Design

With worldwide changes in bridge design philosophy towards Limit State Design concepts, analytical tools have to be developed which address the individual limit states for the local (e.g. anchorage details), regional (e.g. transverse bending) and global (e.g. overall behavior) design of the bridge structure.

E1.1 (L) <u>Service Limit State</u>

Develop linear elastic models which can predict deflections and service stress levels.

E1.2 (H) <u>Overload Limit State</u>

Develop nonlinear models (cracking, yielding, etc.) which can trace the post-working stress behavior for Special Permit Overloads

E1.3 (H) <u>Ultimate Limit State</u>

Develop nonlinear modesl which can trace the complete behavior up to failure, including force redistributions, in redundant systems and simplified models which can easily evaluate possible collapse and failure mechanisms

E2. **Analytical Tools to Assess the Effects of Structural** Rehabilitation

With the volume of necessary structural rehabilitation of the national bridge inventory increasing, analytical tools have to be developed which can accurately predict the current state of existing bridge structures and allow the imzplementation of repair and strengthening measures in the modelling.

E2.1 (H) <u>Assessment of Damaged and/or Existing Structural State</u>

Develop models in which damage can be introduced and the effectiveness of repair studied

E2.2 (H) <u>Models for Repair and Strengthening Methods</u>

Develop models in which the addition of external tendons, composite overlays, and the like can be studied

E3. **Time History Models**

Accurate models are needed which can represent construction stages and associated force redistributions, environmental effects, time-dependent effects and load histories for arbitrary bridge geometry.

E3.1 (H) <u>Dynamic Amplification Load Allowance:</u> impact and braking.

E3.2 (H) <u>Construction Sequences:</u> segmental.

E3.3 (H) <u>Environmental Loading:</u> temperature, wind.

E3.4 (H) <u>Long Term Effects:</u> prestress losses, creep, shrinkage, corrosion.

E4. **System Identification Methods**

To facilitate dissemination of analytical tools and enhance application, black box models need to be developed.

E4.1 (M) <u>State and Capacity Determination of Existing and/or Damaged Bridges</u>

Develop models based on design and field data input, include non-structural components and effects.

E4.2 (H) <u>Determination of Dynamic Response</u>

Develop methods for parameter identification, and response spectra for traffic loads.

E4.3 (M) <u>Analytical Tools for Hazard Scenarios</u>

Establish the effects of a series of events, and how they impact on bridge safety.

E5. **Experimental Verification of Analytical Models**

The high costs associated with large-scale experimental testing precludes large-scale experimental parameter studies. However, these parameter studies can be carried out with analytical models as long as they are properly validated by experimental data.

E5.1 (H) Large-Scale Testing in Controlled Laboratory Environments

Carefully planned laboratory tests are needed for verification and calibration of analytical models.

E5.2 (M) Field Testing

After the appropriate laboratory verification, complex analytical models should be applied to field tests.

E5.3 (M) On-line Testing Procedures

Analytical tools for interactive testing, substructuring must be developed

E6. Special Topics

This special topics category contains all suggested research needs not covered in other application areas.

E6.1 (L) Timber Bridge Analysis Problems

E6.2 (M) Falsework, Fabrication and Erection Stages

E6.3 (H) Pre- and Post- Processing Software: interactive graphics systems, and the like.

F. BRIDGE EXPERT SYSTEMS

The research needs in the area of "expert systems," using the broadest definition of the term, can be categorized under two major headings:

- Computer-aided design, manufacturing, and information exchange, and
- Application of expert system technology in planning, design, construction, maintenance, inspection and rating.

The research activities that need to be carried out under the above-defined general categories are described below.

F1. Computer-aided design, manufacturing, and information exchange:

F1.1 Determine the components of an expert system in bridge design

A critical review of the current and projected capabilities of expert system technology should be undertaken. Similarly, various types of activities carried out in the bridge design process should be identified. Prototype models and expert systems should be developed to execute these activities to demonstrate and verify the practicality of the approach.

F1.2 Extend expert systems into manufacturing process.

It is believed that manufacturing aspects of "bridge engineering" need to be upgraded urgently. The feasibility of the use of expert systems for this process needs to be identified and demonstrated.

F1.3 Develop a "common information exchange 'format'".

A short term solution to developing integrated software systems is to design a universally accepted information exchange format. The "format" should be general, flexible, compact, simple, expandable and "politically acceptable." The format should cover all aspects of bridge engineering, and should be human-readable as well as machine-readable.

F1.4 Software integration through "shared" data and common "architecture"

A truly integrated software system will not merely exchange data among programs, but will operate from a common database. The integrated software should also have a common architecture for all applications. Furthermore, it should be possible to configure the system to reflect regional differences in design styles, and to change it easily to reflect the changes in design codes. Fundamental research is needed to identify appropriate tools and techniques to perform the above missions. Even though the developments in expert systems have not reached full maturity, as compared to, for example, the finite element method, there are still plenty of "tools" at this time that can be expeditiously applied in a "production environment."

F2. Application of Expert System Technology

F2.1 Development of expert systems to be used as a "training guide and surrogate consultant" on the analytical modeling and analysis of bridge superstructures.

Recent developments in the analytical modeling of bridge superstructures have become quite sophisticated. Some of the state-of-the-art tools are too complicated to be digested and used by "average" bridge engineers within the limited time at their disposal. The systems to be developed can be used both as a teaching/training aid and as an advisory tool to be referred to in the production mode.

F2.2 Development of expert systems to be used in construction, quality assurance and quality control, and in the assessment of reliability of "data."

A number of issues frequently encountered in bridge engineering can best be handled via expert systems yet to be developed. The issues which need to be addressed include, but are not limited to: How to relate quality assurance to structural reliability?, Sensitivity analysis as applied to bridge engineering and identification of the parameters most vulnerable to human errors, and development of error control strategies.

F2.3 Development of expert systems to optimize bridge inspection intervals.

Evaluate and, if possible, revise inspection intervals based on the observed deterioration rate and the consequences of local failure and deterioration for each structure type.

F2.4 Development of expert systems for the quantification of inspection reports.

Expert systems provide a means of capturing the knowledge of skilled employees in transforming qualitative inspection reports to quantitative assessments of strength.

F2.5 Development of expert systems to be used in conjunction with "bridge management systems."

Bridge management systems (BMS) require the implementation and interfacing of expert system concepts. This application will permit the uniform application and interpretation of BMS results and findings. Without such application, there exists the probability to make accidental and/or systematic errors.

F2.6 Application of "empty expert system shells" to bridge design, analysis, construction, rating, inspection, and maintenance.

A number of empty expert system shells are available. It is highly desirable to study the feasibility of using these existing systems in bridge engineering. If this feasibility can be successfully demonstrated, possible major investments in the development of expert systems can be substantially reduced and the implementation of the projects can be expedited.

CONCLUSIONS

It is clear that a large and substantial research effort is required if progress is to be made in the rehabilitation of existing bridges and the design of new bridges. This task is too large and too expensive for any one research agency to undertake on its own. A coordinated effort is required among the active funding agencies and research organizations. Such an effort should be directed by a committee of interested parties (researchers and program directors) with the shared responsibilities of technical direction and optimization of the investment of research funds. Other benefits would include the avoidance of duplicating research effort and the increased opportunity for improved interaction and information transfer with national and international agencies.

REFERENCE

1. Buckle, I.G. (Ed.) - "Proceedings of a Workshop Research Needs for Short- and Medium-Span Bridges," Sponsored by the National Science Foundation, organized by Computech Engineering Services, Inc., Berkeley, California, Report No. 5518.02., 1987, 197 pp.

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A SUMMARY OF RAILWAY BRIDGE RESEARCH NEEDS

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SYNOPSIS

Although railroad bridges are a vital part of the nation's infrastructure, very little research on these vital structures has been done during the past several years. In order to identify the most pressing research needs, a workshop entitled, "National Workshop on Railway Bridge Research Needs," was held on October 28-29, 1987, on the campus of the University of Illinois at Urbana-Champaign. The workshop was sponsored by the Association of American Railroads and was attended by representatives of the railroads, consulting engineers, and researchers. This paper presents a summary of the workshop and its findings.

INTRODUCTION

General Background

During the past decade the technical and popular engineering journals, as well as, the newspapers and magazines have reported on a regular basis the declining state of our nation's infrastructure. Highway bridges have been of particular interest to structural engineers. The plight of railway bridges, on the other hand, has received little attention in the literature or in terms of research dollars. This is in spite of the fact that, like highway bridges, these railway bridges are a very important part of the country's lifeline system. One should expect that the physical condition of our railway bridges is worse than for our highway bridges since the railway system is much older than the highway system. Although an official inventory has not been made, it has been estimated that there are over "600,000 linear feet of steel truss railway bridges in the United States, most built more than 70 years ago" [1]. This is probably an underestimate, and this is only one type of bridge that constitutes a small fraction of the total bridge inventory.

The problem has not gone unnoticed by the railroad industry. In a recent address published in Research Highlights [1], George Way (Vice President, Research & Test Development, Association of American Railroads) stated that "...bridge research (has been) an item on our 'wish list' for several years. ...We need to know more about the impact loads that modern traffic generates on these bridges. Because most bridges were probably heavily overdesigned for 40-ton axle loads of the time, research has always been postponed. But given the cost of bridge replacement and inspection, it is time to initiate a study to determine whether today's loads are being accommodated without reducing fatigue life."

Research Needs

Since relatively little funding has been available in recent years for railway bridge research, there have been no federal or private organizational stimuli for developing a consensus on what are the most pressing research needs. About two years ago the Association of American Railroads (AAR) included in their 5 year plan for research a significant increase in the level of funding for bridge research. Because of this, research needs has been an agenda item of the three bridge committees of the American Railway Engineering Association (AREA) Committee 7 (timber), Committee 8 (concrete) and Committee 15 (steel). The primary topics of concern raised in these meetings is the general lack of knowledge of what the actual loads are on bridges today. What are the real wheel loads? What are the impact effects? How are these loads distributed to the members? What are the load history effects? What are the material properties of the old bridges.

On March 17, 1987, a meeting was held in conjunction with the Annual Meeting of AREA in Chicago and was attended by members of each of the three bridge committees of AREA, by AAR staff members, by D. Foutch and by E. Barenberg who is a Professor of Civil Engineering at the University of Illinois and Director of the Affiliated Research Laboratory funded by AAR. Many topics were discussed at the meeting. One of the conclusions that was reached at that meeting was that a national workshop should be held for the purpose of identifying railway bridge research needs. It was also decided that this workshop should be organized by D. Foutch through the Affiliated Research Laboratory and that funding would be provided by AAR. The workshop was held on the campus of the University of Illinois at Urbana-Champaign on October 28-29, 1987. This paper provides a summary, sometimes verbatim, of the workshop proceedings entitled, "National Workshop on Railway Bridge Research Needs: Summary Report," that will be published by AAR during the fall of 1988.

BACKGROUND AND ORGANIZATION OF WORKSHOP

Background

The purpose of the workshop was to develop a list of research needs for railway bridges in the United States. As with any such activity, the success of a workshop is dependent on the composition of the group of participants. During the planning stages, it was decided that the group should be comprised of highly experienced engineers from railroad companies and from the consulting engineering profession and of research engineers from the railroad industry and from universities.

Letters were sent to 168 engineers informing them of the purpose of the workshop and of the dates on which it would be held. Each was asked to respond as to whether or not they would be interested and able to participate and, if they were interested in participating, to indicate in which small group discussion they would most like to participate (steel, concrete or timber). Regardless of whether they wanted to participate, each was also asked to submit several research needs that they believed were important. All members of Committee 7 (timber), Committee 8 (reinforced concrete) and Committee 15 (steel) were contacted as well as several other knowledgeable engineers.

A group of 53 engineers were selected to participate in the workshop. This included 18 railroad engineers, 13 consulting engineers of which a few are retired railroad engineers, 8 research engineers from AAR and 14 university researchers. Thus, a good blend of practicing and research engineers was attained.

Organization of Workshop

The workshop was organized around three types of activities: invited technical presentations, small group discussions and general discussions. The purpose of the technical presentations was to provide stimuli and information to the small group discussions. It was not to add to the list of research needs or to emphasize any particular ones. Fred Lawrence, Professor of Civil Engineering at the University of Illinois at Urbana-Champaign, explained the factors that affect fatigue damage and remaining life of steel bridges. Randy Jackson, Manager of Test Services at the Technical Center of AAR in Chicago, described the current capabilities for measuring railcar dynamics, wheel loads on tracks and bridge response to rail traffic with several examples from their current research programs. William Byers, Bridge Engineer with the Atchison, Topeka and Santa Fe Railway Company, presented several highly interesting case histories on evaluation and repair of existing bridges. All three presentations were excellent and provided valuable input to the small group discussions.

The heart of the workshop were the small group discussions. The participants were divided into three groups according to the three basic materials: steel, concrete and timber. The purpose of these discussions was to develop and discuss the research needs associated with bridges of each material. The groups were asked to summarize their discussions and describe the most pressing needs at the general discussion sessions. The initial letter of inquiry resulted in 108 letters of response which included 168 potential research topics to be considered by the workshop. These topics were condensed into about 50 topics and distributed to the participants in the small group discussions. These were used by the small groups to begin their deliberations. These topics were edited and expanded by the groups.

Summary reports and recommendations by each of the small groups were given at the general sessions which were attended by all workshop participants. These reports were discussed by the entire group. A final discussion session followed the small group presentations where any topic of interest could be presented to the group for discussion. The final editing of the list of research needs was completed at this time.

The final activity of the workshop was to develop a formal ranking of the research needs. This could then be used by the railroad industry and AAR to establish short- and long-range research goals related to bridges. It could also be used by other funding agencies or researchers to establish research priorities.

III. DESIGNATION AND RATING OF RESEARCH NEEDS

Research Needs Identified

The ideas for research submitted in response to the initial letter announcing the workshop were edited, combined and condensed to form an initial list of research needs that was used for discussion during the workshop. The small discussion groups and the workshop as a whole edited and added to the list to form the final list of research needs. The needs were grouped into seven general topics: A. Loadings; B. Analysis and Design; C. Fatigue Effects; D. Rating and Evaluation; E. Repair and Strengthening; F. Design Methods and Specifications; and G. Others. There is some overlap since some research items mentioned are necessary for two or more types of engineering activities. The final list of railway bridge research needs developed by the workshop contained 49 topics and is too long to include in this summary report. An edited version containing the most highly rated topics is given below. The topics have been renumbered for this paper and, thus, do not correspond to the numbers used in the original report.

A. Loadings

1. Conduct very detailed measurements of strains and deformations in primary and secondary members of bridges under static and impact load conditions in order to determine the actual distribution of loads in bridges of different types and configurations. The static and impact forces should be measured and followed from the track level, through the bridge and bearings, and into the substructure and pile caps. The data would be used to evaluate and improve current analysis procedures and to develop more accurate analysis capabilities that could be used for predicting the life of steel, concrete and timber bridges.

2. Study the effects of various parameters such as variations in track modulus, battered rail joints, wheel flats and out of round, speed, vehicle quality and dynamic properties, span length, structural configuration and others on the impact effects on bridges of all types. This study would combine field tests and calibrated analytical investigations.

3. Install permanent instrumentation on selected bridges on different types of lines to determine the long-term (one or more years) loading history and spectrum.

4. Investigate the actual longitudinal forces that are transmitted to a bridge substructure for various bridges with continuous welded or jointed track.

B. Analysis and Design

1. Develop better analysis and modeling procedures for design so that the vertical, lateral and longitudinal distribution of forces may be more accurately predicted. Thus, more accurate estimates of the loads carried by each bridge component can be made. Analysis of selected designs should be completed prior to testing in A1.

2. Develop guidelines for design of skewed spans, including testing of existing spans.

3. AREA Chapter 8 is based in great measure on ACI and AASHTO and research made for them. Member cross-sections (area to perimeter) and the LL/DL ratio are much greater for RRs. This should be investigated under B1.

C. Fatigue Effects

1. Investigate the importance of variable load histories on the fatigue life of typical steel bridge details. This should include laboratory testing and analytical studies using data on load histories from field studies.

2. Develop a microcomputer-based data bank of the fatigue characteristics of typical bridge details for use in design and evaluation.

3. As older bridges are taken out of service, collect samples of the wrought iron or steel so that the material properties and fatigue characteristics may be determined. Selected connection details, including riveted connections, and members should also be tested. This information would be used for the rating and evaluation of existing bridges of a similar vintage.

4. Develop fatigue damage models which can utilize the s-n curves for standard details contained in existing data banks, but which also incorporate other variables that may be important such as variable stress histories, mean stress, nonproportional loading and others. These would be used for design and evaluation. They would also be valuable for evaluating the effects of future increases in allowable car weights on the deterioration of the current bridge stock.

5. Review existing test results for fatigue effects of timber structural members and their connections. Conduct enough additional tests to "fill in the holes" so that reliable fatigue design provisions for timber structures may be developed.

6. Investigate the importance of impact and fatigue on concrete, on reinforcing and prestressing steel and on bond and anchorage so that reliable fatigue design provisions for reinforced and prestressed concrete structures may be developed.

D. Rating and Evaluation

1. Investigate through a literature search and selected verification the reliability and relative cost of inspection methods including nondestructive tests and visual inspection techniques for detecting cracks, flaws or other types of deterioration in steel, concrete and timber structures.

2. Develop methods for using data on annual tonnage carried on lines in conjunction with fatigue damage models that incorporate variable load history effects to evaluate remaining life in existing bridges. This would involve statistical studies of current tonnage figures, train configurations and field measurements of stress histories along with historical data on tonnage and train configurations.

3. Develop rating procedures that would utilize relatively simple field tests and visual or other simple inspection techniques to provide input to an advanced analysis program. The strength of the bridge would then be computed using a nonlinear analysis that would account for the distribution of loads and the state of corrosion of the members and other factors as determined from the field studies.

4. Conduct laboratory tests to determine the effects of corrosion on the strength and fatigue life of steel and wrought iron members and connections. Develop simple field procedures for determining the degree of corrosion in bridge

members and connections and verify the reliability of the procedures in the laboratory.

5. Develop criteria for inspection and for repair and/or replacement of existing concrete bridges. Special attention should be given to concrete spalling and reinforcement corrosion.

6. Develop impact criteria and load factors that are more realistic than current design values for concrete bridges and, therefore, are suitable for rating and evaluation.

7. Develop more reliable nondestructive testing techniques for field use for determining the extent of voids and/or decay in timber structures. This would be very valuable for many types of structures in addition to railway bridges.

E. Repair and Strengthening

1. Develop effective means of strengthening and/or rehabilitating existing bridges that do not require taking the bridge out of service for long periods of time. This should also include developing methods of adding redundancy or alternate load paths. The methods might include selective member replacement and post tensioning of steel members.

2. Develop new ways to prolong the life of very old long span truss bridges.

3. Develop effective methods for strengthening or upgrading existing bridges for heavier vehicles, heavier axle loads and/or tighter axle spacing.

F. Design Methods and Specifications

1. Develop a new design specification for seismic loadings based on the results of recent studies on the behavior of bridges under earthquake loads. The ATC-6 and new AASHTO specifications could be used as a model but would be modified to account for the differences between highway and railway bridges.

2. Using the results of field tests and analytical studies, develop new provisions to account for impact and for longitudinal and lateral load distribution in bridges.

G. Others

1. Investigate how continuous rail across a bridge affects its performance and make recommendations on expansion joints and rail anchorage.

2. Investigate differences in development lengths for various types of strands for use in short prestressed members.

Summary of Small Group Discussions

Steel: It was decided that because of relatively short amount of time available, the discussions would be limited to the problems associated with existing bridges rather than those associated with new construction. It was decided to further limit the discussion to problems peculiar to railway bridges and not those also associated with highway bridges. It was felt that a small part of the research currently being done for highway bridges could be applied to railway bridges and, therefore, it was not necessary to reproduce this.

The most urgent research need associated with existing steel railway bridges is to identify the current loadings that they must carry. Many existing steel bridges were built years ago and were designed for significantly lighter loads than they are now required to carry. The primary tasks involve the identification of the actual wheel loads, of the distribution of these loads throughout the members of the bridge and down to the foundation, and of the nature and magnitude of impact effects. It is also important to identify the load and stress histories and trends for typical bridges under different service conditions for use in evaluation of remaining fatigue life and investigating the effects of further increases in the allowable car capacities. This will require a review of the results of previous fieldtests conducted in the U. S. and abroad and conducting additional field tests of existing bridges.

Another question faced by engineers who evaluate the capacity of existing steel bridges is "What is the effect of corrosion on the strength and remaining fatigue life of steel bridges?".

It was recommended that the effects of continuous welded rail on the response of bridges be investigated. The current specifications are based of experience and opinion with very little research to back the up.

AREA or AAR should publish periodically a list of publications and pending research concerning bridges. It was felt that a lot of research has been done that would be applicable to railway bridges but that is never seen by the average bridge engineer. It was also believed that this would allow limited research dollars to be applied toward filling the holes rather than reproducing existing results.

It was recommended that Committee 15 of AREA consider preparing a railway bridge inspection manual. This might include typical problem details and repair techniques. It might also include information on evaluating remaining life.

The final recommendation was that Committee 8 and Committee 15 complete their work on the development of seismic design guidelines for railway bridges.

Concrete: Since many of the new replacement bridges are now made of precast prestressed concrete, particularly for the shorter spans, there is a lot of interest in research.

As was the case for steel bridges, the most urgent research need is to identify the loadings that reinforced concrete bridges are required to carry today. Some of the questions are as follows: How do impact effects lessen as they go from the rail surface to the pile caps? How are the longitudinal and lateral forces distributed as they pass from the rail, through the ballast, to the deck, and on to the substructure? How much gets to the pile cap and how does the load distribute to the piles? What is the magnitude of longitudinal and transverse forces? How much shear and torsion are developed in skewed spans and do these behave the same as highway bridges?

Much of the current specification for the design of railway bridges was taken from ACI and AASHTO specifications. A closer look at many of the provisions needs to be made to determine if they do apply to railway bridges. Some of the aspects that should be investigated relate to load factors, shear criteria, designation of members as columns or piers, minimum reinforcing requirements and column tie detailing.

The effects of spalls in tension and compression zones should be investigated. How do spalls affect bond, loss of area and necking stresses, and is it worse near bearings or near mid-span?

Impact criteria and load factors that are more realistic than the current design values for concrete bridges should be developed so that they would be suitable for rating and evaluation. The committee organization should be utilized when research programs are organized. Committee members should be associated with specific tasks and incorporated into the research programs to facilitate speedy implementation of research results. New programs aimed at rating and repair should build on existing research. The report of NCHRP 293, "Repair and Strengthening of Highway bridges should be reviewed with the goal of preparing a report to encourage the utilization of pretested techniques. Engineers should be made aware of information as it becomes available, and methods developed in other countries should also be investigated.

A thorough literature search on seismic design of bridges should be undertaken. A workshop organized by experts should be conducted to identify the strengths and weaknesses of the existing seismic design codes for bridges as they apply to railway bridges. The Applied Technology Council has proposed to do this, and this proposal should be strongly considered.

The development lengths of various types of strand should be investigated for use in short prestressed members. This is very important since pile caps have been found that have lost some of their prestressing.

Topics that were believed to be of very low priority included the following: investigate the practicality of using composite construction; develop better design rules for wind loadings; develop better bearings; develop better bridge ties and more effective ways of attaching them to the stringers; and develop more efficient fabrication and construction methods for replacement and rehabilitation of bridges.

Timber: One key research need for timber bridges is to determine the distribution of loads throughout the various members. The abutments and support conditions are critical factors. What is the effect on the load distribution of uneven settlement of the piers or uneven pile settlement at a pier or abutment? The magnitude of impact loads and their distribution to beams and stringers needs to be determined.

Load tests on existing bridges conducted in the past to determine their strength have resulted in measured strengths that were 25% to 100% greater than calculated. Research needs to be done to determine what factors lead to this excess strength.

There is a need to calibrate or evaluate analytical models for analysis based on field tests. As timber bridges are taken out of service some ways need to be developed for saving or securing these bridges for future load tests if funding is not immediately available. Selected members might also be saved for future study. Problems associated with timber bridges will be around for a long time.

The effects of checks and other imperfections in timber girders on damage caused by horizontal shear induced by fatigue and/or impact loads needs to be investigated.

Standard procedures for inspection of timber bridges need to be developed. Load testing in conjunction with analysis should be investigated as a possible evaluation method. Various modelling schemes should be evaluated based on field tests to determine how reliable they are. One needs to be careful about saying how much remaining life a bridge has since these are very uncertain predictions based on incomplete information.

There is a need to study the properties of second generation trees. There is a general feeling that the properties of this timber are marginal as they arrive at the site now. The properties of exotic species arriving from other countries need to be determined. It is proposed that some of these species provide high strength, high durability and/or require no treatment.

A reliability based design code should be developed along the lines of an LRFD specification but explicitely accounting for probabilities.

Rating of Research Needs

After the completion of the small group presentations and the general discussion, a formal rating of the research needs described above was undertaken. A rating sheet listing a one line description of each topic was provided to each participant. Each item was to be rated from 10 to 0, where 10 signifies a need of the highest priority, 1 signifies the lowest priority and 0 signifies an item that should be removed from the list. The average score for each topic is given below.

Item	Description	Rating
A1	Field measurements of static and dynamic stresses	9.6
A2	Investigate impact and its effects	8.7
A3	Measure long-term loading histories	6.1
A4	Determine longitudinal forces transmitted to bridges	7.6
B1	Develop better analysis procedures for design	7.5
B2	Testing and design guidelines for skewed spans	5.6
B3	Determine if ACI and AASHTO concrete specifications are adequate	6.5
C1	Effects of variable load histories on fatigue life	8.8
C2	Develop microcomputer-based fatigue data bank	5.0
C3	Determine properties of old steel and wrought iron	6.1
C4	Develop fatigue damage models for design and evaluation	5.6
C5	Fatigue properties of timber structures	4.8
C6	Impact and fatigue of concrete structures	5.6
D1	Determine effectiveness of available inspection techniques	5.2
D2	Develop methods for determining remaining life	7.0
D3	Rating procedures based on field tests and analysis	7.4
D4	Determine effects of corrosion on strength and fatigue	7.5
D5	Inspection and repair of prestressed concrete bridges	5.6
D6	Develop realistic impact criteria and load factors for RC	7.4
D7	Develop reliable NDT techniques for timber	7.5
E1	Develop strengthening and rehabilitation techniques	5.3
E2	Develop ways to prolong the life of old bridges	6.9
E3	Strengthening and upgrading for heavier loads	5.6
F1	Specifications for seismic loadings	6.0
F2	Specifications for impact and load distribution	6.5
G1	How does continuous rail affect bridge performance	6.5
G2	Investigate development lengths for different types of prestressing strand	6.7

SUMMARY AND RECOMMENDATIONS

The results of the rating of research problems given in the preceeding section provide a valuable indication of which problems associated with rehabilitation, evaluation and design of railway bridges are most pressing. The railroad engineers, consulting engineers and researchers were in general agreement on most items.

Item A1 is clearly thought to be the most urgent problem. This reflects the fact that very little is known about the nature and magnitude of the static, dynamic and impact loads that bridges are carrying today. The static and impact forces need to be measured and followed from the rail, through the bridge and bearings, and into the substructure and pile caps for several typical bridges of each material and for different traffic mixes. This is really the key ingredient for evaluation and repair of existing bridges, for determining remaining life and/or fatigue effects and for developing rational design procedures. It is recommended that this problem be given the highest priority as research funds become available.

Items C1 and A2 also received strong support from all groups. Both of these items also deal with the loadings on bridges. The determination of which physical parameters such as variations in track modulus, battered rail joints, vehicle quality and dynamic properties, span length, structural configuration and others affect the impact forces that all bridge types experience is required if methods of reducing these effects are to be developed. It is important to investigate the effects of variable load histories on the fatigue life of typical bridge details. Recent research has shown this to be a very important for highway bridges. The results of this are important for meaningful and rational prediction of remaining life of steel bridges. These two problems should be considered to have very high priority.

As additional research funding becomes available, the following research topics should be given high priority:

A4 Determine the longitudinal forces transmitted to bridges

B1 Develop better analysis procedures for design

D7 Develop reliable NDT techniques for timber

D4 Determine the effects of corrosion on strength and fatigue

D3 Develop rating procedures based on field tests and analysis

D6 Develop realistic impact criteria and load factors for RC

It is hoped that these results and recommendations will be useful to the railroad industry as a whole, to the Association of American Railroads and to other organizations who share this great concern for the condition of our country's railway bridges.

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ACKNOWLEDGEMENTS

I offer my sincere thanks and appreciation to all of the engineers who donated their valuable time and effort to participate in this workshop. Without their enthusiastic participation the workshop would have been a meaningless exercise. I also want to thank Mr. H. Sandberg, Mr. E. Bard and Professor R. Gutkowski for serving as the small group discussion leaders.

The financial support for this workshop provided by the Association of American Railroads is gratefully acknowledged. Without their recognition of the potential problems faced by many of our old railway bridges this workshop could not have been conducted.

I would like to thank Professors W. Walker and W. Gamble of the Department of Civil Engineering at the University of Illinois at Urbana-Champaign for their help in sorting the research ideas that were submitted for consideration and for their assistance in conducting the workshop and completing this report. Professor E. Barenberg assisted in preparing the details of the daily sessions and provided moral support throughout. His support is gratefully acknowledged.

Finally, I want to express my deepest appreciation to Melody Kadenko for the excellent job she did in preparing the workshop materials and for the evening hours she spent during the workshop updating these materials.

SESSION I

THE DESIGN OF PILES IN INTEGRAL ABUTMENT BRIDGES

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SYNOPSIS

Two alternative approaches for designing piles of integral abutment bridges are presented. The development of the methods was based on the results obtained from full scale field tests, laboratory model tests, and finite element investigations. When a pile design is governed by the capacity of the pile as a structural member, Alternative One (elastic design) should be used for piles with limited ductility. Alternative Two (force redistribution design) should be used when adequate pile ductility (moment-rotation capacity) exists to permit the safe design of bridges that can be significantly longer than those designed according to Alternative One.

INTRODUCTION

Traditionally, a system of expansion joints, roller supports, and other structural releases have been provided on bridges to prevent damage caused by thermal expansion and contraction of the superstructure. However, these joints are difficult to maintain and frequently do not function properly, resulting in a restrained condition. The apparent lack of major structural damage caused by frozen expansion devices contributed to the development of integral abutment bridges. These bridges do not have an expansion joint between the superstructure and the abutments; therefore, the bridge girders and foundation piling are rigidly connected together at the abutments.

This paper presents a summary of the results for a recently completed research project [1] whose main objective was to develop a simplified and rational design approach for the piling of integral abutment bridges. To accomplish this objective, three full-scale field tests on HP 10x42 steel piles were performed, over 40 one-tenth scale laboratory tests on one-inch square tubes were conducted, and many nonlinear finite element analyses were investigated.

FULL SCALE FIELD TEST PROGRAM

Two HP 10x42 piles, having a 40-ft embeddment in a clay soil, were tested and monitored to establish pile behavior when vertical and horizontal loads were applied at the top of the piles. An assortment of instrumentation, consisting of strain gauges, displacement transducers, dial gauges, load cells, surveyor's levels and transits, and scales, was used to measure pile strains below grade, loads applied to the pile, and pile movements at the ground surface. The three field tests conducted were a vertical load test on a laterally restrained pile, a lateral load test on an unrestrained pile, and a vertical load test on a horizontally displaced pile having rotational and lateral restraint of the pile head. Analysis of the experimental results involved differentiation and integration of axial and flexural strain functions, that were developed from the measured strain and displacement data gathered during the pile tests. From these studies, soil response characteristics at various points along the pile length were established for the skin friction and vertical displacement (f-z), tip bearing and displacement (q-z), and lateral resistance and lateral displacement (p-y) relationships. Mathematically, each of these resistance versus displacement data point sets were used in a curve fitting procedure to obtain continuous non-linear functions (modified Ramberg-Osgood equations) that were needed to model the soil as Winkler type soil springs. An initial stiffness, ultimate resistance, and a shape parameter, that describe each Ramberg-Osgood expression, were the analytical parameters needed to model the soil in a previously developed finite element program

[2,3]. Comparisons of axial load versus vertical displacement, axial force versus depth, lateral load versus lateral displacement, bending moment versus depth, and total strain versus depth, revealed close correlation between the experimental and analytical results. A vertical load capacity of about 280 kips was obtained for both the vertical load and the combined load tests, indicating that the applied lateral displacement of about 2-in. at the pile head apparently did not affect the pile strength.

LABORATORY MODEL TEST PROGRAM

Over 40 one-tenth scale model pile tests were conducted on one-inch square by 60-in. long steel tubes representing piles. The scale model tests allowed for investigating different design parameters that could not be accomplished easily with full scale field tests. Both friction and bearing model piles, having a pinned-head or fixed-head configuration, were tested in a soil bin containing either loose or dense sand. Additional pile head conditions consisted of a predrilled hole to eliminate soil contact for the upper 8-in. of pile length and a model abutment that was attached to the top of the pile.

Vertical, lateral, combined lateral displacement and vertical load, and lateral cyclic loading conditions were applied at the head of the model piles. Instrumentation consisted of electrical resistance strain gauges, direct current displacement transducers, and load cells.

The soil behavioral characteristics (f-z, q-z, and p-y curves), represented by modified Ramberg-Osgood expressions, were established by basically the same techniques developed for the full scale field tests. Once again, acceptable correlation existed between the experimental and analytical results for pile load versus displacement and strain versus depth relationships.

PILE DESIGN CRITERIA

According to Article 4.3.4 of the AASHTO Specification [4], the pile capacity is controlled by the minimum of the following three criteria: (1) Capacity of the pile as a structural member (Case A), (2) Capacity of the pile to transfer load to the ground (Case B), and (3) Capacity of the ground to support the load (Case C). A design method [5] that applied the Rankine equation to determine pile capacity, based on a lateral mechanism (pile strength-Case A) and a slip mechanism (unrestrained vertical movement-Case B), was previously developed. However, the Ranking equation is quite conservative for Case A, particularly in the inelastic column buckling region, where the capacities predicted by this equation can be as much as 33 percent below the strengths established by the parabolic column buckling equation given in the AASHTO Specification [4]. To provide a more realistic and rational approach to pile design, two alternatives are presented herein.

AASHTO - Case A

The capacity of the pile as a structural member (Case A) involves pile stability and yielding conditions, expressed by AASHTO Eqs. (10.41) and (10.42) for the Service Load Design Method and by AASHTO Eqs. (10.155) and (10.156) for the Strength Design Method. An equivalent cantilever, which replaces the actual pile and soil system by an isolated pile with a fixed base at a calculated depth, was used to analyze a pile as a beam-column member. For the idealized member, equivalent cantilever lengths can be established for the elastic buckling load, maximum pile bending moment, and horizontal pile stiffness. Each of these three design criteria may produce different equivalent lengths for a pile in a uniform soil. If a layered soil exists, an equivalent uniform soil profile can be established from an energy formulation that equates the work done by the uniform and actual soil systems in moving through the actual lateral pile displacements. The authors in Ref. 1 have presented a non-dimensional formulation for the equivalent cantilever lengths.

Two design alternatives have been developed to establish the capacity of a pile as a structural member. Alternative One accounts for the pile stresses induced by the lateral movement of the pile head caused by thermal expansion or contraction of the superstructure and by the rotation of the pile head caused the end rotation of the bridge girders. The horizontal movement, Δ, and rotation, θ_w, at the pile head cause a primary bending moment, M, to occur at the end of the equivalent cantilever beam-column, given by

$$M = \frac{D_1 EI\Delta}{L^2} + \frac{D_2 EI\theta_w}{L} \qquad (1)$$

where, D_1 and D_2 are moment coefficients equal to 6 and 4 or 3 and 0 for fixed-headed or pinned headed piles, respectively, and E, I, and L are the modulus of elasticity, moment of inertia with respect to the plane of bending, and equivalent cantilever length for bending, respectively, for the pile. Only the end rotation of the bridge girders that occurs after the abutment diaphragm forms a rigid joint between the piles and the girders affects the pile stresses. The moment induced by the lateral displacement of the pile head will significantly affect the pile strength. This elastic design approach is applicable to piles with minimal moment-rotation capacity, such as timber, concrete, and possibly some steel piling. The potential reserve strength associated with plastic hinge formations is not recognized with Alternative One.

Alternative Two does not consider the pile stresses induced by the thermal expansion or contraction of the bridge superstructure, but does consider the pile stresses induced by the vertical load. The justification for neglecting these flexural stresses can be found in first-order plastic collapse theory, where the plastic collapse load is not affected by residual stresses, thermal stresses, imperfect fit, or in this case, support movements, as long as local buckling and lateral torsional buckling are prevented. The bending moment, M, at the end of the equivalent cantilever beam-column, caused by the vertical pile load, P, and by the girder end rotation, θ_w, is given by

$$M = D_3 P\Delta + \frac{D_2 EI\theta_w}{L} \qquad (2)$$

where D_3 is a moment coefficient equal to one-half or unity for fixed-headed or pinned-headed piles, respectively, when the contribution of the shear forces to the moment resistance is neglected. This design approach is applicable to piles with sufficient ductility (adequate moment-rotation capacities). Even though many of the HP-shaped pile sections are classified as "non-compact" sections, they can still be designed according to Alternative Two, if the section possesses a rotational capacity at the plastic hinge locations in excess of the rotation demand induced by the lateral displacement at the pile head.

Based on moment-rotation relationships, the authors in Ref. 1 developed a ductility criterion that was expressed in terms of lateral displacement. The allowable horizontal movement, Δ_i, at the pile head is given as

$$\Delta_i = \Delta_b\,(D_4 + 2.25\,C_i) \qquad (3)$$

where, Δ_b is the elastic lateral displacement of the pile head when the extreme fiber flexural stress is equal to the allowable bending stress; D_4 is a ductility coefficient equal to 0.6 or 1.0 for fixed-headed or pinned-headed piles, respectively; and C_i is an inelastic displacement capacity reduction factor expressed as

$$C_i = \frac{19}{6} - \frac{b_f\sqrt{F_y}}{60\,t_f} \qquad (4)$$

where, b_f and t_f are the flange width and thickness, respectively, and F_y is the yield point stress for the pile. Upper and lower bounds of unity and zero, respectively, apply to C_i.

To obtain a comparison between the pile strength predicted by the two design alternatives and the experimentally and analytically verified finite element model, the factor of safety was removed from the AASHTO interaction equations. Local buckling of the elements forming the cross section and lateral torsional buckling of the entire section were assumed to be prevented, since the finite element model did not accomodate these types of behavior. When lateral displacement of the pile head was prevented, Alternatives One and Two predicted the same pile strength, which closely matched the finite element solution. However, when the pile head was displaced horizontally, both alternatives gave conservative results for the pile strength. As expected, Alternative One was excessively conservative for beam-columns having equivalent cantilever lengths corresponding to small slenderness ratios. Here, the lateral pile head displacement induces large flexural stresses, resulting in a small or no elastic reserve strength to resist the axial load. For this range of slenderness, Alternative Two predicted pile strengths that were more comparable with the finite element results, since this design alternative permits redistribution of internal forces, when sufficient pile ductility exists. When the slenderness ratios for idealized beam-columns corresponded to moderate and larger magnitudes, both alternatives predicted pile strengths that were in close agreement with the finite element results.

AASHTO - Cases B and C

Lateral displacement of the pile head can affect the capacity of the pile to transfer load to the ground (Case B). Soil displacements near the top of a friction pile greater than approximately 2% of the pile diameter, b, have been considered sufficient to disturb the interface between the pile and the soil. Therefore, the length of pile, along which the lateral displacement is greater than 0.02b, should be considered ineffective in transferring vertical load to the soil. This coupling of lateral displacement and vertical resistance was noted in the evaluation of the vertical load phase of the combined load tests on the one-tenth scale laboratory pile tests. The end bearing resistance of flexible piles was assumed not to be affected by lateral displacements at the pile head.

The authors of this paper believe that the horizontal displacement of the pile head does not affect the capacity of the ground to support the load (Case C). The AASHTO Specification [4] addresses group action for piles based on spacing and bearing strata strength requirements.

SUMMARY AND CONCLUSIONS

A rational approach to the design of integral abutment piles was developed from a study of pile behavior involving full scale field tests, laboratory model tests, and analytical investigations. To verify the predictions of pile response established from a previously developed finite element model, soil resistance and displacement relationships were experimentally established for vertical and lateral loads applied to the pile head. After the validity of the analytical model was established, pile strength comparisons were made with capacities predicted by Alternatives One and Two to determine the conservativeness of both design alternatives for all ranges of beam-column slenderness, associated with the capacity of the pile as a structural member (Case A). When the pile design is governed by Case A, Alternative One is recommended for piles that have a limited amount of moment-rotation capacity, while Alternative Two is recommended for piles that have a moment-rotation capacity that exceeds the moment-rotation demand at the plastic hinge locations, as confirmed by an established pile ductility criterion. When adequate pile ductility exists, Alternative Two will permit the safe design of integral abutment bridges that are significantly longer than those designed according to Alternative One.

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DESIGN OF STRESSED TIMBER T-BEAMS FOR HIGHWAY BRIDGES

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SYNOPSIS

The design of an innovative timber bridge incorporating laminated veneer lumber as stringers and a stressed red oak deck required experimental and analytical verification of several design assumptions. The experimental program includes the areas of load distribution, composite action, creep, laminate separation, and interlaminate slip. Experimentally obtained data is also used to confirm the accuracy of finite element and plate bending analytical models in order that a final design can be performed.

The use of timber as a bridge building material is experiencing a resurgence after years of virtual neglect. New manufactured timber products and the use of prestressed timber decks has brought timber design into a new era. West Virginia University Department of Civil Engineering recently was given the opportunity to contuibute to this rebirth of structural timber as a participant in the Regional Timber Bridge Conference held in Charleston, W. Va. in May 1988. Our participation in this conference included design, testing, and monitoring of a 75 foot long demonstration bridge which was erected during the conference. The most innovative aspects of this structure is the combined use of laminated veneer lumber (LVL), acting as the stringers, and longitudinal red oak decking transversly post-tensioned with the stringers. Together the decking and stringers act as a composite unit-in effect creating a timber "T-Beam." The bridge components were factory assembled into two 75'x 9' halves, trucked to the bridge site, craned into position, and then transverly tensioned as a unit.

Design and testing of the design are the topics of this presentation. The innovative nature of the structure meant that very little design guidance was available. Stressed timber decks are well accepted in Canada and are well covered by the Ontario Highway Bridge Design Code (OHBDC)- this source was extremely useful but even this code did not cover the use of LVL stringers acting compositely with the stressed deck. This lack of accepted guidelines made experimentation and testing a necessity prior to the final design. The preliminary designs required several assumptions about the behavior and performance of the bridge; the specific areas tested were:

1) Load distribution, 2) Composite action, 3) Creep, 4) Laminate separation and, 5) Slip.

The information gathered during these experiments was also used to verify the analytical modelling techniques used for the design. Both computerized finite element analysis and generalized plate equations were used to predict the deformations of the actual bridge and also those of the experimental model.

Figure 1 shows the preliminary (before experimentation) design of the bridge. Models were constructed which emphasized the potential problem areas and which allowed the study of the design assumptions. The following sections describe the experimental process and the

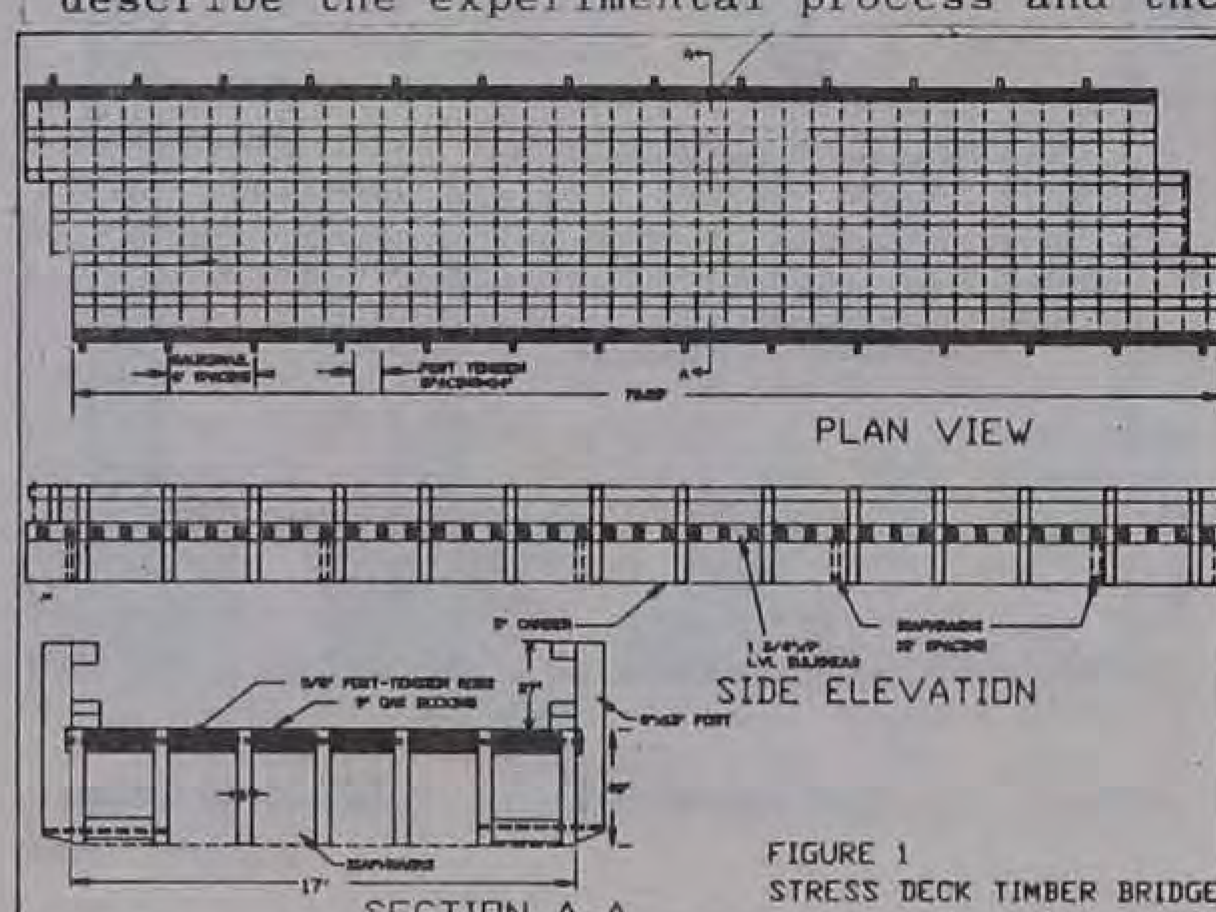

FIGURE 1
STRESS DECK TIMBER BRIDGE

analytical methods used for the final design.

LOAD DISTRIBUTION

The problem with which we were confronted was to determine the correct percentage of an applied load to assign to each stringer in order that the design could be made safe and economical and also to enable other designers of the bridge type to be confident in suggested load fraction values.

Experimentally, the share of an applied load on any of the stringers can be measured by comparison of the strain on that stringer to the strain on the other stringers. Analytical methods of determining the load sharing are based primarily on the stiffness of the deck in the transverse direction. The correct value of this transverse stiffness depends on the elastic properties of the deck planks and the ability of the transverse tensioning system to negate the effects of the many joints. For this reason transverse deflected profiles from experimental data are compared to deflected profiles generated by analytical methods using various transverse stiffnesses; the most accurate analytical methods can then be used to aid in the design of the actual bridge.

The bar graph of Figure 2 shows the sharing of the applied load, up to 16 kips, of the three adjacent stringers of the model.

At the 16 kip load level the stress in the stringer adjacent to the loaded stringer is approximately 25% of the stress in the stringer directly under the load. The transverse distribution profiles of Figure 3 describe the experimentally obtained deflections for a 10 kip load applied between two stringers and also the analytically obtained values from two methods also using 10 kip load.

A comparison of analytically generated profiles, in which the transverse stiffness is varied by Dy=1/25 Dx to Dy=1/10 Dx, to the experimentally obtained profile shows that the best approximation of Dy is 1/20 of Dx.

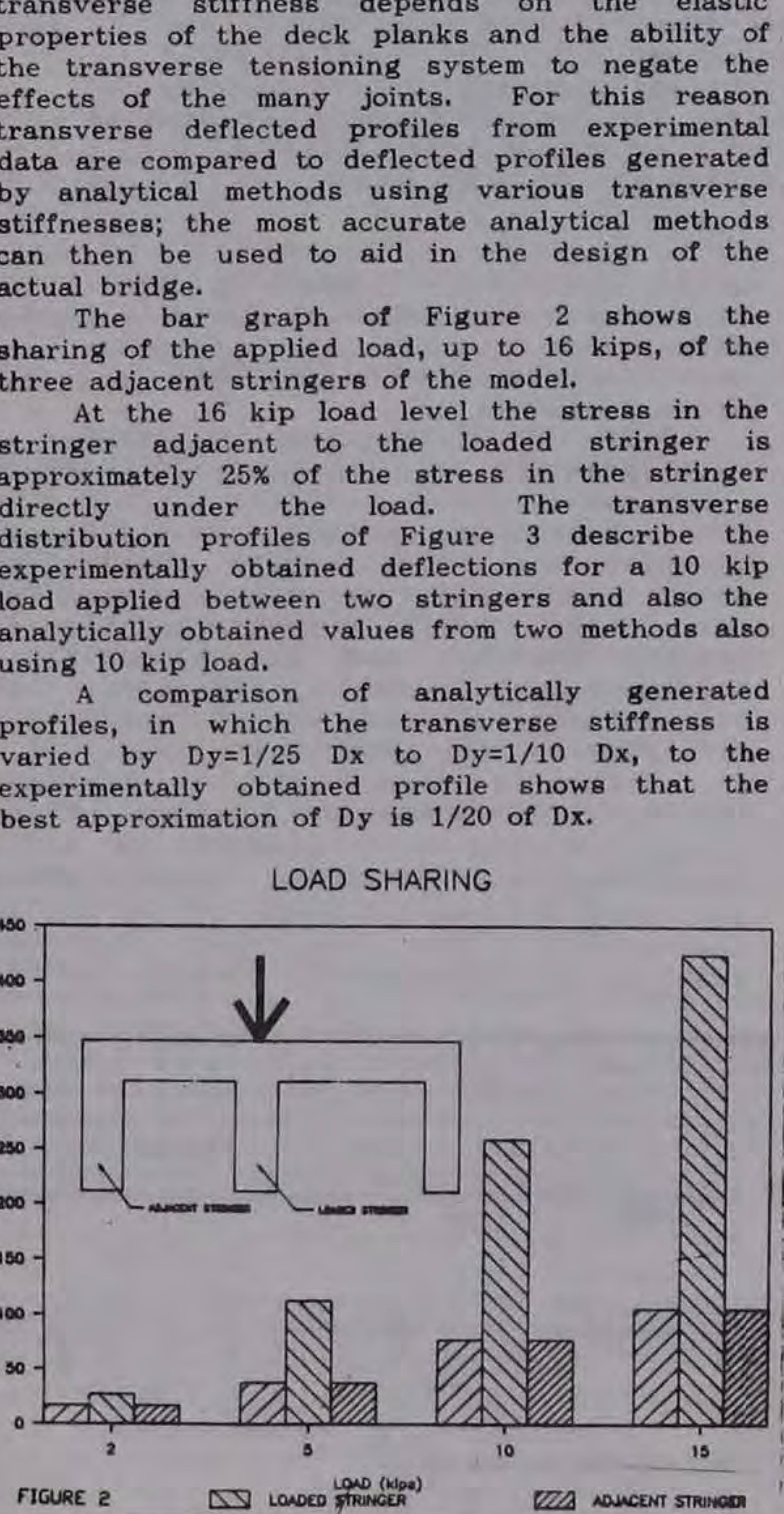

FIGURE 2

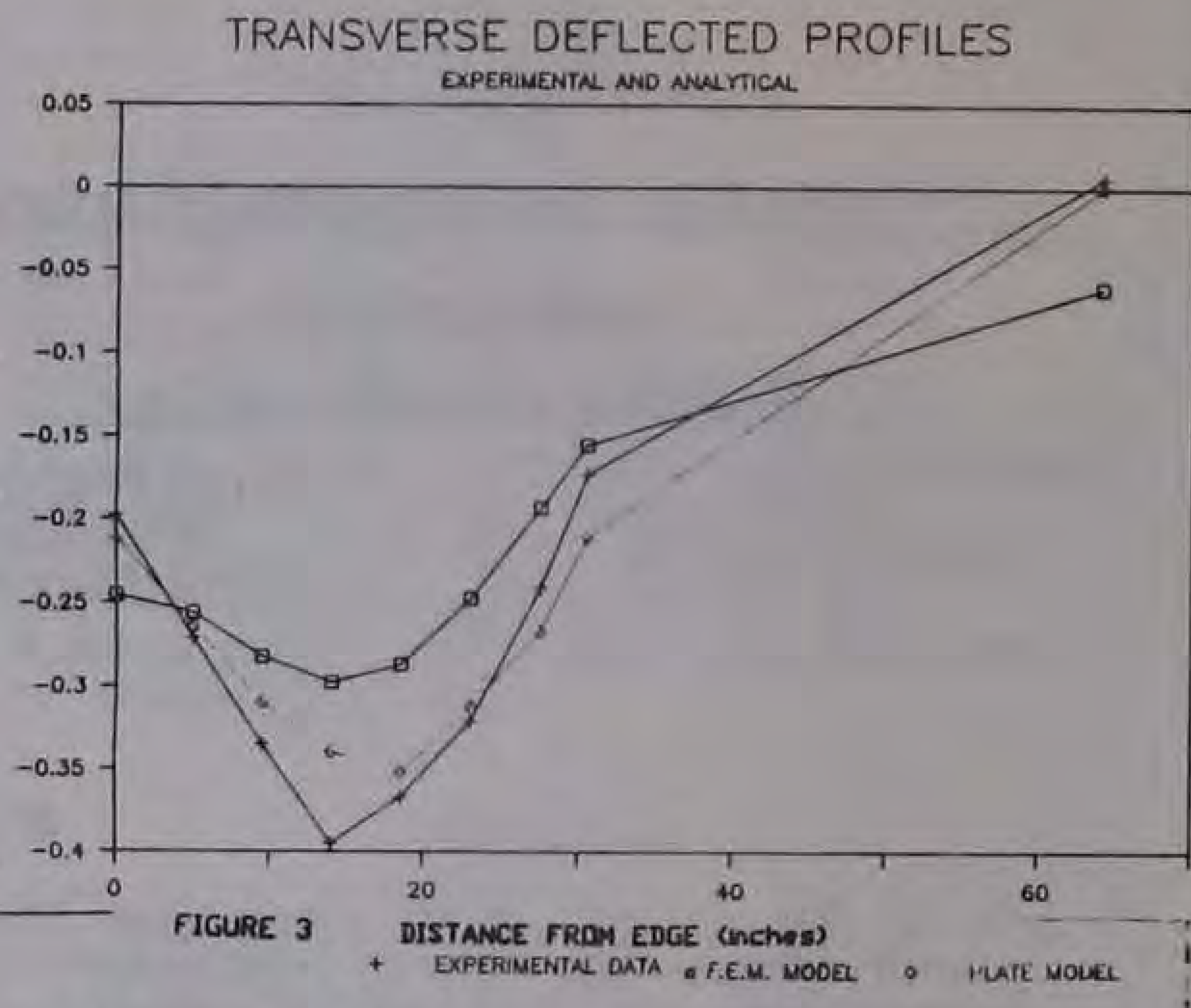

FIGURE 3

COMPOSITE ACTION

Another very major assumption of the preliminary design, and actually the premise upon which this design type was chosen, is that the combination of stringers and decking act as a composite unit. The intent of the design is to take advantage of the much stronger longitudinal bending strength of the decking by installing the decking so that its strong axis is parallel to the stringers and contributes to the overall longitudinal moment capacity of the structure. The post-tensioning system is intended to contribute to the overall strength by augmenting the transverse stiffness of the deck and thus its transverse moment capacity.

In this design the intended composite unit is a "T-Beam" with the stringer acting as the web and the decking acting as the flange. Of course a T-beam has a much larger moment of inertia than a simple rectangular section and consequently lower calculated bending stresses. The problem is to confirm that the configuration of the structure can be modelled as a T-beam and if so, what is the effective width of the flange.

The neutral axis of a rectangular member, subjected to a transverse load, would be coincident to its center of gravity; the neutral axis of a T-beam with the same rectangular section as its web would be substantially higher. Calculations, using the geometry of the model bridge, locate the neutral axis at 13.4 inches from the bottom fiber, assuming a flange width equal to the center to center spacing of the stringers. Since, by definition, the stress level at the neutral axis is zero, its location can easily be found graphically using experimentally obtained strains (see Figure 4). Expected deflections also can be calculated using the moment of inertia of a T-beam and compared to experimental deflections.

Clearly the compressive strain in the top fiber of the stringer is a decreasing fraction of the tensile strain in the lowermost fiber. At a load of 16 kips the height of the neutral axis is very nearly the same as that of the calculated neutral axis with flange width equal to the center to center spacing.

BENDING STRAINS IN T-BEAMS

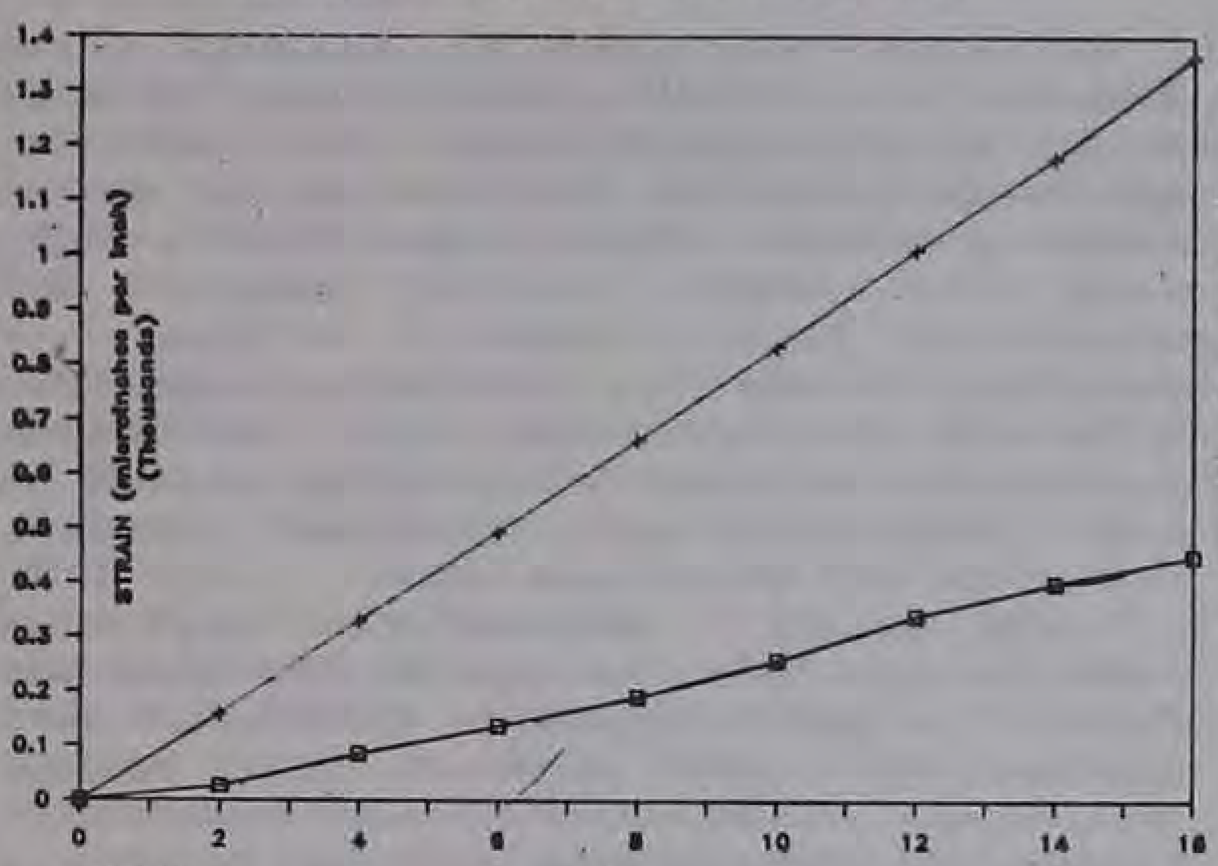

FIGURE 4
□ COMPRESSION AT TOP + TENSION AT BOTTOM

CREEP LOSSES

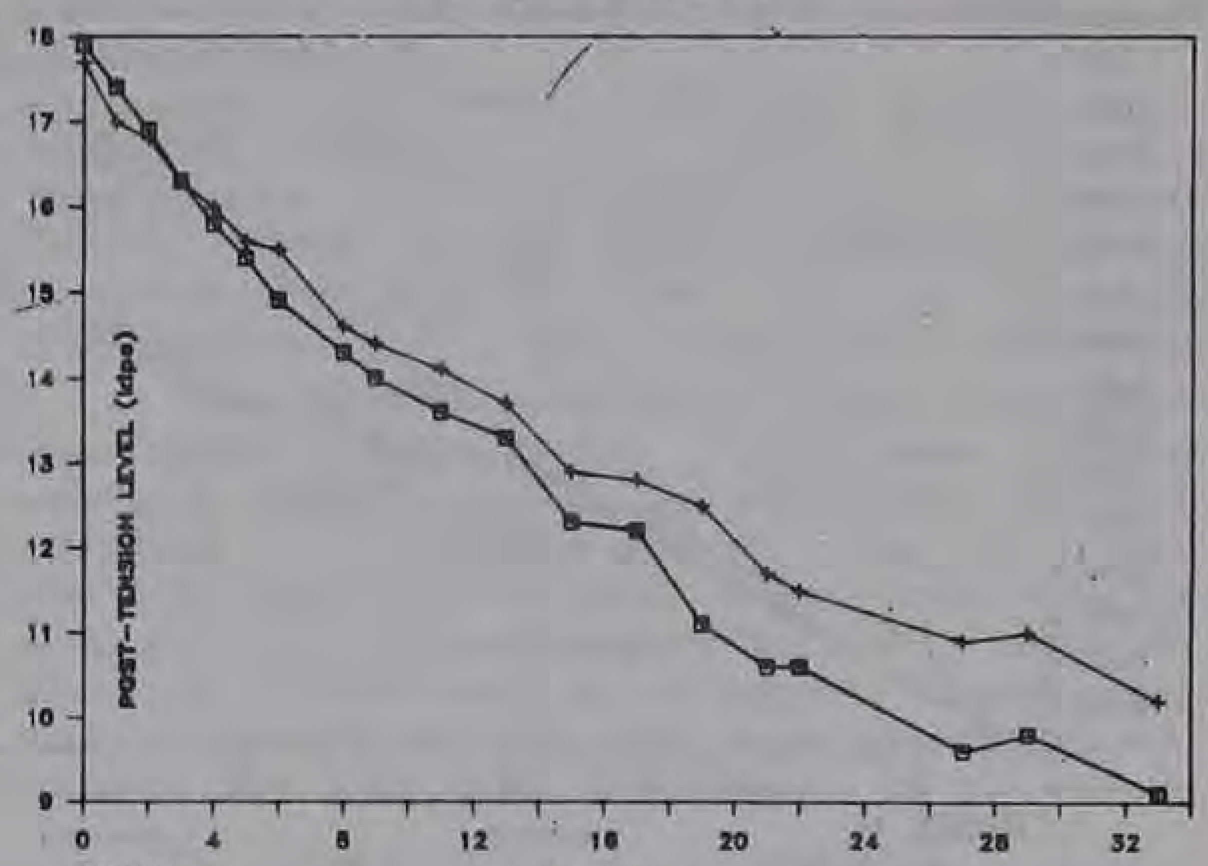

FIGURE 5
□ MIDSPAN LOAD CELL + EDGE LOAD CELL

CREEP

Creep is the time dependent relaxation of a stressed material. All construction materials experience creep to some degree but it is especially pronounced in stressed wood decks. One of the major disadvantages of a stressed timber deck is the maintenance required to check the post-tension level in the deck and the retensioning required to maintain an adequate operating level. Our experimental program has shown some of the problems which are likely to occur when the tension level is allowed to drop below 50% of the jacking tension level-excessive deflection, poor load sharing, and slip are the most severe. Fortunately, even at low levels of tension the bridge exhibited high load carrying capacity since the decking and rods create a flexible, net-like structure.

Compression of the decking was accomplished by tensioning the Dywidag rods with a hydraulic jack which was braced against the model bridge bulkhead. Two of the rods were longer than the others and load cells were installed between the bulkhead and the rod fastening mechanisms. Readings of the load level were taken immediately after the jacking sequence was complete and on a daily basis after that.

Figure 5 shows the rapid initial loss of compressive force in the system and the levelling off of this loss with time.

The stress level after losses in the model bridge are quite close to the level which OHBDC recommends as the maintenance level. Since the shear resistance of the decking is the primary load transfer mechanism of the system, and since this shear resistance depends upon friction and normal force provided by the post-tensioning system, this maintenance level is a critical value.

LAMINATE SEPARATION

Both slip and gapping between the deck laminates can be construed as evidence of inadequate normal force (post-tensioning) in the system. Gapping is the term we use to describe the separation of the lower portion of the deck laminates when subjected to a load. If conditions are ideal, a compressive stress should be created by the tensioned rods which is uniform and constant. The stress from the applied load creates a tensile force in the lower fibers of the deck which should be less than the compressive force created by the post-tension system, if gapping is to be prevented. If a gap is created between adjacent planks then the contact area of those two planks decreases and the potential for slip may increases. However, as the net compression decreases in the lower fibers it must be increasing in the upper fibers which should increase the shear resistance of that portion of the decking.

Cupping and warping of the individual deck laminations prior to assembly was extensive and the application of 20 kips of post-tension force (the equivalent of 150 psi average compression in the planks) was inadequate to create full contact between all of the planks.

Line A of Figure 6 shows the relationship of the load and decompression of the planks which were initially in contact with each other. These planks remained in contact with each other throughout the loading sequence. The gap between the planks which existed prior to loading increased (Figure 6, Line B) as expected and although the accuracy of these measurements is questionable, the trend is clear.

Clearly, the linearity of line A shows that the expansion (or more accurately, the decompression) of the planks which were in full contact conforms to $\Delta = PL/AE$ where P is the net compressive force and is decreasing as the load increases. Observation of the underside of the deck, while under load, showed that the planks initially in contact stayed in contact and that the gaps which existed prior to loading, increased in size. Slipping was not observed; this was our most serious concern but does not appear to be a problem even with the deck irregularities and gaps. The best conclusion that we can draw is that the areas in which compression is increased by the applied loading can adequately compensate for the loss of contact area and decompression caused by the same loading.

TRANSVERSE SPREADING

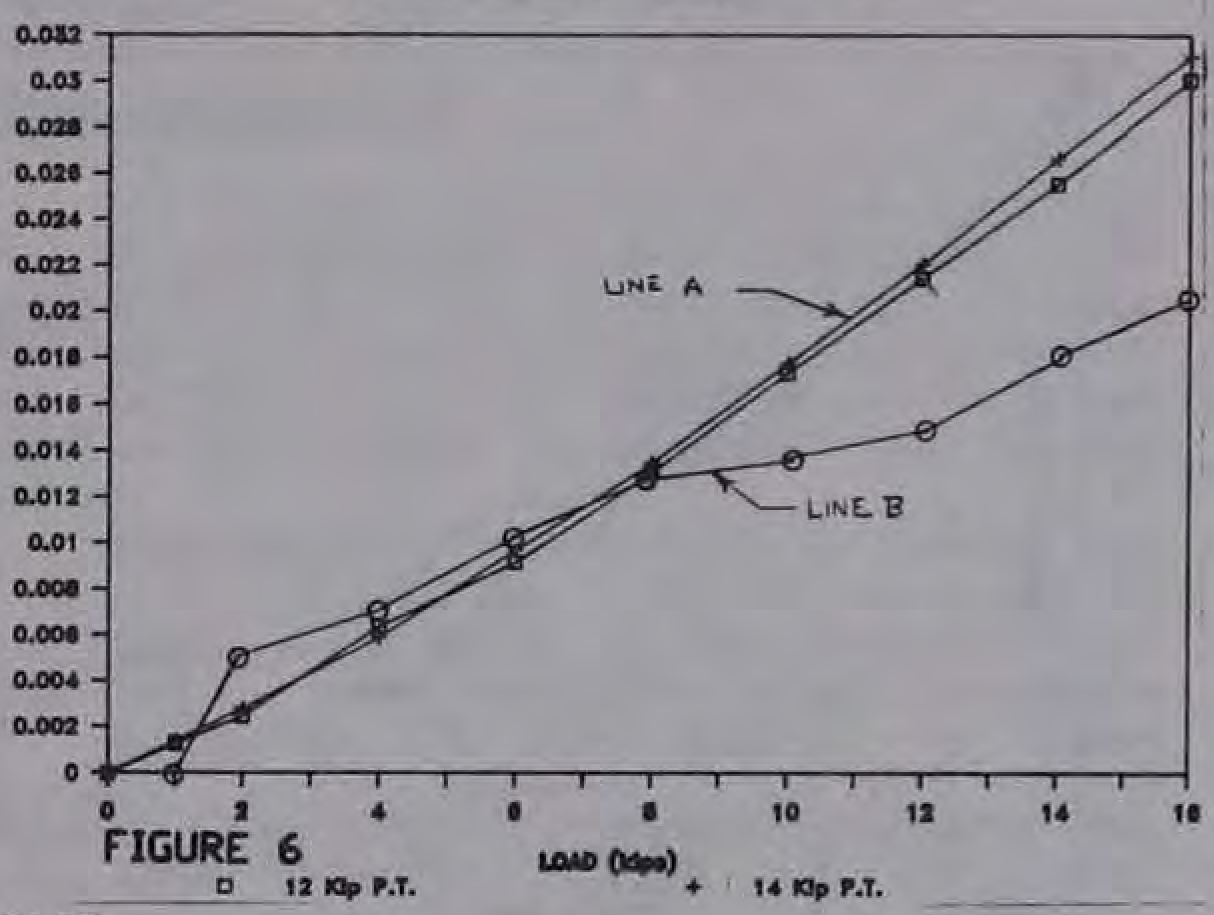

FIGURE 6

SLIP

Because the loads applied to the deck of the bridge are intended to be transferred to the stringer by the interlaminar shear strength of the system, the issue of slip is vital. Slip is defined as the abrupt vertical sliding of one lamination past another lamination or stringer when under load. Should slip occur in the bridge deck the live load may take an unintended path which relies on the shear capacity of the rod-wood connection and would most probably result in the cracking of the wearing course. One of the safety features of the preliminary design is the capacity of the deck planks to sustain an AASHTO HS-20 live load with zero post-tensioning (complete loss of tension in one rod) but this is certainly not the preferred method.

Slip is prevented by adequate interlaminar shear capacity. The OHBDC design equation $V = N_f \times d \times \mu$ where:

V = shear resistance in lb/in.
N_f = normal force from post-tension,
d = depth of deck members,
μ = coefficient of friction,

gives a value of shear resistance per inch which must be greater than the shear resulting from the expected loads.

Using conservative values for the variables in the OHBDC design equation, the shear capacity of the model bridge is approximately 190 lb/in.

From our definition of slip it will be apparent from deflected profiles if slip has occurred; any abrupt changes in curvature would constitute a slip. Dial gages attached to the stringer and indicating the deflection of an adjacent plank should yield values consistent with the expected deflected shape.

Short of loading the deck to failure there is no evidence of slip occurring. The transverse profiles are continuous (within the limits of experimental error) and the adjacent plank deflection test conforms well with the calculated values. No discontinuities are evident to indicate slip.

ANALYTICAL TECHNIQUES

In order to transfer the knowledge gained from the experimental program to the full scale bridge two mathematical methods were used. For our design project we have concentrated on two modelling methods: Finite Element Methods (FEM) using the ANSYS computer program and generalized plate equations developed by GangaRao. Results from both methods correlated quite well with experimental data. Both models have proven accuracy in predicting moments in both longitudinal and transverse direction, deflections, and distribution width.

FEM Method: Many different approaches were attempted before an accurate FEM model was found. The final model utilizes quadrilateral shell elements with both membrane and bending capabilities. Additionally, to create compatability between the deck elements and the stringer, a thin "interface" element was introduced. The model bridge was divided into a total of 800 elements with each node (4 nodes per element) having 6 degrees of freedom. Reduction of the number of simultaneous equations was possible by disallowing deflections in the X and Y directions as well as rotation about the Z axis. Symmetry of the model and support boundary conditions further reduced the number of equations.

The natural orthotropy of timber requires 9 separate elastic constants to calculate complete state of stresses and deflections. For this particular ANSYS FEM model only E_y, E_x, G_{xy} and μ were required since only these values affect the calculation which we needed. Figure 7 is a graph of three models in which the thickness (and thus the stiffness) of the interface elements was varied. Although the maximum value of deflection, near the center of the loaded span, is not quite as large as the experimental value (Figure 3) (0.27" vs .38"), the shape of the profile and the deflection of the stringers match very well. The value of the transverse stiffness for model 3 was taken as 1/20 E_x. Since the deflection of the FEM model was less than that of the experimental model the conclusion can be made that the global stiffness of the deck in the transverse direction is slightly less than the estimated stiffness of the individual planks. Also, typical finite element formulations of plate

TRANSVERSE DEFLECTED PROFILE

FEM MODELS

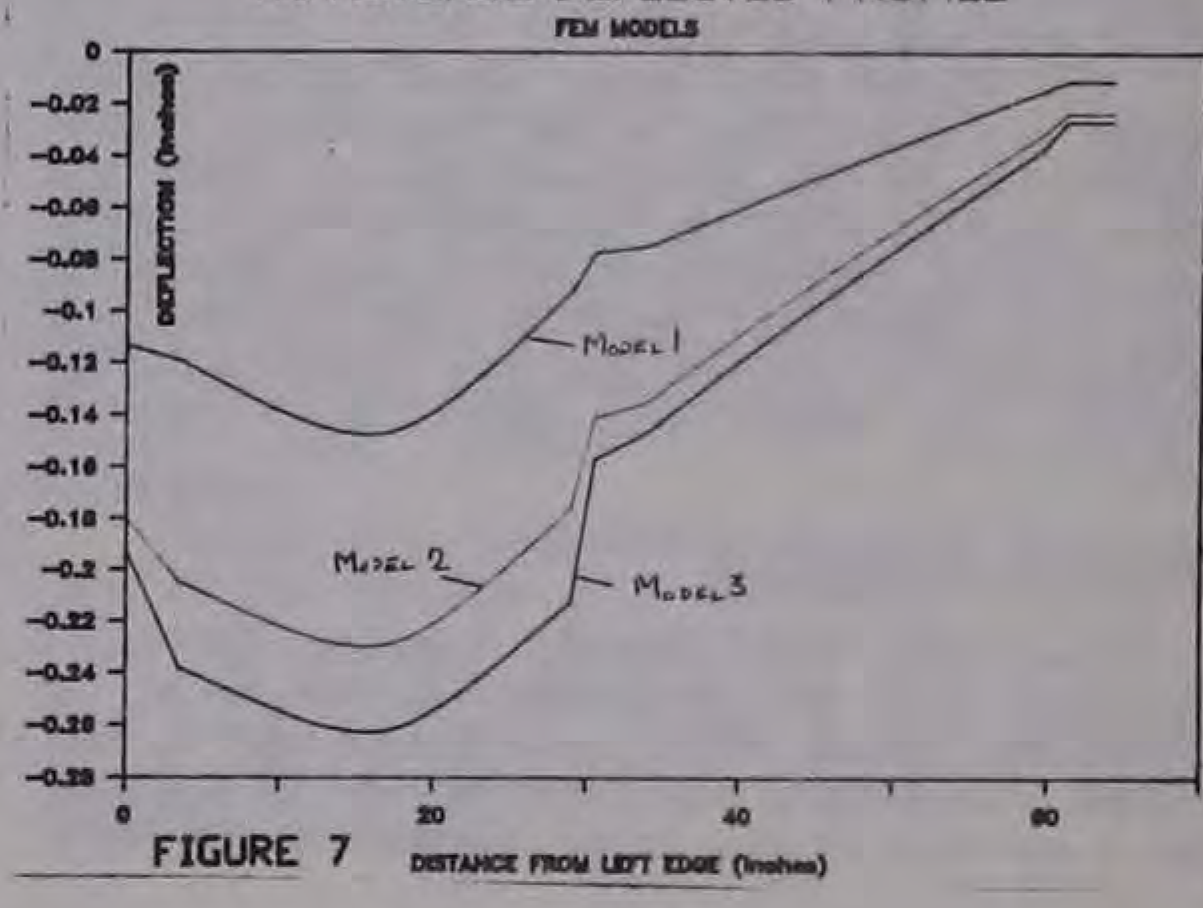

FIGURE 7

bending equations yield stiffer elements; hence lower deflections.

Generalized Plate Equations: A system of design equations was developed by GangaRao, specifically for use with steel grid decks but which can be applied to any orthotropic bridge deck. These design equations account for most of the parameters affecting the plate behavior of the deck such as the aspect ratio of the deck, the orthotropic properties of the deck, loading conditions, and number of lanes as well as stringer spacing. Unfortunately, the increase in accuracy must be accompanied by some loss in simplicity.

To test the accuracy of the application of the technique, we used the same input as that of the FEM Model discussed earlier. Since the applied load on the model structure simulated only one rear wheel (the model bridge was not wide enough to accommodate the standard 6' spacing of wheel loads), the loading of the analytical model was a combination of symmetric and antisymmetric cases. As in the FEM model, the input value of E_T modulus of elasticity in the transverse direction was varied to produce the best correlation with the experimental value. The best correlation of experimental deflections and analytically determined deflections was achieved when a value of $E_T = 1/25\ E_L$.

CONCLUSIONS

The experimental program at W.V.U. Civil Engineering was undertaken to verify assumptions made for one particular, innovative design of a timber stressed deck bridge. All of the major uncertainties of the design have been resolved satisfactorily. The range of areas covered by the research program is quite broad; unfortunately time constraints have limited the depth of the research program. Despite these limitations, we feel that we can contribute to the state of the art, at least in an introductory capacity.

Conclusions: Foremost among the conclusions we can derive from our experimental program is that many of the guidelines established by the OHBDC are quite applicable to our red oak - LVL bridge system. Creep behavior of the model, despite construction as a "worst case" situation, was accurately predicted. The shear capacity of the structure, calculated using the OHBDC design equation, exceeded the applied loads, including a simulated HS-20 AASHTO load. Although the OHBDC did not include all the guidelines for the stringer-deck combination of our particular system, it proved to be a most valuable design aid and would be an excellent basis upon which to build.

The research program at W.V.U. Civil Engineering accomplished more than verification of the stressed deck section of the Ontario Code. Most importantly, the combination of longitudinal decking and stringers has been shown to provide composite action. This behavior allows the strong axis of the timber decking to be used to its greatest advantage while the weaker, tangential axis of the timber is made more rigid by post-tensioning. Although our design could not take full advantage of this asset due to the time constraints of the project, later designs should be able to reduce the stringer depth or increase the stringer spacing.

Load distribution, although tested in a simplified manner, was found to be less conservative than the AASHTO would allow in Section 3.25.2 of the Bridge Specifications for longitudinal decking; this is an expected consequence of the stressing operation.

The performance of the system as a whole was quite good; certain components showed excellent characteristics. In particular, the LVL stringers and the Dywidag rods worked very well. The LVL material has good strong axis bending strength and when adequately braced in the weak direction served well as stringers. Field tests and monitoring of the full scale structure will measure the long term performance of the entire system.

DEVELOPMENT OF A BRIDGE MONITORING TECHNIQUE

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SYNOPSIS

Recent highway bridge collapses and concern over the structural health of those in service indicates the desirability for bridge monitoring techniques. Vibrational monitoring has been successfully used to evaluate machinery, aircraft and power plants. This paper presents the initial results of a study to apply this technology to the continuous monitoring of bridges. A physical bridge model with a motorized vehicle has been used to detect changes in the vibrational signature due to alterations in the structure.

The average age of the nation's bridges is increasing. Many of these bridges are subject to greater vehicle weights than anticipated. Additionally, the number of heavy vehicles on many bridges is much larger than originally predicted. Economic constraints will not allow replacement of all of the nation's bridges which are judged deficient, either due to age or use.

Bridge inspection is generally carried out at two year intervals. However, problems may develop between inspections or may not be perceptible by visual observation during the inspection. In some cases, these problems may result in collapse.

Work is underway at the University of Connecticut to develop an automated monitoring system for continual surveillance of bridge structures. The system will function under normal traffic operations. This paper reports on the first phase of this work.

Basis of Monitoring System - Bridges are dynamically loaded structures. Vehicle loading subjects the bridge to significant and continual vibratory motion.

Vibrational monitoring has been used for machinery, power plants, aircraft, etc. This technology utilizes piezoelectric sensors to provide acceleration information. A computer based analysis then provides a vibrational signature for the equipment or structure. Since vibrational characteristics are a function of stiffness, changes due to failure of a component are reflected by changes to the signature.

Monitoring systems for bridges must be capable of detecting small changes in the structure which can lead to failure mechanisms. It is not possible to use deflections or strains for this purpose.

Previous bridge vibration studies have been concerned with either experimental studies to determine the accelerations, deflections, mode shapes and natural frequencies for existing bridges subject to vehicle or wind loading, or analytical approaches to estimate the natural frequencies and mode shapes. Signature analysis under normal operating conditions as done for machinery, power plants, etc. has not been used to evaluate bridge performance.

Work at the University of Connecticut (Refs. 1 - 3), including the field monitoring of a continuous 4 span bridge in Connecticut, has determined that monitoring accelerations as now done for machinery will indicate when a change in the bridge structure occurs.

Model Study - A model study has been used to supply preliminary information on the use of vibration measurement for bridge monitoring (4). The model is providing (1) additional information on what is needed to determine vibrational signatures for bridges; (2) analysis techniques for determining the changes in the signatures when cracks, support movements or loosened connections will initiate failures.

The model was made from aluminum angles and plates, as shown in Fig. 1. The depth is 2.5 in. and the length is 15 feet. The model has been tested as both a single and two span bridge. Three different vehicles have been pulled across by a motor on one end. The vehicle mass, speed, and roadway roughness have been varied.

During vibration measurements, accelerometer response is input into a dual-channel spectrum analyzer and processed using random data analysis techniques. Integrations can be performed to convert accelerations to velocities or displacements. Response spectra are obtained in the form of autospectral functions for each measurement station, and represent the mean square response magnitude as a function of frequency. Subsequent processing allows determination of natural frequencies, damping, and relative response magnitudes. Cross-spectral functions which provide necessary phase information between measurement points must be obtained when generating mode shapes.

During the tests, vehicle mass, vehicle speed, and roadway roughness were varied. In general, the natural frequencies were not affected by these variables when the ratio of single vehicle to bridge mass was in the range expected for real bridges. However, the response magnitude and comparative intensity of resonant peaks can be significantly influenced by these parameters.

Typical acceleration spectra in the form of autospectral functions are shown in Figs. 2 through 5. These plots are based on accelerometers at the midspan (Figs. 2 & 3) and at 0.1 times the length from the end (Figs. 4 & 5). Both the natural frequencies and response magnitudes changed.

The four lowest natural frequencies for the simply supported single span bridge are shown in Table 1. The mode shapes are also noted, with three representing flexural and one representing torsional, or twisted, deformations. Two sets of tests are shown, one in which the bridge was uncracked and one in which one of the two girders had a crack which extended through the bottom flange on both sides of the web and then up into the web. The resulting change in the overall bridge moment of inertia at the crack was 25.8 percent. The change in moment of inertia is smaller than might be expected since the neutral axis of the combined cross sections shifts due to the crack.

Work is now underway to review other failure mechanisms through additional model tests. The experimental data will then be fully evaluated to develop analytical monitoring techniques. Both natural frequencies and relative response magnitudes, as well as mode shapes, are possible indicators of when problems are developing. The next phase will include field monitoring on bridges in Connecticut.

Acknowledgments - The writers are grateful to the Connecticut Department of Transportation for their support of this investigation. Joe Gartner, Professor of Mechanical Engineering, offered advice and equipment for the earlier part of the model studies. John Fikiet, Head of the Electronics Shop in the School of Engineering, has offered much advice and help in the investigation. The Engineering Shop assisted in the fabrication of the model. The overall concept for the investigation was first proposed by John Judd, President of Vibra-Metrics, Hamden, Connecticut.

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TABLE 1
Frequency Changes for Single Span Bridge

Mode	Uncracked Beam Frequency	Cracked Beam Frequency	Percent Change
Bending 1	8.12 Hz	7.31 Hz	10.0
Bending 2	31.3 Hz	31.1 Hz	0.6
Torsion 1	46.0 Hz	39.5 Hz	14.1
Bending 3	70.1 Hz	66.1 Hz	5.7

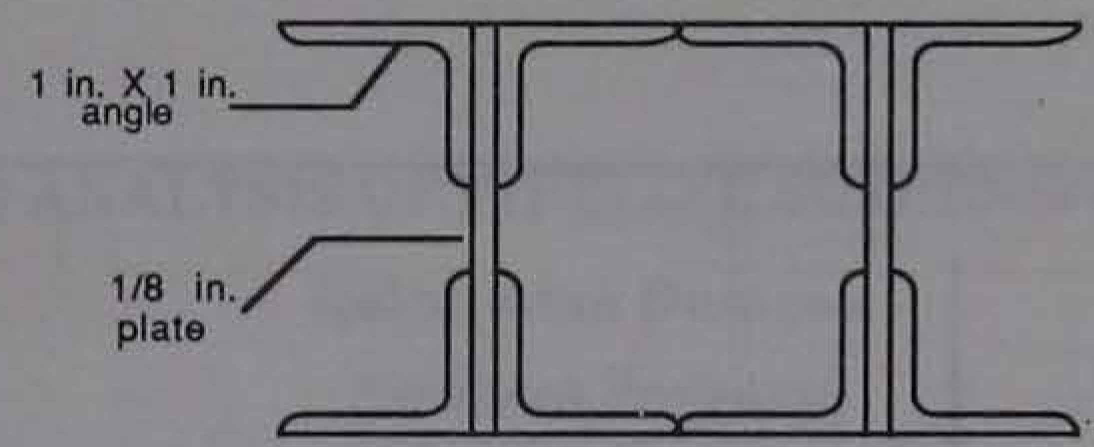

Figure 1. Cross-section of Bridge Model

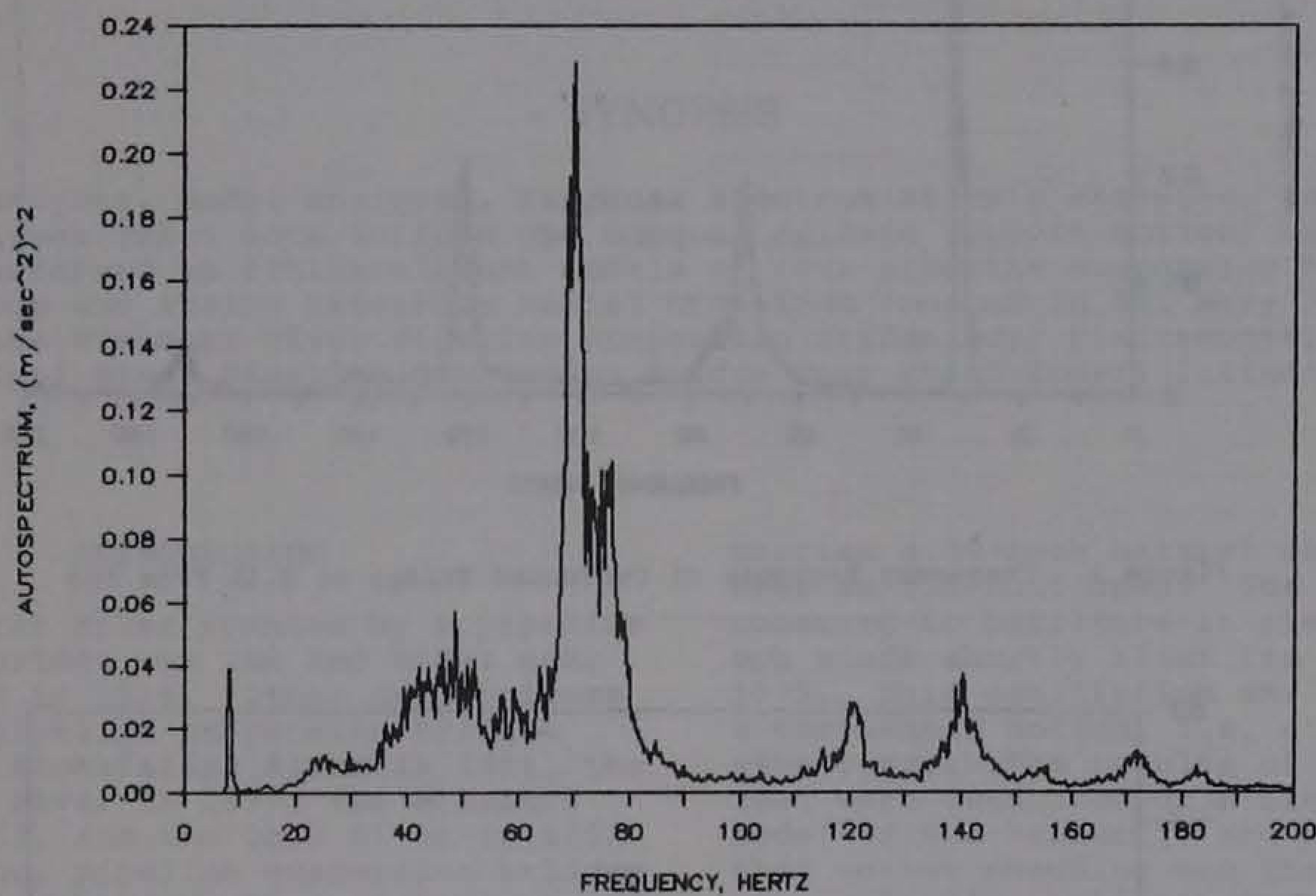

Figure 2. Frequency Response of Uncracked Bridge at Midspan

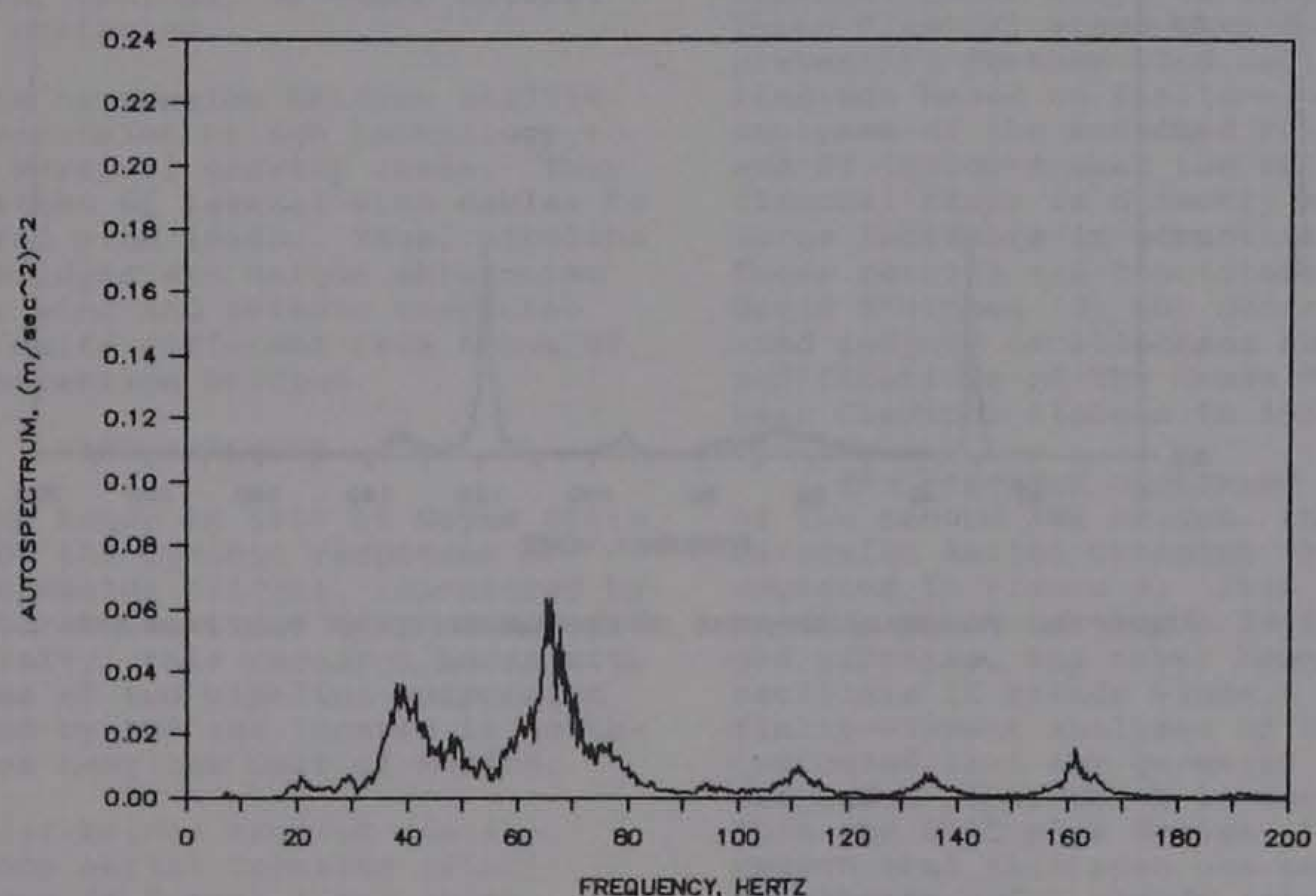

Figure 3. Frequency Response of Cracked Bridge at Midspan

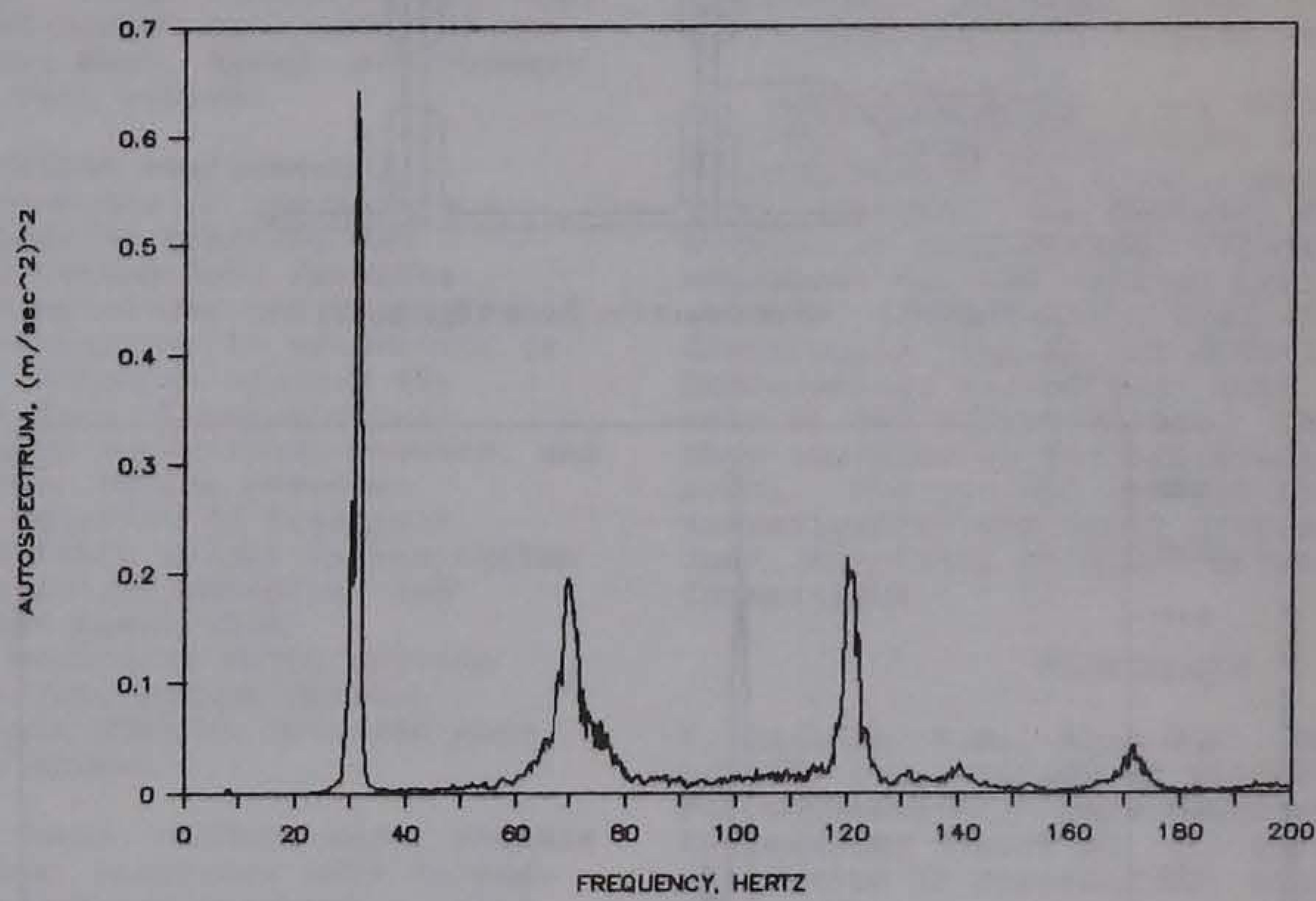

Figure 4. Frequency Response of Uncracked Bridge at 0.1L From End

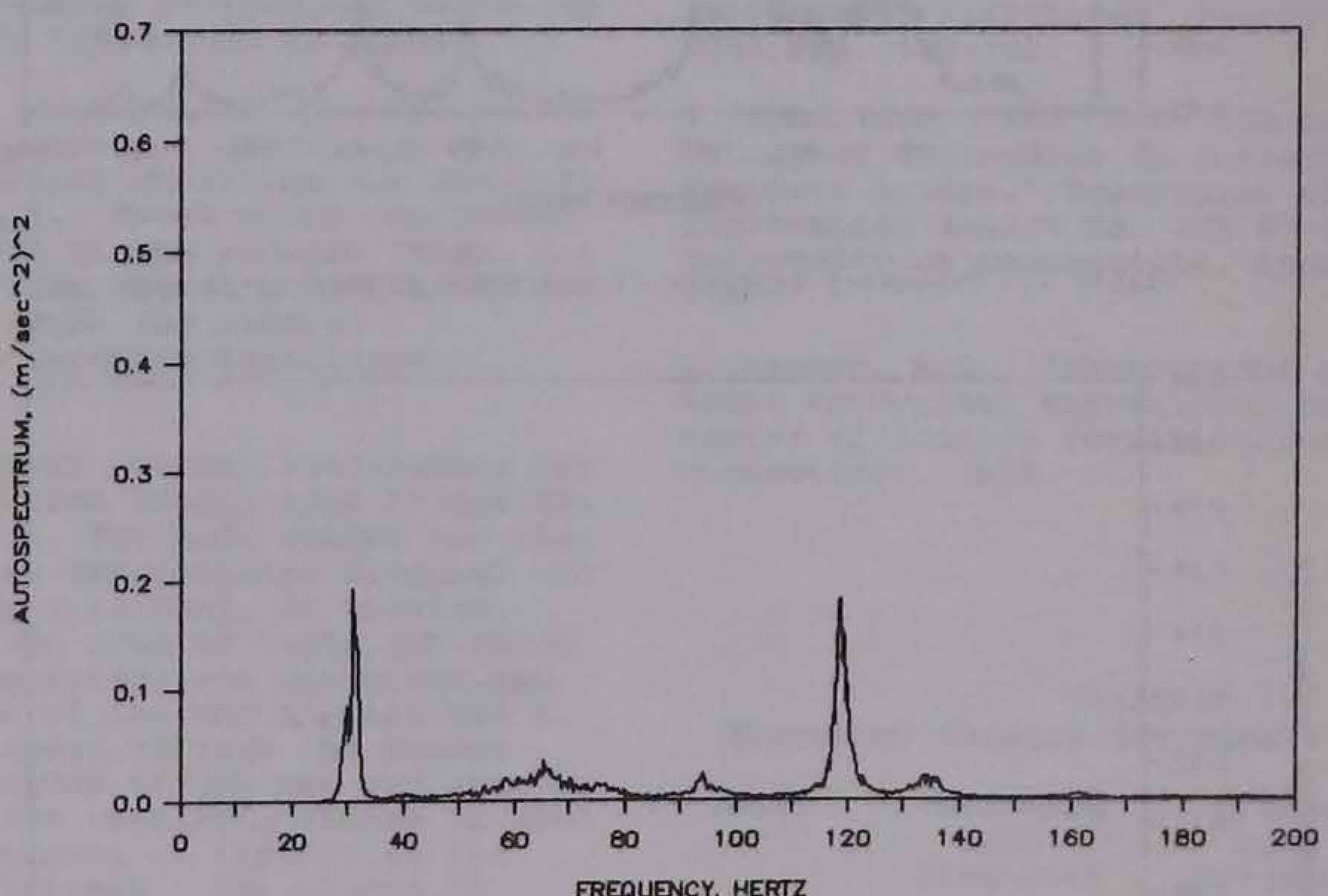

Figure 5. Frequency Response of Cracked Bridge at 0.1L From End

DYNAMIC ANALYSIS OF PIPELINE SUSPENSION BRIDGES

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SYNOPSIS

Wind analyses, modal analyses, response spectrum seismic analyses, and time history seismic analyses (with both uniform and unequal seismic support motion) have been and are being performed on finite-element models of four pipeline suspension bridges: the Patterson Loop and Avalon Extension Aerial Crossings located in St. Mary Parrish, Louisiana, the Missouri River Pipeline Suspension Bridge near Plattsmouth, Nebraska, and the Mississippi River Pipeline Suspension Bridge near Grand Tower, Illinois.

INTRODUCTION

The first river spanned by a pipeline suspension bridge was the Red River near Byers, Texas in 1926. Other major rivers spanned by pipeline suspension bridges include the Atchafalaya River in 1951, the Mississippi River in 1955, the Missouri River in 1957, and the Ohio River in 1962. By definition, pipeline suspension bridges are suspension bridges that carry segments of natural gas, oil, or water pipelines (with one, two, or sometimes three pipes) across rivers, canyons, or other natural or man-made obstacles.

Pipeline suspension bridges utilize classical suspension bridge technology to support the vertical gravity loads. They also use systems of lateral wind cables to resist lateral wind loads. Thus, pipeline suspension bridges are unique structures with dynamic wind and seismic responses that may be quite different from those of vehicular suspension bridges.

WIND ANALYSES

Research began in 1985 at Wayne State University on the dynamic responses of pipeline suspension bridges. Sponsored by American Natural Resources (ANR) and Wayne State University, this research began with wind analyses of two pipeline suspension bridges owned by ANR and located in southern Louisiana near the Gulf of Mexico.

The first bridge studied was the Patterson Loop Aerial Crossing (PLAC) which is shown in Figure 1 and which carries a 30-inch natural gas pipeline over an 850-foot span. The PLAC was observed to oscillate in steady 5- to 8-mph winds shortly after its completion in 1975. This oscillation was in the form of a three-node motion, i.e. a 1 1/2 sine-wave curve. The results of modal analyses that were conducted on a finite-element model of the bridge [1 and 2] indicated that vortex shedding was the probable cause of this vertical wind excitation.

In 1981, the PLAC was modified with diagonal cable stays as shown in Figure 2. These diagonal stays have succeeded in preventing further wind oscillations. Findings based on finite-element modal analyses of the modified PLAC design [1 and 2] indicate that the success of the diagonal stays is directly attributable to large increases in structural damping. These results are consistent with those of David Steinman [3] who described similar wind induced oscillations and structural modifications of the Coosa River Bridge near Clayton, Alabama in 1952.

The research continued with a study of the second ANR bridge, the Avalon Extension Aerial Crossing (AEAC), which is depicted in Figure 3. This 1008-foot bridge, which carries a 20-inch natural gas pipeline, has never been observed to oscillate in steady winds. The results of finite-element analyses of this bridge [2] indicated that the geometry of the AEAC suspended pipe, which is much more rigid than the PLAC pipe design, is the likely reason that this span has never been seen to vibrate under steady lateral winds.

RESPONSE SPECTRUM SEISMIC ANALYSES

Response spectrum seismic analyses were performed on the finite element models of the modified PLAC and the AEAC, and on finite-element models representing two major pipeline suspension bridges: the 1500-foot Missouri River Pipeline Bridge (MoRPB) depicted in Figure 4 and the 2150-foot Mississippi River Pipeline Bridge (MiRPB) shown in Figure 5. While the latter is a nearly symmetric structure carrying twin 30-inch pipes over the main span only, the former is asymmetric with twin 30-inch pipes suspended over the main span and over one of the side-spans.

The towers in the MoRPB and the MiRPB have very high stiffnesses relative to the suspension cables and thus the towers could not be included in the models of these bridges. At the point where the main cables are attached at the top of the towers, the finite-element models use rollers to represent the towers in the longitudinal direction (the weak direction of the towers) and pins to represent the towers in the lateral direction (the strong direction of the towers).

Modes of Vibration

For the four finite-element bridge models, the principal modes of vibration in the vertical direction consisted of either two- or three-node motions (full sine-wave or 1 1/2 sine-wave curves). For the modified PLAC, the AEAC, the MoRPB, and the MiRPB models, the two-node mode shapes had frequencies of 0.52, 0.24, 0.31, and 0.28 cycles per second (cps), respectively, while the three-node modes had frequencies of 0.43, 0.37, 0.21, and 0.22 cps, respectively.

In the lateral direction, the principal modes consisted of a symmetric one-node motion (a symmetric one-half sine-wave curve) for the modified PLAC and the AEAC, an asymmetric one-node motion for the MoRPB, and a combination of one-node and three-node motion for the MiRPB. The frequencies for these principal modes were 0.26, 0.16, 0.13, and 0.13 cps for the modified PLAC, the AEAC, the MoRPB, and the MiRPB models, respectively.

Response Spectra

The response spectra used for the seismic analyses were the Horizontal and Vertical Design Spectra of the Atomic Energy Commission [4]. These spectra were chosen because they are widely accepted and used by earthquake engineers around the world, and because they were generated using critical damping ratios as low as 0.5%. Based on the wind analysis studies of the PLAC and the AEAC, a conservative damping ratio of 0.5% was used for all the response spectrum seismic analyses that were conducted.

The two ANR spans and the MoRPB are located in low-risk seismic zones and thus the relatively low maximum ground acceleration of 0.09g, which is recommended by AASHTO [5] for highway bridges in low-risk seismic zones, was used for the response spectrum seismic analyses of these spans. The MiRPB lies in a high-risk seismic zone and was analyzed using the higher maximum ground acceleration of 0.5g recommended by AASHTO. The Horizontal Design Spectra was applied horizontally to the bridge models in the longitudinal direction, the lateral direction, and at an angle of 45 degrees to the longitudinal and lateral directions. The Vertical Design Spectra was applied in the vertical direction only.

Analysis Results

The most critical members under seismic loading of the modified PLAC (Figure 2) and the AEAC (Figure 3) are the towers under longitudinal ground motion. In both bridges the towers consist of three box-shaped members that form an inverted "Y". For the towers in the modified PLAC, the seismic stresses could reach 14.5% of the yield stress under a maximum ground acceleration of 0.09g and the total stresses including dead load could approach 22.9% of the yield value. In the AEAC towers, the seismic stresses may reach 22.0% of the yield stress and the total stresses may approach 32.5% of the yield value.

The most critical members in the MoRPB and the MiRPB are the end segments of the suspended pipes. The pipes on the west shore of the MoRPB (the left side of Figure 4) are carried from the main span to the ground by a truss that spans from the tower to the bluff. The pipes are anchored at the bluff by concrete thrust blocks and the stresses at these points could reach 24.2 ksi under a maximum longitudinal ground acceleration of 0.09g. The vertical pipe segments on the east shore of the MiRPB (the right side of Figure 5) could reach 42.9 ksi under a maximum longitudinal acceleration of 0.5g.

Also critical for both the MoRPB and the MiRPB are the diagonal cables that run from the main suspension cables to the lateral wind cables. These cables provide additional stiffness in both the vertical and longitudinal directions. The MoRPB diagonal cables may reach 9.3% and 7.2% of their breaking strength under 0.09g longitudinal and vertical motion, respectively. For 0.5g acceleration of the MiRPB, the diagonal cables may reach 10.1% and 21.9% of their breaking strengths under longitudinal and vertical motion, respectively.

TIME HISTORY SEISMIC ANALYSES

Time history seismic analyses using both uniform and unequal seismic support motion are currently being conducted on the MoRPB and MiRPB models. Previous research by the author in the area of steel deck arch bridges [6] indicated that under longitudinal loading, the arch seismic stresses due to unequal motion could be 25% to 150% larger than the stresses under uniform motion. The same may also be true for the main cables in pipeline suspension bridges. Thus the effects of longitudinal ground motion are being determined first.

The earthquake accelerograms that are being used for these analyses include the the two directions of horizontal motion for the 1940 El Centro Earthquake, for the 1952 Taft Earthquake, and for the 1971 San Fernando Earthquake. The analyses are being performed using time steps of 0.01 seconds (s) with displacements recovered at each step and with stresses recovered at intervals of 0.2 s. The first 30 s of each accelerogram will be used for the ground motion input. To determine the efficacy of using 0.01 s time steps, analyses will also be conducted using a much smaller time step of 0.004 s over the first 12 s of each accelerogram.

Under uniform seismic support motion, each accelerogram will be applied to all bridge supports simultaneously. Under unequal seismic support motion, the ground motion histories will be applied with time lags between adjacent supports. These time lags will be multiples of 0.01 s and will be based on a shock wave speed of approximately 4000 feet per second.

Preliminary results for longitudinal input indicate large increases in the main cable axial stresses due to unequal motion as opposed to uniform motion for both the MoRPB and the MiRPB. Analyses using uniform and unequal seismic support motion in the lateral and vertical directions will also be performed on the MoRPB and MiRPB models. Similar analyses are also planned for the PLAC and AEAC models under longitudinal, lateral, and vertical motion.

TOWER ANALYSES

Based on the response spectrum seismic results for the PLAC and AEAC towers, and because the MoRPB and MiRPB towers were not included in the models of these bridges, detailed seismic analyses of the towers in all four bridges are currently being performed. The PLAC and AEAC towers will be modeled using several finite-elements and mass points for each of the three box-shaped segments in the towers. The MoRPB and MiRPB towers are trusses that lie in a single two-dimensional transverse plane. All of the elements in these towers will be represented by equivalent beam or truss elements with masses lumped at all element node points.

All of the towers in the four bridges studied have pairs of wind booms that support the lateral wind cables. These booms consist of individual steel tubes for the PLAC and AEAC towers, and trusses for the MoRPB and MiRPB towers. The booms will be modeled as single truss elements for the PLAC and AEAC tower models, and as systems of truss and beam elements for the MoRPB and MiRPB tower models.

The boundary conditions at the base of each tower model will be the same as in the actual towers. The restraints at the top of the tower model and at the ends of each wind boom will consist of truss elements with the same axial stiffnesses as the main suspension cables and the wind cables, respectively, and with the same initial dead load axial stresses.

Modal analyses and response spectrum seismic analyses will be performed on each tower model using the same procedure that was followed for the bridge models. Time history seismic analyses may also be performed on the tower models.

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Figure 1 Original Patterson Loop Aerial Crossing

Figure 2 Modified Patterson Loop Aerial Crossing

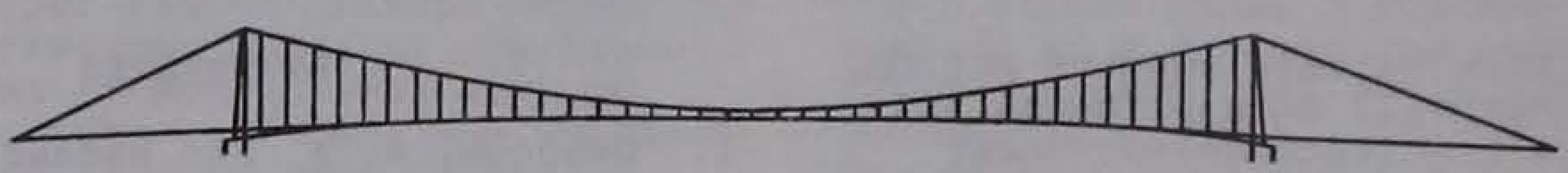

Figure 3 Avalon Extension Aerial Crossing

Figure 4 Missouri River Pipeline Suspension Bridge

Figure 5 Mississippi River Pipeline Suspension Bridge

DYNAMIC INSTRUMENTATION OF A CABLE STAYED BRIDGE

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SYNOPSIS

The dynamic instrumentation of a cable-stayed, segmental prestressed concrete bridge currently under construction is described. The instrumentation program, executed during construction, will measure vibration characteristics of several sub-units of the bridge and the completed structure. Instrumentation consists of accelerometers and anemometers. Response data will be automatically recorded. Subsequently natural frequencies, damping characteristics, and mode shapes will be estimated from the field data, and compared with estimates obtained from finite element models.

INTRODUCTION

Long span bridges are subjected to a variety of dynamic excitations. The most critical dynamic responses are likely to be caused by natural hazards, specifically wind and earthquake loadings. It is essential that the dynamic characteristics of such bridges be determined. Toward this end, full scale dynamic testing is needed for evaluation of actual in-service performance, to serve as a benchmark for existing computer models used in design, and to provide valuable comparisons with small scale model studies.

Cable-stayed bridges have become increasingly popular and are important lifeline structures. Several studies have addressed the need for reliable estimates of natural frequencies, mode shapes and damping characteristics of cable-stayed bridges,but considerable uncertainty remains in defining the appropriate system characteristics, such as appropriate damping, influence of local flexibility on overall system response, and soil structure interaction.

Field studies concerned with measuring the dynamic response of such bridges can contribute even more valuable information if construction phase studies can be conducted. During this period, measurements can be taken on portions of the bridge which will provide data not available on the completed structure. In particular, piers and pylons can be studied as independent structures, and a more accurate assessment of the substructure characteristics thereby obtained. Moreover, cable-stayed bridges are particularly vulnerable during the construction phase when unsupported cantilever spans have relatively little stiffness. Thus, data obtained during this phase of the structure's life contribute valuable insight into potential dynamic response during severe natural events.

A number of earlier studies have addressed the need for reliable data on the dynamic characteristics of long span bridges and several models have been developed to assist in predicting such behavior. Abdel-Ghaffar and coworkers obtained response data for the San Pedro-Terminal Island bridge and more recently conducted ambient vibration studies of the Golden Gate Bridge [1,2]. Accelerometers were used to obtain all dynamic response measurements and subsequent Fourier analysis allowed estimation of natural frequencies and mode shapes. Bampton and others reported on wind and motion data on the Pasco-Kennewick Bridge where anemometers identified potentially dangerous wind speeds and then accelerometers provided response data along the span [3]. Similar data was obtained on the Sitka bridge in Alaska. A sophisticated instrumentation and analysis package has been installed on the Luling Bridge in Louisiana and has been reported by Bourgeois [4]. All of the previous studies of dynamic response measurements were conducted on bridges where construction was complete.

OBJECTIVES

The overall objective of this study is to obtain dynamic experimental response data from a cable stayed bridge under construction in order to provide an enhanced understanding of the dynamic response characteristics of the overall structural assembly and to better define the response of the individual constitutive elements making up the bridge, namely the deck, piers, pylon and stays. The development of the dynamic properties of the various elements and the total structure during the different stages of posttensioning and stay installation and tensioning will provide valuable insight into the basic characteristics of the bridge and into any potential problems that might arise during construction.

Several distinct tasks have been undertaken, which can be treated separately. These include

1. Field measurements ofthe responses of:
   - foundation piers and pylons.
   - superstructure
   - stay cables
2. Estimation of important natural frequencies and mode shapes from the field data.
3. Comparison of the field study results with available analytical models.
4. Evaluation of critical dynamic responses during the construction period.

DESCRIPTION OF BRIDGE

The bridge which is the subject of this study is the I-295 bridge over the James River 10 miles southeast of Richmond, Virginia. An elevation sketch of the central cable-stayed portion of the bridge is shown in Fig. 1. The river crossing will be the 630-ft. middle span of a seven span continuous structure which consists of twin precast, segmentally post- tensioned concrete box girders. The five middle spans will be supported by 52 stay cables arranged in a single plane harp configuration from 300-ft. tall pylons located at either end of the middle span and attached to delta frame assemblies located between the twin boxes, as shown in Fig. 2. The middle span will be constructed by cantilevering the segments from the piers located adjacent to the pylons. The main span structure is essentially symmetrical both longitudinally and transversely about the centerline.

BASIC INSTRUMENTATION PLAN

All dynamic response data will be collected using accelerometers as the transducers and collecting and processing the data through the use of a dynamic data acquisition system. Data analysis will be accomplished through the use of specifically designed software on a main frame computer and a signal analyzer for use both in the field and later in the lab.

The data acquisition system is a Megadac System 2210C which is a portable, high speed data acquisition, signal conditioning and data recording system. This system captures and records dynamic analog data at any continuous rate up to 20,000 samples per second with 16-bit resolution. The 2210C accepts up to 128 channels of differential inputs from a wide variety of transducers which are scanned in any order. In this particular study, only 16 channels of input will be gathered at one time. Data is recorded in real time, digitized upon input, and stored on magnetic tape cartridges. Signal conditioning and filtering is provided as part of the Megadac system, eliminating the need for separate equipment for this purpose. The data will subsequently be read into an AT-class personal computer for analysis using a Megadac 2000CR tape reader. Software provided as part of the overall system can be used to carry out spectral analysis on the personal computer or a communications interface can be utilized to analyze the data on the University of Virginia CDC Cyber 855 mainframe computer which supports a full complement of signal processing software.

An alternative means of data analysis used in this study will be a model CF-350 Ono Sokki signal analyzer, a dedicated two channel FFT analyzer. The use of a dedicated signal analyzer will greatly facilitate the estimation of mode shapes, since the peak spectral responses of the active modes can be easily obtained. The signal analyzer also is capable of distinguishing between closely spaced modes, using its zoom features and obtaining displacement response spectrum estimates using its integration features.

Model SA-102 servo accelerometers manufactured by Terra Technology will be used. The accelerometers have a frequency response of 0 Hz to 100 Hz., and were chosen for their ruggedness and low hysteresis. Gill U-V-W anemometers will be used to provide three dimensional wind data during the ambient vibration studies.

For dynamic studies of individual components such as piers and pylon, and for the partially completed main structure as well, and accelerations due to either forced or ambient excitation will be recorded. Natural frequencies and mode shapes will be determined by comparison of transducer responses with transducers placed at reference locations. Preliminary analytical studies on finite element models of the bridge and its components have been used to estimate the location of nodes in the first several modes of vibration to insure that the reference measurements avoid these locations. Where it is considered important, arrays of transducers will be placed to identify torsional, flexural, and axial responses of the relevent stations.

The structure under investigation is essentially a doubly symmetric structure as seen from Fig. 1 and 2. Hence, considerable economy in the instrumentation will be obtained by instrumenting only one side of the box girder over one half of the structure. The exception to this strategy consists of placing vertical accelerometers as far away from each other on the cross-section as possible, in order to obtain the best possible estimate of the torsional accelerations.

Prior to closure of the main cantilever span, instrumentation will be placed along the cantilever portion to clearly identify the response characteristics of this part of the structure. Results from the analytical studies indicate that the natural frequencies for this discontinuous part of the structure will be significantly lower than that for the span after closure as would be expected. Similar studies will be carried out on the pier and pylon prior to their being connected to other components of the bridge structure. Finally, studies will be conducted after completion of the structure, to complete the data on the evolution of the bridge properties.

METHODS OF EXCITATION

Most of the dynamic data will be obtained using ambient excitation of wind and equipment, or, after bridge completion, vehicular traffic. This form of excitation should be sufficient to provide information on modes and frequencies as well as some indication of damping. More specific dynamic characteristics can be measured from excitations that are either harmonic or from an impulsive type load, and these types of excitations are more difficult to apply at the necessary scale.

A number of schemes for providing forced input to the bridge are being considered. One of the most obvious techniques would be to use a counter-rotating mass shaker to provide harmonic forcing to various bridge components. Unfortunately, the size and weight of this type of shaker necessary to excite the low natural frequencies expected make their use impractical. These limitations are primarily of concern in the dynamic analysis of components such as the piers and pylons, where access is difficult. After completion of the bridge, it is fully expected that some form of harmonic exciter will be used for subsequent data collection.

Alternative forms of excitation are also being considered. One that shows some promise of being both practical and functional is the use of a crane-attached cable for a snap-back type of loading. Another involves the use of a variable load pile driver mechanism to provide for periodic impulsive loads. At the present time, both of these alternatives are still under development, and use of either alternative will require contractor approval. While the plans to excite the bridge are under development, dynamic response mesurements will depend on ambient excitation only.

DATA ANALYSIS

Data collected during this investigation will include the simultaneous measurement of vertical, lateral and longitudinal response of the cable-supported deck as well as the lateral and longitudinal motions of the pier and pylon both before and after the bridge elements are connected. In addition, simultaneous wind data will also be measured and recorded from the anemometers.

The initial analysis of data will consist purely of extracting natural frequencies and mode shapes. The frequency information will be automatically provided through the spectrum analysis performed by either the personal computer or by the signal analyzer. Amplitudes provided in the frequency plot, which are a measure of the strength of a particular mode, will be compared with data measured at a fixed reference station and these relative amplitudes will define the mode shape at that frequency. Care must be taken in the location of transducers, and especially the location of the reierence transducers, to ensure that good modal identification is achieved.

Since this investigation will have available only sixteen transducers initially, it will be necessary to take successive readings at the different locations on the bridge, leaving only the reference station transducers in a fixed location, a technique that is widely used in ambient vibration studies. Also, the anemometer data will require three channels for each station and this will reduce the simultaneous accelerometer channels accordingly.

The transducer excitation will be provided by the data acquisition system and the response measured and recorded on the tape cartridge unit which has a capacity of 64 Mbytes of memory. This data will, in turn, be read in to the hard disk unit of the PC where appropriate analysis functions will be performed using the OPUS software provided by Optim, the manufacturers of the Megadac unit. Alternatively, the data from the hard disk, in digitized form, may be reconverted to an analog signal and fed directly into the signal analyzer for direct processing. Output from either the software analysis or from the signal analyzer can be easily presented in either printed format or used as input to a plotter for easier representation.

PROJECT STATUS AND SUMMARY

At this time, the data acquisition system, the signal analyzer, the AT-class personal computer, the anemometers and the accelerometers have been received. The preliminary finite element modeling has been completed, and reasonable first estimates of the transducer stations have been determined. Reference stations have been selected. Construction of convenient mounting devices for the transducers has been completed. Time constraints have limited the amount of data acquisition that could be accomplished since much of the equipment did not arrive until late August of this year. It is anticipated that a significant data collection will commence in late September, 1988.

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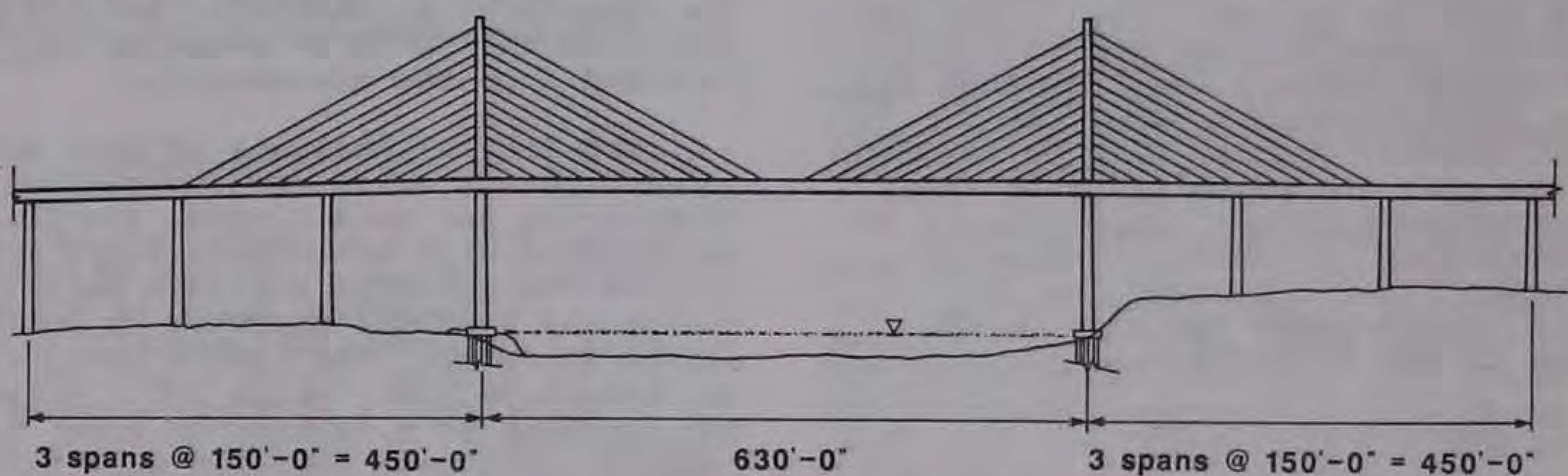

Figure 1. Elevation of the I-295 James River Bridge

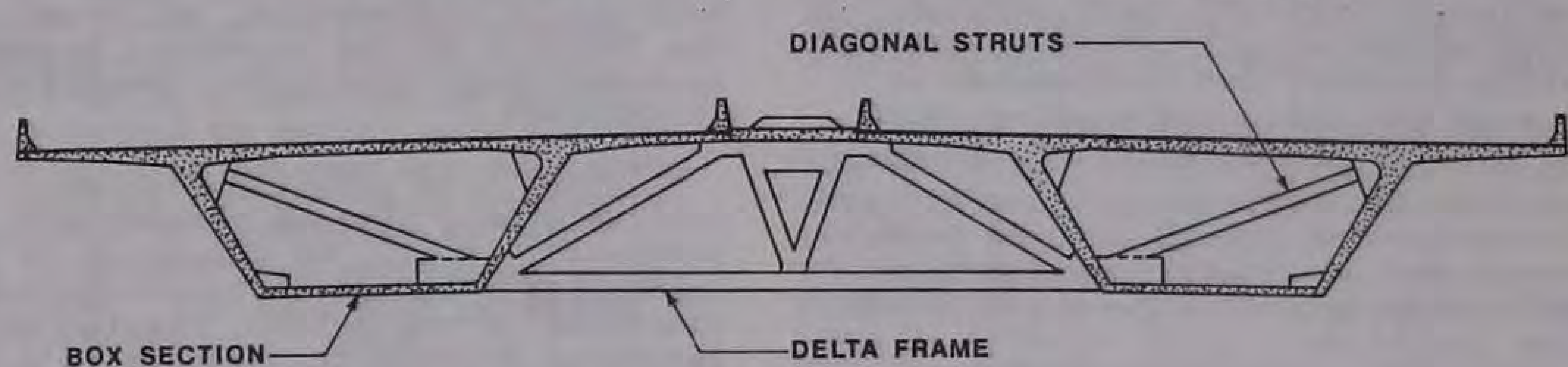

Figure 2. Transverse Section at Typical Stay Anchorage

IMPACT TESTING MODAL IDENTIFICATION OF A CABLE-STAYED PEDESTRIAN BRIDGE

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SYNOPSIS

Vertical and torsional modes of a cable-stayed pedestrian bridge are measured by the impact-hammer technique. Natural frequencies, damping ratios, and mode shapes are identified using the polyreference method. The measured mode shapes are compared with a finite element dynamic analysis of the bridge. Differences between the experimental and analytical modes are probably related to the uncertainty of the mass distribution and stiffness of the bridge's laminated wood deck. The impact-hammer technique is shown to be an efficient method of modal identification of a cable-stayed pedestrian bridge that requires only modest easily transported instrumentation and a few hours of on-site testing. It should be applicable to real full scale bridges.

INTRODUCTION

A knowledge of the modal properties of a bridge is a means to assess its structural integrity. It is necessary to predict the bridges response to earthquake, wind, and traffic loading, and to evaluate fatigue. Experimental modal analysis is also used to verify assumptions in, and results from, analytical models. This paper presents the comparison of modal properties of a cable-stayed pedestrian bridge which were measured by using the impact hammer technique and calculated by a finite element model.

Impact hammer modal testing has gained widespread popularity in mechanical and aerospace engineering applications [1,2]. The basic technique involves striking an elastic structure at a point with a hammer instrumented to allow measuring the force of impact, and then to observe the response of the structure at one or several points by accelerometers or other suitable transducers. The sharply peaked impulse given the structure usually has sufficiently broad frequency content to excite many, if not all, of the modes of the structure. The processing of digitized records of the impact response by any of several available algorithms gives frequency response functions, impulse response functions, natural frequencies of vibration and damping ratios. Analyzing the response of the structure by striking it at several points that span the structure or by instrumenting the structure at several points will yield the mode shapes.

Impact hammer testing has several advantages that make it a competititve alternative to other forms of modal testing. Some of the advantages are: 1) The broadbanded nature of the hammer impulse cuts down on test time by exciting several modes simultaneously; 2) the test equipment is comparatively light and portable and requires a minimum of setup time; 3) data processing is now practical due to the advent of self-contained modal testing hardware and software systems; and 4) the use of an instrumented hammer gives a known input to the structural system - a quantity that is lacking in ambient vibration studies. A disadvantage of using impact testing on civil engineering structures is that the energy required to excite the structure demands relatively large hammers that are designed not to cause local damage to the structure. However, the energy must still excite a broad frequency range and be much greater than any ambient vibration.

BRIDGE

The bridge under study is a cable-stayed bicycle-pedestrian bridge crossing over Vermont Highway 127 in Burlington, Vermont. A layout of the bridge in a span view is shown in Figure 1, and a cross-sectional view is shown in Figure 2.

The total bridge span is 180'. Two fabricated box steel towers straddle the main deck and slope out towards the center of the span. The towers are hinged at their bottoms and each is supported by two cables (one on each side) that slope back and are anchored to the ground. Each tower has four radial cables (two on each side) that support the main spanning girders. All cables are bridge strand cables with a diameter of 1 5/8".

The main girders are 36" deep I-shaped fabricated steel that arch over the span. The center of the arch is 3' above the horizontal. Between the two main girders are W16x77 I-beams spaced at approximately 18' intervals. These smaller I-beams support a 11'2" wide laminated wood deck.

THE TESTING TECHNIQUE

The on-site instrumentation consisted of a

PCB-08B05 instrumented sledge hammer, 4 PCB-393C prezoelectric accelerometers, and a TEAC-R-71 FM tape recorder. The testing technique involved placing the accelerometers at four locations shown in Figure 3 and impacting the bridge at the 22 points of intersection between the main spanning girders and the cross I-beams. All impacts and acceleration measurements were in the vertical direction. The force of impact and the resulting deck acceleration time histories were recorded in an FM-analog format. Six separate impact and response tests were performed at each point so that at least five could be averaged to improve the coherence of the input and output signals. In a series of preliminary tests, it was found that ambient vibrations due to traffic underneath and wind loads, considerably reduced the accuracy and consistency of the results. Therefore, it was decided to test on a calm evening when the traffic levels were reduced. The total on-site test time was 4 1/2 hours. However, additional time was required to process the data off line.

In addition, since it was felt that the cable pretensions may have a significant effect on the dynamic properties of the bridge by the action of geometric stiffening, the cable tensions were estimated by measuring the fundamental frequency of vibration of each cable [3,4]. These are shown along with their corresponding cable tensions in Table 1. Also, as an estimate of the bridge's lateral stiffness, the first lateral mode due to ambient vibration was measured as 2.8 Hz.

DATA PROCESSING TECHNIQUE

A GENRAD 2515 four channel spectrum analyzer, with SDRC Modal Plus software was used to perform the data reduction. The analog recorded time histories were played into the spectrum analyzer. The spectrum analyzer then in sequence: a) digitized the time histories; b.) exponentially windowed the time series; c.) calculated the discrete Fourier transform via the FFT; d.) calculated the FRF using the $H_1(f)$ estimator, eq. 1; e.) averaged the FRF's of up to five impacts; and f.) calculated the coherence of the averaged FRF. The $H_1(f)$ FRF estimation is defined as:

$$H_1(f) = \frac{Sfx\ (f)}{Sff\ (f)} \qquad (1)$$

where $H_1(f)$ = estimated frequency response function

$Sfx\ (f)$ = cross-spectral density of force & response

$Sff\ (f)$ = auto-spectral density of force

The $H_1(f)$ estimator is one of several available. It is distinguished by having a bias toward antiresonance and over estimating damping. The estimated FRF's were then inverse Fourier transformed into the time domain to give estimates of the impulse response functions of the system. The polyreference technique [5] then approximates the impulse response functions by least-squares fitting of complex exponentials.

$$I(f) = \sum_{m=1}^{2i} Am\, e^{\lambda_m t} \qquad (2)$$

where

$I(f)$ = approximate impulse response function.

The frequencies of vibration, damping ratios and mode shapes are then identified directly from the coefficients of the impulse response functions series. The polyreference technique will occasionally produce spurious modes characterized by overly large damping ratios. These spurious modes are readily identified and rejected.

An evaluation of the quality of the mode shapes is performed by calculating the modal assurance criterian (MAC). The MAC is a matrix of the normalized data products of the identified modes. Mode shapes that are lineraly independent and approximately orthogonal ideally will produce a matrix that has ones on the diagonals and zeroes on the off-diagonals.

TEST RESULTS

A typical FRF appears in Figure 4. Since accelerometer locations #1 and #2 appeared to have the best FRF's, only they were used to extract modal properties. Sixteen modes were discovered in the frequency range of 0-16 Hz. The frequencies and damping ratios are tabulated in Table 2. The mode shapes are shown in Figure 5. The MAC matrix for the first nine modes is shown in Table 3. The MAC matrix shows the correlation of the asymmetric modes with the second bending mode.

FINITE ELEMENT MODEL

The bridge structure was analyzed using Algor's Super SAP on a personal computer. The finite element model is shown in Figure 3. The FEM model consists of 133 beams and 107 nodes with 576 D.O.F.

The wooden deck was considered nonstructural and its mass was lumped at the interconnections of the main girders and I-beams. The cable tensions from Table 1 were used to include geometric stiffening in the cables. 16 modes were found below 16 Hz. The frequencies are in Table 2. The mode shapes are shown in Figure 5.

COMPARISON

Table 2 and Figure 5 show the modal analysis to have natural frequencies that are a little higher in bending and much higher in torsion.

The modal analysis found a number of asymmetric modes that are not in the FE analysis. The experimental modes did not contain the bending modes associated with the tower modes nor the number of third torsional modes.

The differences in the bending and torsional natural frequencies may be due to the wooden deck stiffness not being included in the FE model. This stiffness would raise the torsional more than the vertical bending mode.

From Table 1, the cable tensions are seen to be asymmetric. This could mean that the deck mass is also asymmetric. The FE model assumes the mass is uniformly distributed, and does not have the asymmetric modes.

CONCLUSIONS

Although ambient vibrations can be a problem, full scale impact hammer modal testing is an efficient and relatively easy method of identifying modal properties of a cable-stayed pedestrian bridge. The polyreference technique was able to consistly extract the modal properties from the FRFs from more than one accelerometer location at one time.

The modal test and FE model compare reasonably well. However, the modal analysis has higher natural frequencies and asymmetric modes not found in the FE analysis. These differences are the result of uncertainty in modeling both the mass and stiffness of the wooden deck.

Future research will attempt to include the deck properties in the FE model to see if an improved correlation of the experimental and analytical modes results.

ACKNOWLEDGMENT

The authors gratefully acknowledge the loan of the GENRAD specrtrum analyzer and SDRC Model Plus software from the General Electric Co., plant in Burlington, Vermont.

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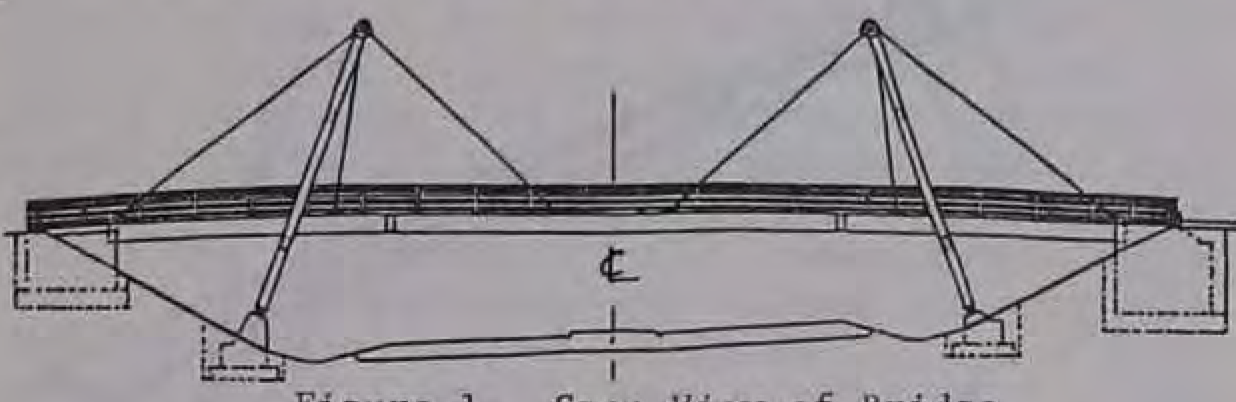


Figure 1. Span View of Bridge

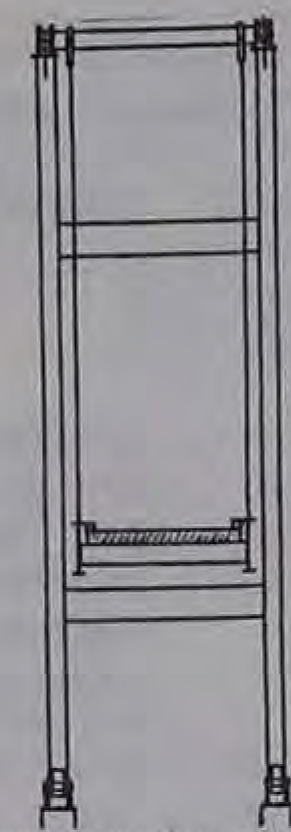


Figure 2. Cross-sectional View of Bridge

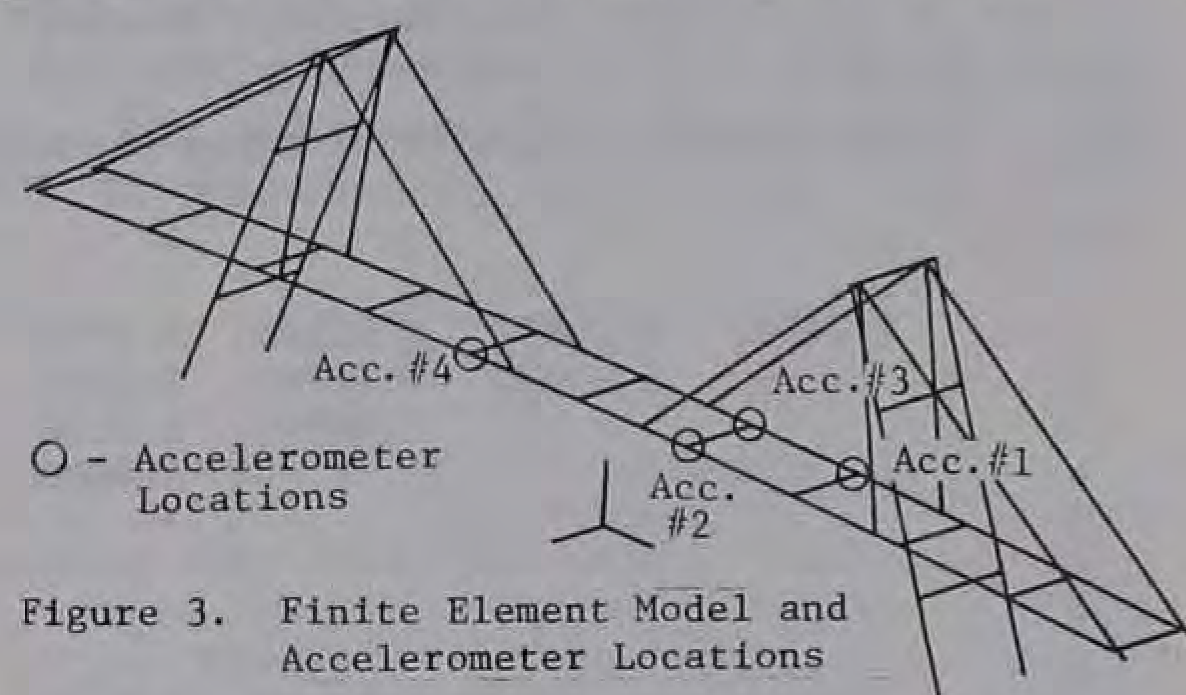


Figure 3. Finite Element Model and Accelerometer Locations

Table 1 Measured Cable Frequencies and Calculated Tensions

Cable Number	Frequency (Hz)	Length (inches)	Tension (lb)
#1	5.12	629.84	50,340.
#2	4.52	629.84	39,190.
#3	4.64	629.84	48,000.
#4	5.00	629.84	41,310.
#5	7.56	345.03	32,390.
#6	7.88	345.03	35,250.
#7	8.00	345.03	36,350.
#8	8.04	345.03	36,730.
#9	3.52	519.22	15,940.
#10	3.32	519.22	14,140.
#11	3.40	519.22	14,850.
#12	3.12	519.22	12,460.

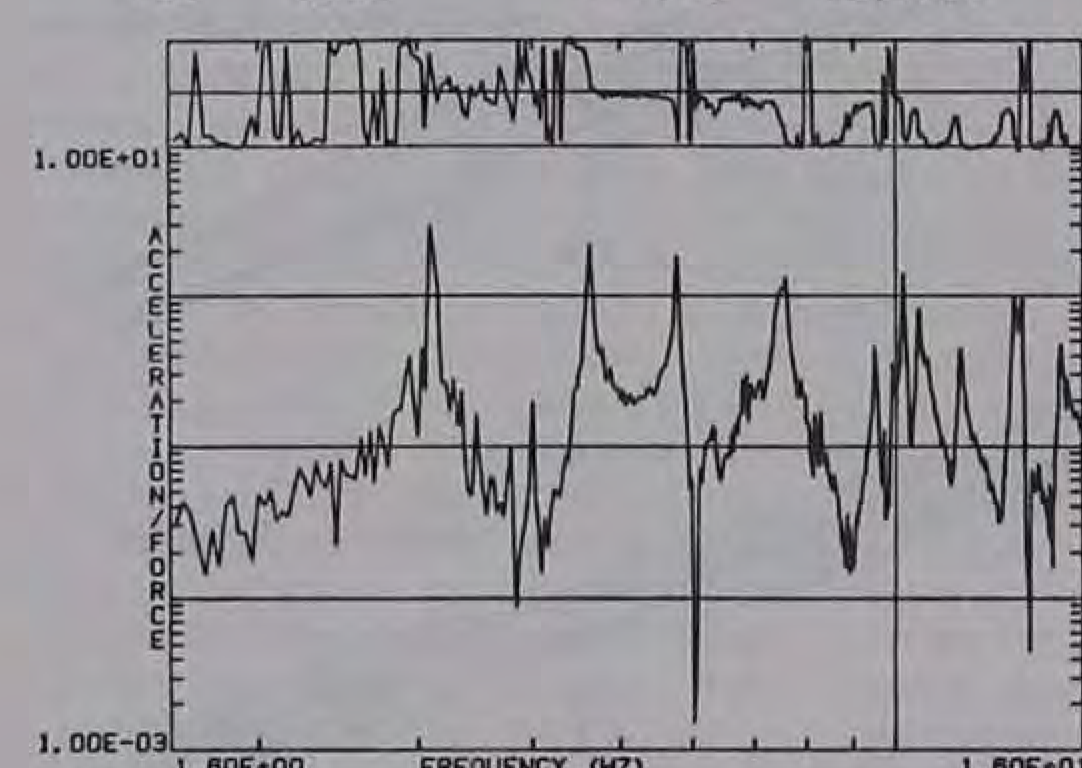


Figure 4. Typical FRF with Phase on Top and Magnitude on Bottom

Table 2 Natural Frequencies of Modal and Finite Element Anaysis

Modal Freq. (Hz)	Damping Ratio	Finite Element Freq. (Hz)	Mode Shape
3.10	0.006	3.05	1st Bending
		3.13	1st Bending
4.62	0.006	3.96	1st Torsion
5.78	0.003	6.13	2nd Bending
6.83	0.012		Asymmetric 2nd Bending
7.07	0.008		Asymmetric 2nd Bending
7.50	0.007		Asymmetric 2nd Bending
7.58	0.004	6.48	2nd Torsion
		8.96	Tower Mode w 2nd Bending
		9.06	Tower Mode w 1st Bending
		9.20	3rd Torsion w Bending
9.48	0.006	9.29	3rd Bending

Table 2 Natural Frequencies of Modal and Finite Element Anaysis

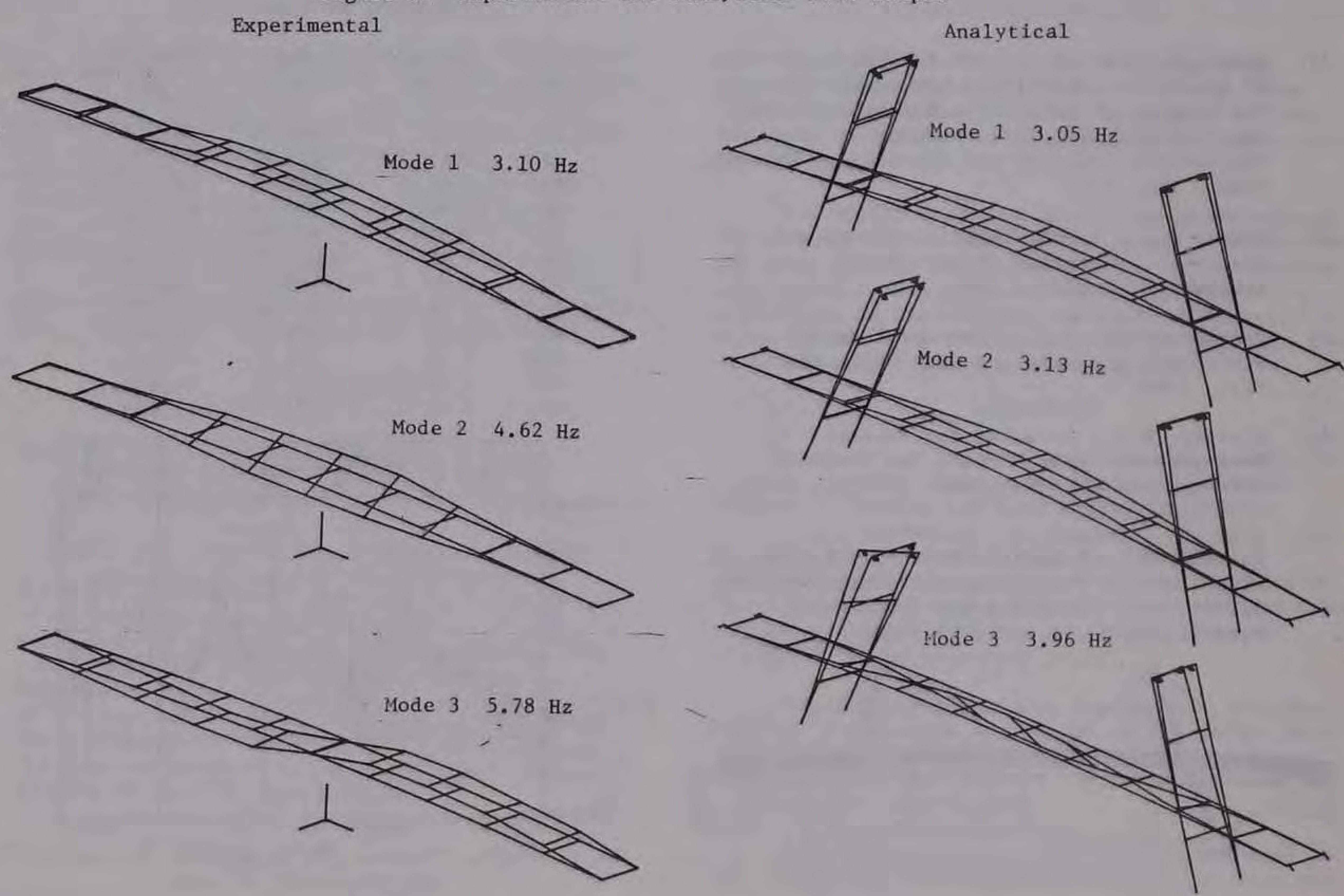
cont'd

Modal Freq. (Hz)	Damping Ratio	Finite Element Freq. (Hz)	Mode Shape
		9.34	3rd Torsion w Bending
		9.43	3rd Torsion w Bending
9.91	0.004		Asymmetric 2nd Torsion
10.21	0.006		Asymmetric 2nd Torsion
10.60	0.008	9.67	4th Bending
11.72	0.005		3rd Torsion
13.46	0.006	9.74	4th Torsion
		10.56	Asymmetric 4th Torsion
13.56	0.018		Asymmetric 4th Bending
13.75	0.003	13.67	5th Bending
		13.98	5th Torsion
15.13	0.007		Out of Phase 5th Bending

Table 3 MAC Matrix of First Nine Modes (Symmetric)

Mode#	1	2	3	4	5	6	7	8	9
1	1.00000	0.00007	0.00268	0.59487	0.22407	0.00335	0.00794	0.00024	0.00301
2		1.00000	0.00390	0.00001	0.00459	0.00045	0.00303	0.00048	0.71054
3			1.00000	0.31362	0.76362	0.92265	0.05842	0.00102	0.00060
4				1.00000	0.01504	0.27623	0.01128	0.00104	0.00000
5					1.00000	0.70124	0.06108	0.01805	0.00067
6						1.00000	0.00041	0.00204	0.01437
7							1.00000	0.00020	0.21273
8								1.00000	0.00024
9									1.00000

Figure 5. Experimental and Analytical Mode Shapes



MOVEMENTS AND BRIDGE BEARINGS

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SYNOPSIS

This paper describes the research results for two research projects which are in progress at the University of Washington. These research projects are concerned with understanding and predicting movements in bridges and selecting bearings which can support the required loads and accommodate these movements. The paper briefly summarizes the work which has been completed to date, describes the work in progress, and notes some of the future concerns and unresolved issues.

Introduction

Bridges move or deform due to creep and shrinkage of concrete, thermal expansion and contraction, and the braking and acceleration forces of traffic. These movements must be accommodated by bearings or expansion joints or very large internal forces may develop. These internal forces may damage the structure or cause problems which reduce the serviceable life of the structure. It is therefore important that the structural engineer accurately estimate these movements and understand the behavior and movement capability of bridge bearings if problems are to be avoided. The paper will briefly describe the progress on two research projects at the University of Washington. These projects are directed toward better understanding the movements in bridges and the behavior of bearings which are needed to accommodate these movements.

The bearing research is funded by the National Cooperative Highway Research Program (NCHRP Project 10-20 with J. F. Stanton as Co PI). This research was started in 1981 and is now in the third phase of study. The first phase of the research was directed toward the development of improved design specifications for plain and reinforced elastomeric bridge bearings. These bearings are very economical and require minimal maintenance, but the usage of these bearings had increased dramatically since their initial conception while the design methods did not keep up to date prior to the start of this research. As a result, a comprehensive state of the art review was performed in Phase I, and a proposed specification was developed This proposed specification was the basis of major changes made to the AASHTO Specification in 1985. The second phase of the research was directed toward improved understanding of the behavior and modes of failure of reinforced elastomeric bearings and further improvements to the design specification. This work was completed in 1987. Numerous experimental studies were performed on reinforced elastomeric bearings and further changes to the AASHTO Specification were recommended. These changes have not yet been adopted by AASHTO, but they are presently under consideration. The third phase of this research is in progress, and it is directed toward acceptance testing criteria of elastomeric bearings and a review of other bearing systems such as pot bearings and mechanical bearings such as spherical bearings and cylindrical bearings which use PTFE sliding surfaces.

The second research project was recently funded by the National Science Foundation (CES 8714894), and is still in the early stages of development. It is directed toward better understanding thermal movements in bridges and improved estimates of these movements. It is essential that these movements be accurately predicted. If the movements are overestimated the structure will require excessive movement capacity in the bearings and expansion joints. This increases the cost of the bridge and it may increase the potential for deterioration, since deterioration frequently occurs at these locations. If the movements are underestimated, large internal forces may develop, and there may be increased repair costs or a reduction in the service life. It should be noted that the initial cost of bearings and expansion joints may be relatively small when com-

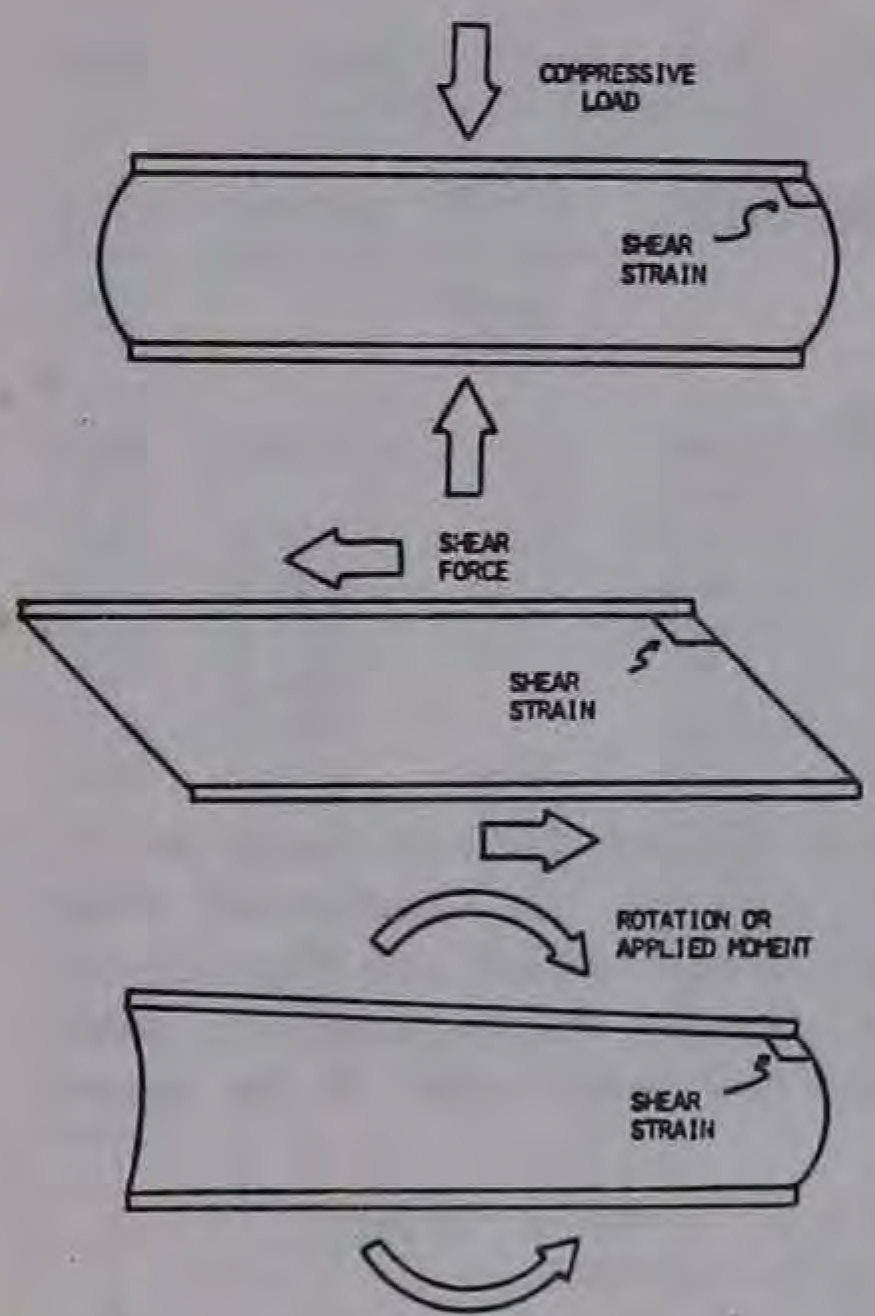

Figure 1.

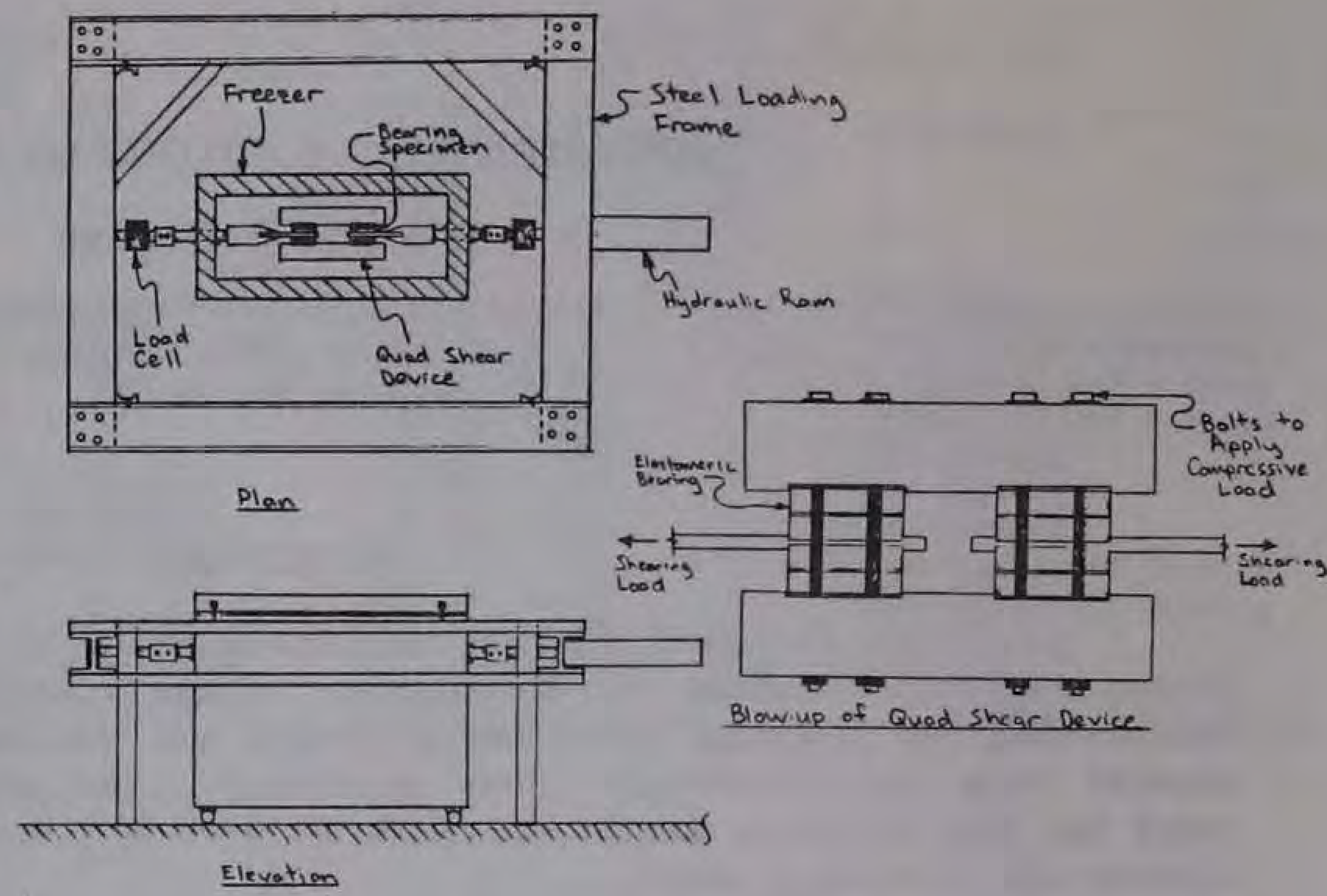

Figure 2.

pared to the total cost of the bridge, but they may contribute a major element of cost when the bridge is damaged or requires repair and rehabilitation.

Research on Bearings

Phase I. Phase I of NCHRP Project 10-20 was completed in 1982, and several publications summarize [1,2,3] the results of this research. This research showed that elastomers are complex visco-elastic materials. The stiffness of the elastomer depends upon the elastomer compound, the rate of loading, the duration of loading and temperature. Elastomeric bearings commonly utilize steel or fiberglass reinforcement with the elastomer, and the combined effects of the nonlinear material behavior with the geometry of the bearing results in bearing behavior which is difficult to mathematically predict. The bearing may be very stiff in compression because of the lateral restraint provided by the reinforcement as shown in Fig. 1a. The stiffness depends upon the elastomer compound, and the geometry (or shape factor) of the bearing. They are relatively flexible in shear (see Fig 1b) and this permits shortening and elongation of the bridge with minimal resistance Rotations of the bridge girders induce rotations in the bearings, and this effect is depicted in Fig. 1c. All three types of deformation induce shear strains in the elastomer as shown in the figure, and these shear strains must be an important consideration in the design. Unreinforced elastomeric bearings are commonly used in the United States, and their behavior is also similar to that depicted in Fig. 1. However, the lateral restraint for bulging is provided by friction between the elastomer and the load surface rather than the reinforcement. The research showed that friction is a much less reliable method of restraint and slippage frequently occurs. Slip results in increased strains in the elastomer, and as a result it was noted that unreinforced bearings should be designed to lower stress levels than comparable reinforced bearings.

Numerous modes of failure were noted for elastomeric bearings. These include excessive strain of the elastomer, fatigue of the elastomer, debonding or separation of the elastomer and the reinforcement, tensile failure of the reinforcement, buckling or stability failure of the bearing, and slip of the bearing from its desired position. In addition, it was noted that serious structural problems may occur due the increased bearing stiffness caused by low temperatures. The stiffness of the elastomer may be much larger at low temperatures, and this may cause large internal forces in the bridge piers and superstructure because the bridge tends to shorten at these reduced temperatures.

A simple proposed design specification was developed from this research and was adopted by AASHTO in the 1985. This proposal was a strain based specification, and it considered all of the critical modes of failure and true bearing behavior. It resulted in more liberal rules for using reinforced bearings and generally greater restrictions upon unreinforced bearing pads.

Phase II. The second phase of the research was started in 1983 and completed in 1986, and so the results [4,5] are not yet widely published. This phase of the research examined the failure modes of elastomeric bearings in much greater detail. Experiments were performed on reinforced elastomeric bearings to examine fatigue of the elastomer, tensile failure of the reinforcement, debonding of the elastomer and the reinforcement, buckling of the bearing and the stiffness and deformation capacity under different loadings. In addition, theoretical models for these phenomenon were examined. These improved models for bearing failure were then incorporated into a proposed specification. This proposal results in more liberal load and deformation limits for some reinforced bearings, but it also requires greater quality control during the manufacture and more refined design calculations.

The effect of low temperatures was also analytically considered during the second phase of the research. The research showed that elastomers stiffen with continued exposure to low temperature through crystallization and they may also stiffen instantaneously at very low temperatures at the second order transition. The stiffening depends strongly upon the elastomer compound. Very limited experimental data was available for this low temperature behavior, but the existing information indicated that this stiffening could have a significant impact on the forces transmitted to the bridge piers and superstructure. Therefore, it was clear that an reliable testing and acceptance procedure was needed to assure that the elastomer compound is appropriate for the environment of the bridge.

Phase III. The third phase of this research was started in 1986 and should be completed in 1989. It includes extensive low temperature tests of elastomeric bridge bearings. Different elastomer compounds are exposed to different temperatures for different periods of time and the shear stiffness is measured periodically with the test apparatus depicted in Fig. 2. This measurement is much more direct than earlier research which focused on elastomer hardness and compression set. The objectives of these tests are the development of a rational acceptance testing criteria for the proposed specification developed in Phase II and an improved understanding of low temperature stiffness. Low temperature acceptance testing should obviously be different for different parts of the US because of the wide variation in temperatures and climates noted. Therefore, temperature data for 6 different locations are being examined to develop a rational method for accomplishing this variation. While it is not possible to draw final conclusions from the test data completed to date, it is clear that some important results are evolving. First, the results show that some elastomer compounds stiffen significantly while others are much less sensitive to low temperatures. Second, the results appear to correlate fairly well with the earlier research, but it also indicates that some of the results obtained in earlier studies are not truly relevant of bridge bearing behavior since they focused on indirect measurements such as elastomer hardness.

A second major objective of the third phase of this research program is the initiation of a study into the behavior of other bearing systems. Elastomeric bearings are the most widely used bearing type in the United States today, but they are used mainly on shorter span bridges with moderately light gravity loads. Long span bridges or very heavy bridges typically employ others bearings such as pot bearings, spherical bearings, cylindrical bearings, or sliding bearings. These other bearings are used less frequently, but they tend to be more expensive. Further, there have been some serious problems noted with their performance. A comprehensive state of the art review of the performance of some of these other bearing systems was recently completed. The review focuses on pot bearings and mechanical bearings which utilize PTFE sliding surfaces. The results are not yet available for publication, but a few major items are worth noting. A number of documented problems with pot bearings have occurred in the US in recent years. These problems include leakage of the elastomer past the sealing rings, abrasion of the elastomer disk, and bottoming out or binding of the metal parts under rotation or deformation. Many of these problems appear to be caused by possible quality control problems in the manufacture of pot bearings, but there do not appear to be well defined methods for assuring the necessary quality with present US practice. There are presently no reliable guidelines for predicting the rotational stiffness of pot bearings. PTFE sliding surfaces are used in conjunction with stainless steel or anodized aluminium mating surfaces on pot bearings, flat sliding surfaces, and spherical or cylindrical bearings. These bearings are commonly designed with friction factors such as those specified in the existing AASHTO Specification. However, extensive research on these sliding surfaces has been performed in Europe and limited research has been performed in this country. This research indicates that the friction factor decreases with increasing load and is very sensitive to the lubrication between the PTFE and the mating surface. The lubrication tends to migrate with time and movement and the friction factor and wear of the PTFE increases with this migration. As a result, European practice has developed dimpled PTFE to hold the lubricant in place, but the dimpled material is seldom used in this country. The existing AASHTO Specification does not address many relevant questions such as the effect of lubrication on the friction factor, the difference between dimpled and flat PTFE, and wear or erosion of the material, and there are other limitations with the existing understanding of the behavior of sliding surfaces.

Future Research. A number of research requirements can be ascertained. It is not possible to list all of these requirements, but a few are worth noting. First, better guidelines are needed to assure reliable manufacture and behavior of pot bearings. Second, research is need to resolve the differences between the true behavior of existing PTFE sliding surfaces in this country, the recommendations of the AASHTO Specification, and the research performed in other countries. Some bearing types have proven to be relatively trouble free while others have caused serious problems, and so research is needed to help the bridge design engineer to decide when it is appropriate to select one bearing system over another.

Thermal Movements

A second research program started in December 1987 and is in the early stages of development. This research program is directed toward better understanding thermal movements in bridges and developing better methods for estimating the movements during the bridge design. It will attempt to resolve questions such as -

1. Why can some bridges be designed without expansion joints for thousands of feet with no apparent problems while other bridges may have problems with very close expansion joint and bearing spacing?

2. When are simple methods for predicting thermal movements adequate and when are more refined and sophisticated methods required?

3. What are the temperatures and temperature ranges that should be considered in the prediction of thermal movements?

4. Is it necessary or desirable to consider the restraint of the bearings, the stiffness and temperature distribution in piers and abutments, or shading or local exposure conditions caused by typography or environmental conditions?

It should be noted that it is not enough to make a conservative estimate of bridge movements, since less deterioration, longer service life and better bridge performance may sometimes result if fewer expansion joints and bridge bearings are employed.

The research will include a survey of state DOT's, bridge engineers, and governmental agencies. The survey will attempt to define how bridges are designed for thermal movements; what research has been performed; and what problems have been noted. The survey will also help to isolate specific structures which will be analyzed in greater detail. Different analytical models of heat flow and thermal deformation will be applied to these bridges, and the computed results will be correlated to measured results or observed behavior.

It is too early to draw significant conclusions from the research. The survey is well underway however, and a few preliminary observations can be made. First, it appears that even the most complex and sophisticated bridges are designed with the most simple models for predicting thermal movements. It appears only uniform thermal expansion of the bridge deck is considered. Thermal deformation of piers and abutments are not included. Second, it appears that one of the major problems noted with thermal movements in bridges is not that the total movements are larger than expected but that the distribution of movements is very different than expected. One engineer notes that one bridge, which was anchored near the center and designed for relatively uniform expansion at each end, developed most of its expansion at one end, and this has a potential for serious problems at both the bearings and the expansion joints. Third, engineers who note movement problems frequently attribute them to other causes. For example, several engineers noted that the anchorage of some bridges pulled out. They commonly attribute this to inadequate cover or embedment, and they don't appear to ask what caused the large force or what was the source of the movement. Fourth, some engineers noted specific deformation problems such as the uplift of curved girders from their bearing points. Finally, the engineer frequently has difficulty isolating the source of the movement when large movements were noted. Creep and shrinkage of concrete, traffic loading or bridge geometry (for example a bridge on a slope) may contribute to the movements, and the engineer appears to unable to separate these effects. The survey appears to be defining some for the future research effort, however the author welcomes any suggestions or recommendations for this research since it is in the early stages. He would particularly welcome any information concerning bridges with good experimental data which could be used in the future analytical studies.

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TIMBER BRIDGE RESEARCH BY THE UNIVERSITY OF WISCONSIN AND USDA FOREST SERVICE

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SYNOPSIS

The University of Wisconsin-Madison and the USDA Forest Service are cooperating in a comprehensive research program to define the behavior of stress-laminated timber bridges and to develop design procedures for this innovative system. Completed research and work underway are described.

BACKGROUND

The state of the art in timber bridge design has advanced considerably in the past forty years. Most notably, traditional nail-laminated transverse decks have been superseded by glued-laminated (glulam) panel decks. Rigid laminated deck systems offer many structural advantages over the nail-laminating system.

Previous research by the USDA Forest Service (FS) demonstrated the structural advantages of glulam decks over nail-laminated and presented analytical methods for predicting the performance of glulam panel decks and steel dowel connectors [3]. Simplified methods [4] were adopted into the design specifications of the American Association of State Highway and Transportation Officials (AASHTO). Erection procedures were suggested that facilitate the construction of glulam decks with dowel connectors [12].

During the summer of 1983, FS researchers investigated the long-term performance of timber bridges by inspecting eighteen bridges across the northern United States [2]. A major feature of this study was the acquisition of extensive moisture content data. The researchers found that glulam decks with wearing surfaces protected the substructures from water penetration. Based on this finding, the American Institute of Timber Construction (AITC) allows dry-use stresses for designing the stringers and other parts of the bridge substructure, but requires wet-use stresses for the decks themselves. The study also found that glulam panel decks are performing very well structurally after up to 20 years of service.

Other FS research has addressed the performance of Alaskan native-timber bridges [5,6,10,13] and the use of an innovative laminated veneer lumber (LVL) product, called press-lam, in bridge construction [1].

The Ministry of Transportation and Communication of Ontario, Canada, has developed a new concept, stress-laminating, for restoring longitudinal nail-laminated bridges [11]. In this system, steel stressing rods are installed into the bridge to compress the deck transversely and thereby provide continuity among the individual laminations. Although this system was conceived as a method for rehabilitating bridges, it has also been applied to new construction. It has been used successfully in at least nine rehabilitated bridges and four new bridges in Ontario. A recent variation of the stress-laminating system is the parallel-chord bridge, intended for spans that are too long for the solid-deck type of bridge. This system also uses stress laminating to ensure transverse continuity. The stress-laminating system, in both its variations, offers great promise for providing efficient longitudinal timber decks.

Recent and current FS research in the area of timber bridges is in cooperation with the University of Wisconsin-Madison (UW). The UW and FS are developing methods for designing and constructing stress-laminated bridges with both solid decks and parallel-chord trusses [8,9]. Before design criteria can be proposed for these bridges, further information is needed on their long-term performance. This paper reviews the results of completed research and describes the work underway.

RECENT RESEARCH

Research has been completed recently on three projects: the Wheeler deck, FPL deck, and parallel-chord bridge.

Wheeler Deck

The Wheeler deck project, conducted at the UW, investigated the performance of a bridge contributed by Wheeler Consolidated of St. Louis Park, Minnesota. A test bridge, 9 ft wide by 48 ft long, was constructed from 4- by 16-inch Douglas-fir timbers placed edgewise and stressed together on their wide faces. This study showed that the bridge behaved acceptably when designed in a manner similar to the Ontario methods, but that the maximum span capability for 16-inch decks is probably 35 to 40 ft [7].

FPL Deck

Research at Forest Products Laboratory (FPL) included theoretical analyses and experimental evaluations of a 9-ft wide by 24-ft long stressed deck constructed of 2- by 12-inch sawn lumber. The bridge was evaluated under many configurations and conditions, and it has provided a great deal of information on the behavior, performance, and limitations of the stress-laminating system. A complete report on this work will be published soon.

Parallel-Chord Bridge

A limiting factor in the performance and span capabilities of the stress-laminated deck system is deflection under vehicle loads. Because the stressed deck is basically a slab, its span limitations are approximately the same as that of conventional decks constructed of glued-laminated timber or nail-laminated sawn lumber. To increase the stiffness and span of stress-laminated bridges, a parallel-chord bridge was developed in a FS-UW cooperative project. A 12- by 54-ft bridge was evaluated in the laboratory, and the data were extensively analyzed. Based on this work, an experimental bridge was designed and constructed over the Mormon Creek on the Hiawatha National Forest in September of 1987 (fig. 1). Reports on this research are being prepared.

CURRENT RESEARCH

Research underway is directed at problems that need to be resolved before final design criteria for the stress-laminated deck can be implemented. The research includes a moisture study to determine stress changes resulting from moisture variations in the wood, field monitoring of stress-laminated bridges, investigation of wood crushing problems, and development of the actual AASHTO design standards.

Moisture Study

The stress-laminating system features steel stressing rods that compress the deck laminations transversely and thereby provide continuity among the individual laminations and a degree of plate action. Changes in the moisture content of the wood change the dimensions of the deck, and creep of the compressed wood perpendicular to the grain may relax the forces in the rods. Definitive data are needed to determine whether the system retains its effectiveness under such changes and to quantify the changes so that construction and design criteria may be developed.

At FPL and the UW, we have initiated a laboratory study to determine the long-term effectiveness of the stress-laminating system. Five deck panels, 5 ft square by 1 ft deep, will be subjected to varying moisture conditions over time. Four control panels will be maintained at a constant moisture content. The panels will be made from creosote-treated and untreated Douglas-fir, penta-treated and untreated LVL, and CCA-treated southern pine. Four small specimens of untreated lumber will supplement the larger panels. In all specimens, the forces in the rods and the dimensions of the panels will be monitored. Industry cooperators in this work include Wheeler Consolidated, the Trus-Joist Corporation, and the Southern Forest Products Association.

Bridge Monitoring

Perhaps the most important requirement for implementing the stress-laminated timber bridge research is demonstrating the successful performance of the system under field conditions. Design criteria and guidelines cannot be formalized until field performance is verified. Although considerable research has been completed by the UW, FS, and the Ontario Ministry of Transportation and Communication, little quantitative information exists on the field performance of stress-laminated bridges. Of primary concern are the effects of moisture changes and the long-term effects of wood creep on the level of tension in the stressing rods. The bridges in Canada have performed well, but many of the bridges constructed in the United States differ from their Canadian counterparts in their configurations and design parameters. Thus, we have initiated a study to monitor and evaluate the performance of prototype structures in the United States.

We have developed a general procedure for monitoring field performance. During initial stressing of a bridge, or subsequent restressing, load cells are installed on several rods to monitor changes in the forces. In addition, string lines may be installed to monitor creep-related deformation. Moisture contents of the wood are also monitored. Some monitoring activities may continue for as long as five or ten years.

To date, instrumentation has been installed on three National Forest (NF) bridges: the Mormon Creek parallel-chord bridge in the Hiawatha NF in upper Michigan, the Zuni Creek LVL bridge in the Kaniksu NF in Idaho, and the Iron River solid-sawn bridge in the Chequamagon NF in Wisconsin. Three new structures will be installed in the Monongahela NF in West Virginia this fall. We have cooperated with NF engineers in developing the design criteria for these bridges.

Changes in bridge stiffness will be monitored through field load tests (fig. 2). The bridges will be load tested as soon after installation as possible and again about a year later. To date, the Mormon Creek and Zuni Creek Bridges have been tested. The former is scheduled for a second load test in the fall of 1988.

In addition to the bridges that are being monitored as a part of the FS-UW cooperative program, two stress-laminated bridges built in mid-1987 are being monitored as part of other research programs at Pennsylvania and Colorado State universities. Reports on a bridge in Pennsylvania have been distributed and reviewed. Work is underway to monitor the performance of a bridge near Fort Collins, Colorado. In addition, Forest Products Laboratory and the Trus-Joist Corporation are cooperating with Dr. Hota GangaRao of West Virginia University in monitoring a stress-laminated bridge built of LVL and hardwood. One load test has been completed on this bridge.

Crushing Tests

One problem noted in several experimental bridges has been excessive crushing of the area beneath the bearing plates. We have recently tested laminated blocks that could be used to support the bearing plates. We conducted these tests in compression perpendicular to the grain using laminated oak and southern pine blocks. Preliminary indications are that these materials will have substantially less initial deformation and compressive creep than the materials currently used beneath the bearing plates.

Design Specifications

We are developing a guide for designing stress-laminated decks based on the results of all the research described in this paper. An important first step in obtaining approval of this system by AASHTO was to present a summary of our research to meetings of both the Transportation Research Board and the AASHTO Committee on Timber Bridges. A first draft of a design guide is being prepared and will be sent out for review soon. The guide is not complete; it is missing information on moisture effects and protection against corrosion. This information is being collected as part of our present research program. We are working with the National Forest Products Association (NFPA), who will take the lead in proposing new design criteria for timber bridges to AASHTO.

CONCLUDING REMARKS

The University of Wisconsin-Madison and the USDA Forest Service are engaged in a comprehensive cooperative research program to investigate and define the performance of stress-laminated bridges. Proposed design criteria based on completed and present research are being developed for consideration by AASHTO.

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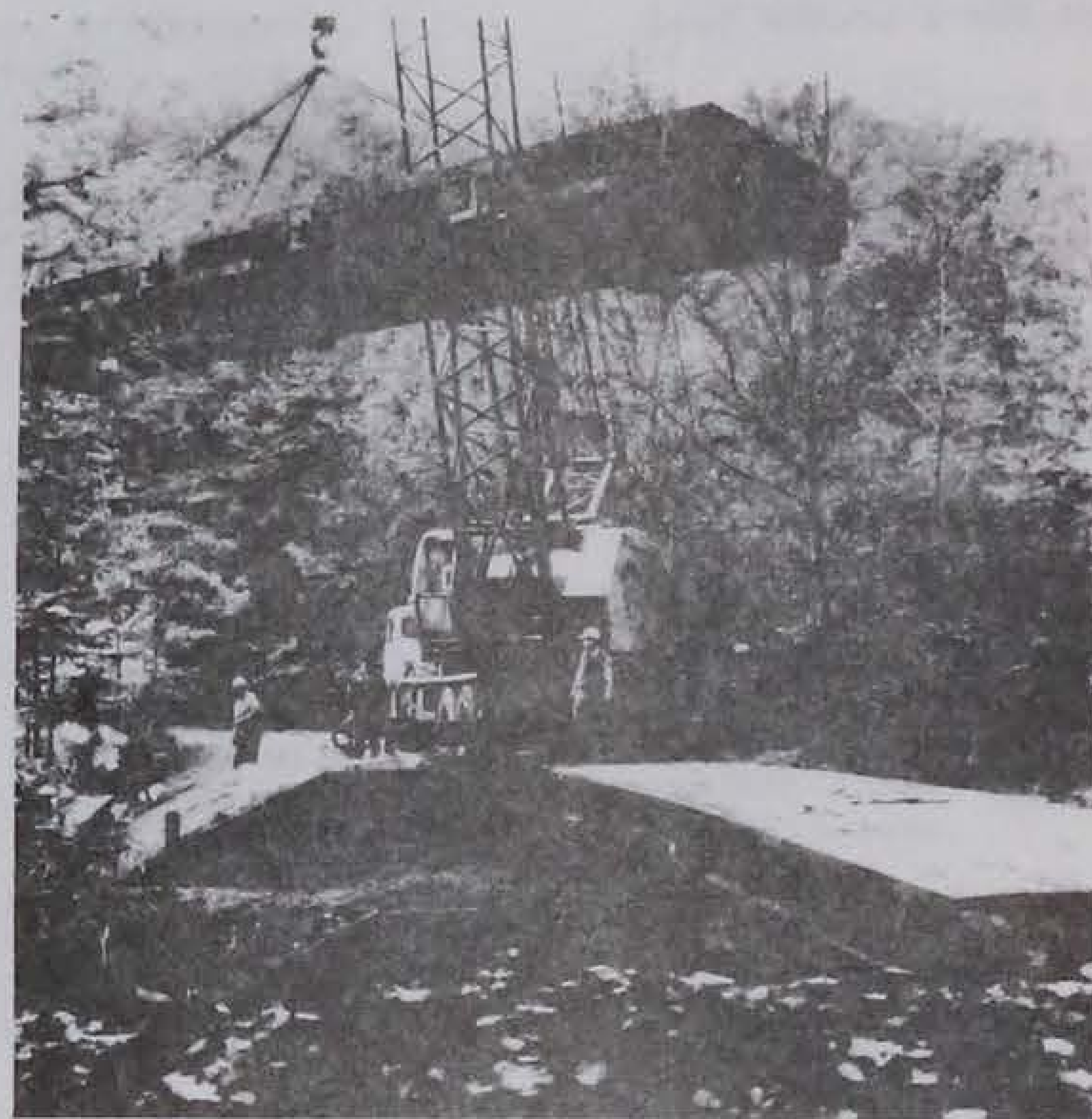

Figure 1. Parallel-chord stress-laminated bridge under construction.

Figure 2. Load-testing an experimental stress-laminated bridge.

SESSION IIA

ALTERNATE METHODS OF BRIDGE STRENGTHENING

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SYNOPSIS

An experimental and analytical research program has been developed to investigate two methods of strengthening existing bridges. One method investigates the effects of applying rotational restraint at the supports of existing stringers to reduce midspan moments. A second method employs external post-compression forces in the negative moment regions of continuous stringers to reduce stress in this region. Experimental results will be used to validate finite element models for use in investigating the effect of these strengthening methods on a broad range of bridges.

INTRODUCTION

Through the completion of several research projects sponsored by the Iowa Department of Transportation (Iowa DOT) [1, 2, 3], a procedure for strengthening bridges (simple and continuous spans) by post-tensioning has been developed. Research related to the strengthening of simple-span bridges by post-tensioning was completed in 1985; it resulted in a strengthening manual for use by practicing engineers. Phase I (laboratory investigation) of the research on post-tension strengthening of continuous-span bridges has been completed [3]; however, the field study (Phase II) is still in progress.

As a result of work on the previously mentioned projects, as well as National Cooperative Highway Research Program Project 12-28(4) [4], several other bridge strengthening approaches have been conceived. Two of the more promising of these strengthening concepts are presented in this summary paper. It is believed that in certain situations, these methods will provide strengthening alternatives that may be more efficient than post-tensioning. The two proposed strengthening concepts are presented separately for clarity in the following sections.

STRENGTHENING TECHNIQUE I: PROVIDING PARTIAL END RESTRAINT

Objective and Scope

The primary objective of this research is to determine the feasibility of strengthening stringer bridges by the addition of partial end restraint. This method will reduce the existing positive moment at midspan of the stringers. The investigation includes the following tasks

Task 1: A comprehensive literature review has been conducted.

Task 2: An analytical study using finite element analysis is being used to investigate thoroughly the strengthening potential of additional end restraint. Several bridges are being analyzed to determine the magnitude of end restraint required to produce desired stress reductions. Several other items are being examined in this phase of the research, including

- the feasibility of strengthening continuous as well as simple-span bridges.
- the most efficient placement of end restraint brackets for simple-span and continuous bridges.
- the effect of increasing loads due to the end restraint brackets on the abutments or piers.

Task 3: A bracket or other device for providing end restraint will be designed on the basis of data obtained during Task 2. A series of laboratory tests will be performed on the most promising bracket configurations to determine their physical characteristics, both for use in the analytical model and for field application. The practicality of field installation and maintenance, as well as widespread applicability, will be important considerations in the design of the brackets.

Task 4: The most promising brackets will be placed on an existing 1/3-scale model bridge in the laboratory. This bridge model was developed for a previous Iowa DOT sponsored research project. Data from laboratory bridge tests will be used to verify the feasibility of the end restraint method. Changes in load distribution due to the various bracket configurations and locations will be evaluated experimentally. These data will also be used to validate the previously developed analytical model.

Task 5: The results obtained from the previous four tasks will be summarized and presented in a final report.

Progress to Date

A review of literature related to the general behavior of connections has been completed. No information on the design of end restraint connections for bridge stringers was found. Most of the connection literature was related to building connections; some of this information has been useful, particularly that related to the method of characterizing connection behavior.

An analysis of typical bridge configurations has been performed for the purpose of determining the amount of end restraint required for reasonable stress reductions. In addition, the magnitude of the resulting forces at the supports due to the restraint was of interest for designing the end restraint mechanism and for determining the effect of the forces on the substructure. A general purpose finite element program, ANSYS, was used in the analysis.

A number of concepts for the restraint mechanism have been developed. Several brackets utilizing these concepts have been designed and fabricated. They are currently being evaluated with a test setup consisting of a simple-span test beam. The supporting abutments for this test beam have been constructed to simulate actual field conditions.

A steel wide-flange section W24x84, 20 ft. in length is being used for the full-scale experimental program. To determine the end restraint mechanism effectiveness, strains and deflections are being monitored at various locations along the beam. In addition, rotations are being monitored at the beam end with a transit mounted on the beam's top flange. An overall view of the test setup is shown in Fig. 1. One of the end restraint mechanisms being tested is shown in Fig. 2. Preliminary data using only the lower flange bracket shown in Fig. 2 indicate that midspan stresses can be reduced by approximately 10 percent.

STRENGTHENING TECHNIQUE II: POST-COMPRESSION OF STRINGERS

Objective and Scope

Work recently completed on an Iowa DOT-sponsored research project determined that when the positive moment regions of continuous bridges are post-tensioned, stress reduction can also be obtained in the negative moment regions. However, in certain instances, additional stress reduction is required in the negative moment region, which obviously requires additional post-tensioning force. Because of the proximity of the bridge deck in the negative moment region, the required connections in most instances would require removal of a portion of the bridge deck. One method of avoiding this problem is to apply tension to the lower flange rather than compression to the upper flange (see Fig. 3). This portion of the investigation is broken into the following tasks

Task 1: A comprehensive literature review has been conducted.

Task 2: Various types of compression struts and required brackets will be designed, fabricated, and load tested. Of particular interest will be the buckling strength of the compression strut.

Task 3: The most promising compression strut will be instrumented and attached to the negative moment region of the full-scale composite beam fabricated in an earlier strengthening investigation [3]. The desired force in the strut will be applied by use of the hydraulic cylinder shown in Area C of Fig. 4. After the force is obtained, the hydraulic cylinder will be removed and "filler plates" placed in this area. Strain data (from strain gages previously installed) and deflection data from LVDT's will be obtained from the postcompressed composite beam for various magnitudes of moment and tension forces.

Task 4: Compression will then be applied to the various negative moment regions of the one-third scale, three-span model bridge. Because of the space restriction, which results from the scale of the model, the compression struts will be placed either below the beams or off to the side. With strain gages already in place, the behavior of the system can be determined as well as strain distribution data, elastic shortening data, and the like. The model will be subjected to various combinations of post-compression, post-tensioning, and vertical loading.

Task 5: A finite element model is being developed and calibrated with the data obtained from the bridge model. The verified finite element model will then be used (a) for the investigation of post-compressing other types of continuous bridges and (b) for the application of post-tensioning and post-compression to the positive and negative moment regions, respectively, of a continuous bridge. The analytical model will also be used to determine load distribution in various bridges that have been subjected to post-compression.

Task 6: Results of the laboratory investigations and the finite element analysis will be presented in a final report.

Progress to Date

The research to date has concentrated on Tasks 1, 2, and 3. Little additional literature beyond that found during previous investigations has been located under Task 1; thus, most of the effort has been directed toward Tasks 2 and 3.

Seven different compression struts with variations were given a preliminary review. On the basis of the review, two of the most promising struts were selected. Those struts are illustrated in Fig. 5.

The tube compression member shown in Fig. 5a is essentially the strut originally proposed but with an improved end bracket. The truss shown in Fig. 5b is a variation of the strut system originally proposed that should provide improved force distribution because of the angle of the tubes with respect to the beam.

Both strut systems have been fabricated; the tube compression strut member is currently being tested.

ACKNOWLEDGEMENTS

This research program is being conducted by the Bridge Engineering Center under the auspices of the Engineering Research Institute of Iowa State University. The investigation is funded by the Iowa Highway Research Board and the Highway Division, Iowa Department of Transportation.

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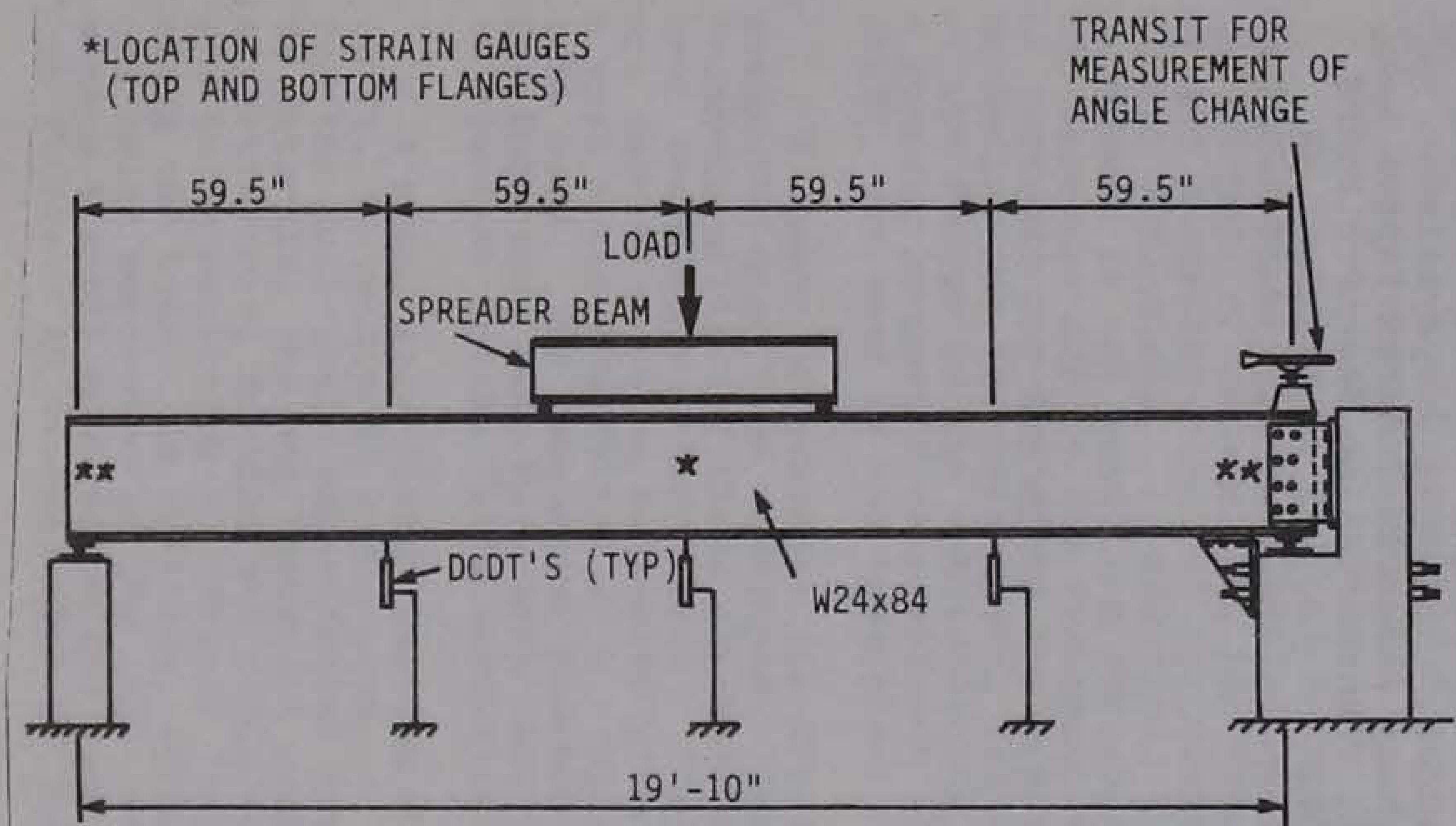


Fig.1. Experimental test set-up for end restraint strengthening.

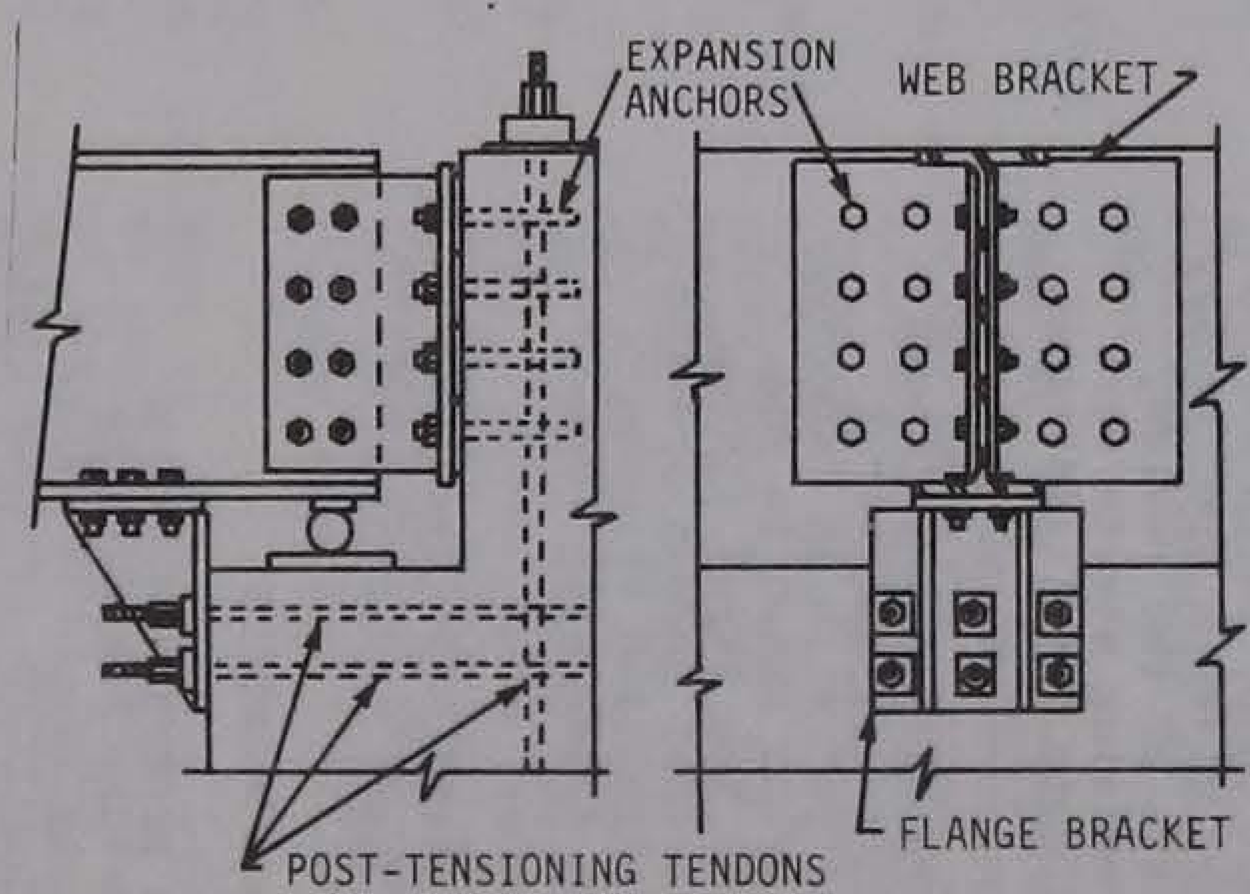


Fig 2. End restraint brackets attached to test beam.

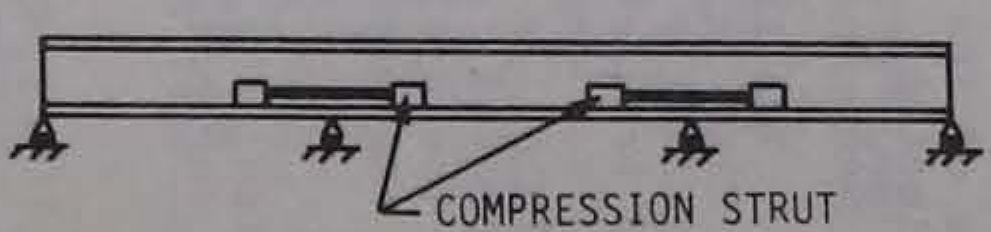


Fig. 3. Stringer strengthened with compression strut.

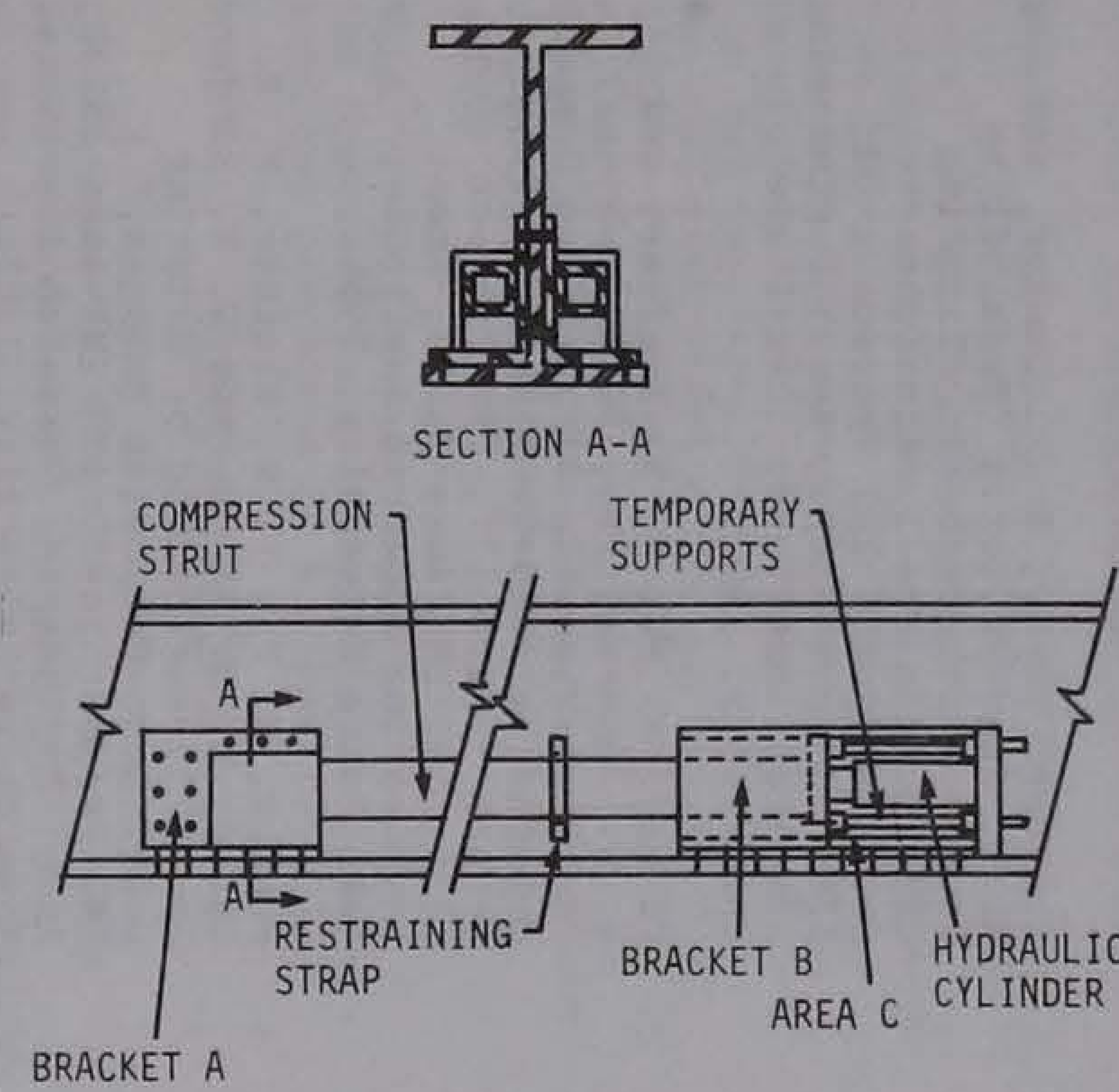
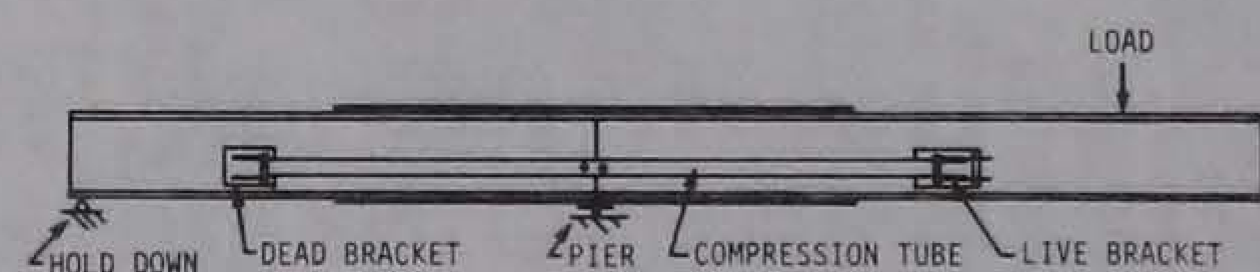
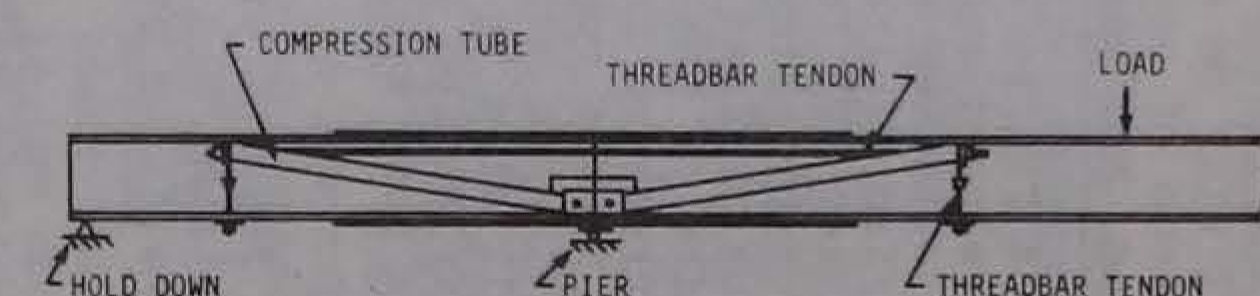


Fig. 4. Details of compression strut.



a. tube compression member



b. tube plus thread bar truss

Fig. 5. Post-compression strut details.

BRIDGE REPLACEMENT SYSTEM

William D. Lockwood, P.E.; President

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Dayton, Ohio 45429

SYNOPSIS

Replacement or rehabilitation of deficient bridges on the nation's highway system is one of the major challenges in restoring our aging infrastructure system. Any improvements that will effect the durability and economy of replacement structures will generate dramatic effects in program of this magnitude. The Con/Span bridge replacement system utilizes new technologies, construction capabilities and current labor and material considerations to achieve a unique combination of economy and durability.

NATIONAL DILEMMA

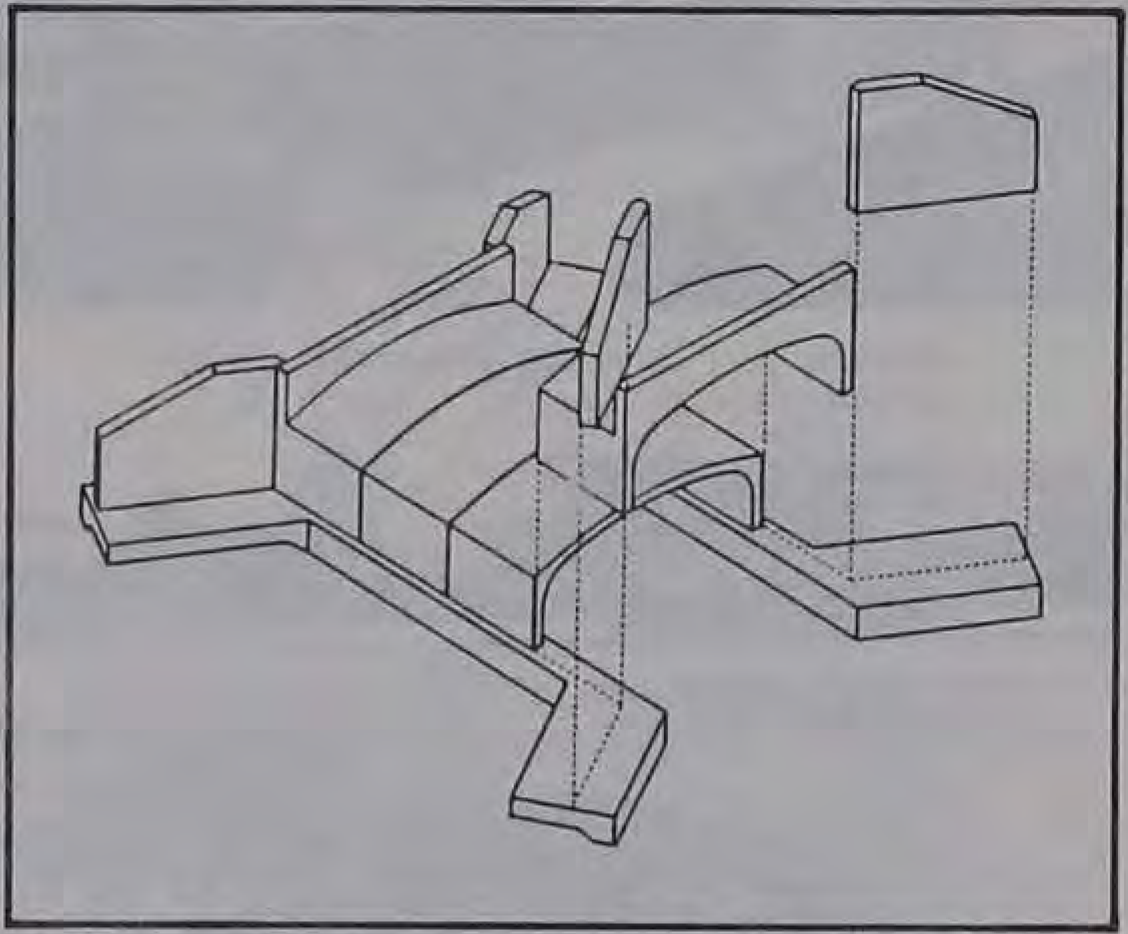
Deficient bridges on the nation's highway system are an infrastructure problem that continues to worsen despite increasing funding for replacements and repairs. Age, corrosion and increasing traffic volumes and load intensities have taken their toll on structural performance. Also, many installations can no longer meet geometric requirements for safe operations. A recent analysis of this country's bridge inventory indicates that 40% of the nearly 600,000 bridges in the U.S. need replacement or rehabilitation.

Failures of large structures have drawn attention to the dilemma and have been the incentive for national programs for both inspection and repair. Most of the attention has been focused on major bridges where a collapse is catastrophic. Remaining are an endless number of small bridges that are obsolete.

CON/SPAN SYSTEM

Culverts are often a preferable solution for small bridge replacements. They have several advantages over conventional structures. Their initial costs are lower for most site conditions. Considerations for materials and details can nearly eliminate maintenance requirements. Culverts also permit flexibility for widening of roadways and change in alignment. Their use eliminates the hazard of bridge deck icing.

A precast concrete arch-box shape unit developed and patented by Con/Span Culvert systems in Dayton, Ohio extends the economy of culverts to greater spans. It generates efficient arch action while maintaining vertical side walls. This provides large waterway openings with minimum headroom and a compact shape. Precasting allows excellent quality control and speed of installation.



BRIDGE REPLACEMENT SYSTEM

Precast units are supplied in 16, 20 and 24 foot spans with rises from 4 to 10 feet. Headwalls can be supplied with the units. They are set on cast-in-place footings and wingwalls are added to complete the structure.

Concrete is certainly the favored material for culverts and small bridges. Long, maintenance-free service and hydraulic efficiency are well established attributes. The strength of the Con/Span section eliminates most restrictions for cover and live load limitations and allows backfill to be placed expediently with heavy equipment.

Figure 1. Replaced structure.

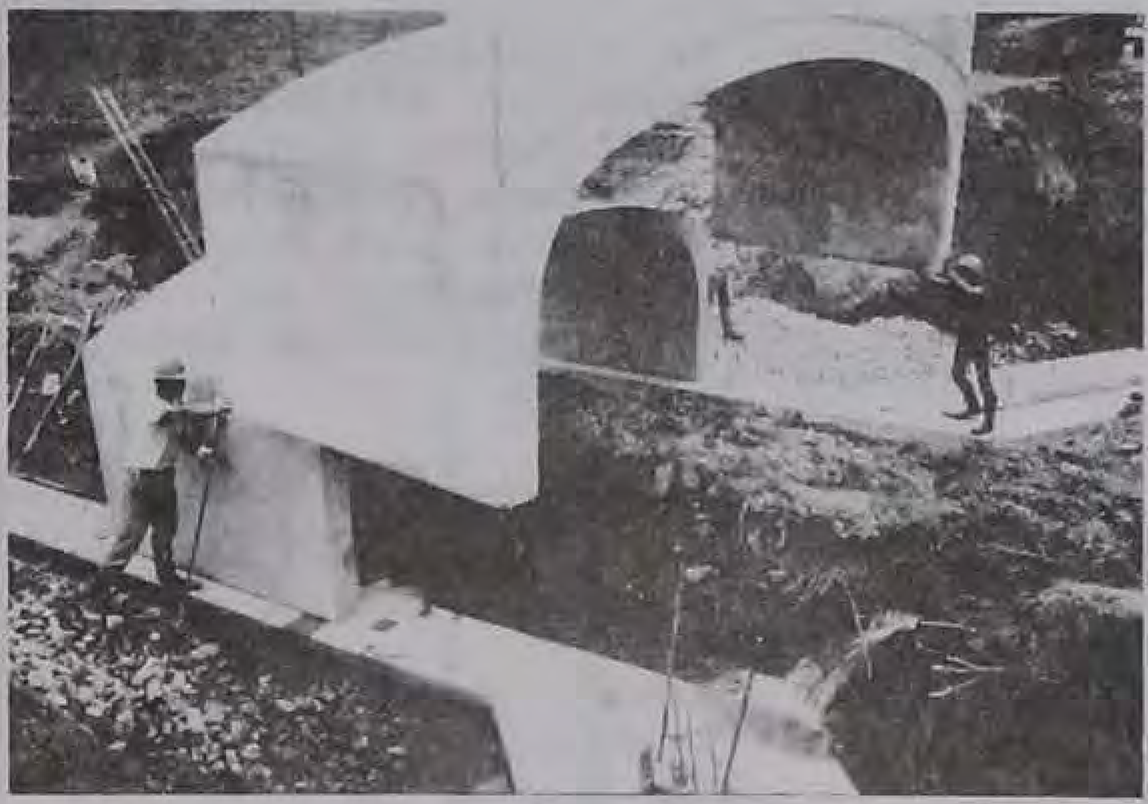


Figure 2. Setting pre-cast units.

Figure 3. Completed structure.

INITIAL RESEARCH & DEVELOPMENT

The system is both the result and the subject of an extensive R & D program. Initially, shape configurations were studied that would effectively mobilize soil interaction to allow efficient structural action. The finite element computer program, CANDE, was adapted and used to model the action of the structure in a soil mass. Refinements were made in the design and wide ranges of soil types, unit dimensions, cover depths and loadings were studied.

Manufacturing and installation considerations were vital bases for economics. The unit geometry was coordinated with a forming system that allowed all span and rise combinations to be produced from the same form. Weights and lengths of units were selected for economical shipping and setting. The field installation process is fast, simple, and accomplished with minimal plan preparations.

LOAD TESTS

Full scale testing began in 1986. All testing was done in cooperation with the Ohio Department of Transportation. Loads were applied to a buried structure and performance was carefully monitored. Correlation was verified to the CANDE finite element soil-structure analysis. The load intensity was carried to a failure load of five (5) times the AASHTO design load. It was significant that the structure maintained this ultimate load through continuing displacements induced by the jacking system.

Figure 4. Load test.

Additional full scale testing was carried out early this year with loads applied at the edge of the precast units to verify that splices for shear transfer was not required between units.

RESEARCH IN PROGRESS

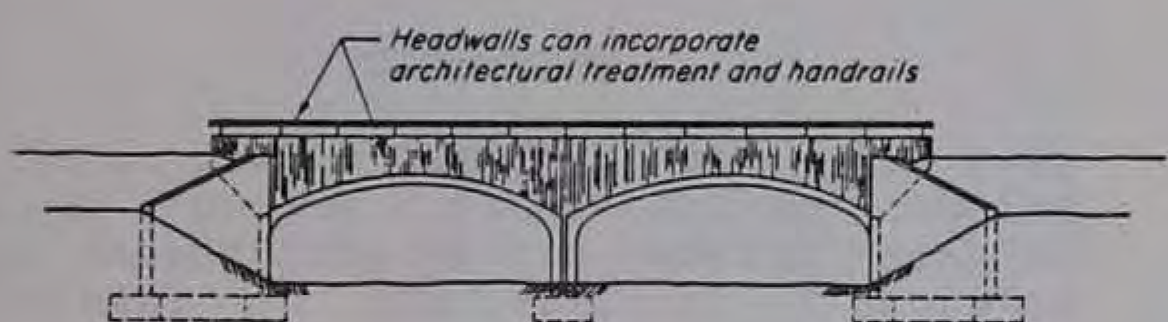
Current research and development is underway in several areas. Software is being completed to fully automate the plan preparation process. Data corresponding to the many variables concerning an installation are entered as spread sheet information that will generate final construction plans on CADD equipment.

Larger spans are being analyzed. A second system to construct 30 or 36 foot spans will soon be load tested. Reinforcing will probably be achieved by post-tensioning at the precast plant. This will facilitate better handling and shipping conditions, as well as provide the required strength for the applied loads.

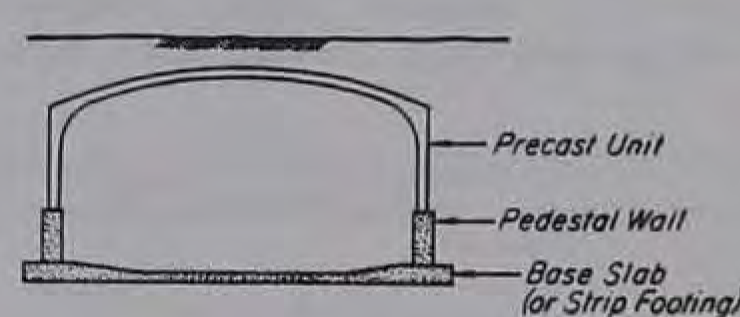
The inherent structural capacity of the system makes it attractive where extreme loads are involved. Designs are being studied for applications under high fills, railroads and airports.

Additional development is underway to furnish a total precast system. Wingwalls and optional precast footings will be available to allow total prefabrication.

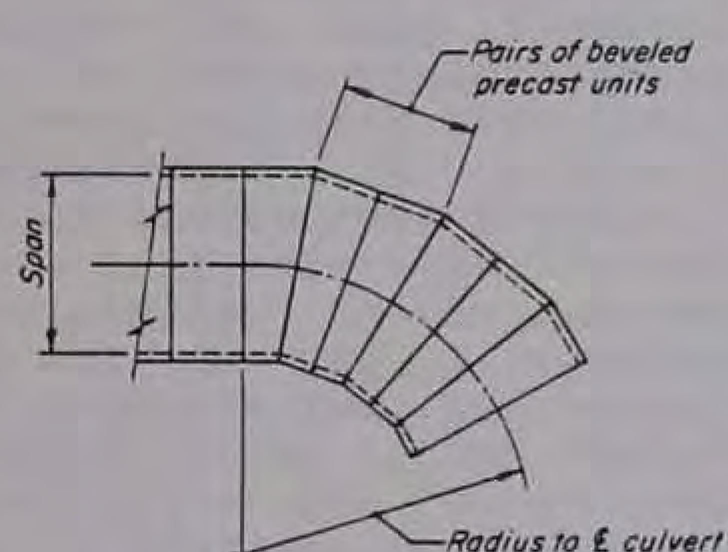
While the system is currently being approved and supplied in much of the U.S. and Canada, research and development is far from complete to utilize the full advantages of this bridge replacement system.



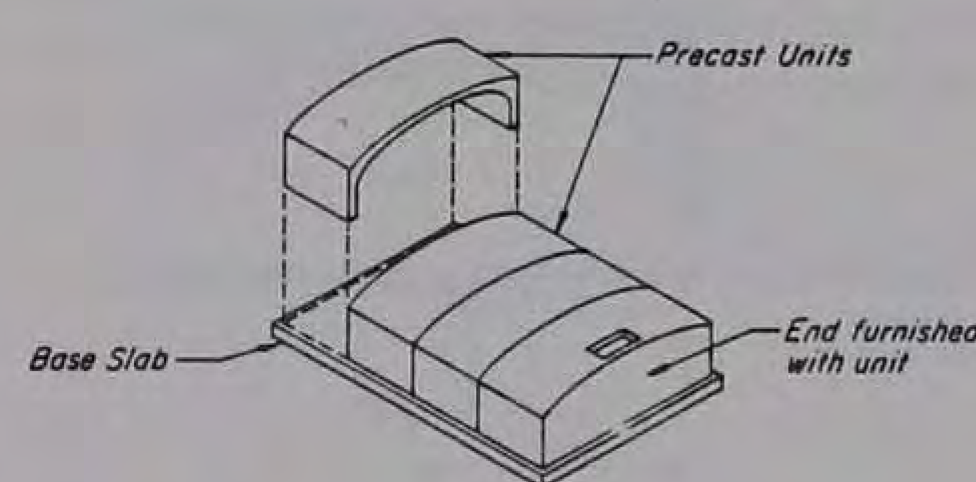
MULTIPLE CELL INSTALLATION



PEDESTALS TO INCREASE RISE



INSTALLED ON HORIZONTAL RADIUS



UNDERGROUND STORAGE OR WATER RETENTION

ADDITIONAL APPLICATIONS

COUPLING JOINTS OF PRESTRESSING TENDONS

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SYNOPSIS

The reduced fatigue life of post-tensioned segmentally erected concrete bridges with prestressing tendons coupled in the construction joints is addressed. Factors contributing to this problem such as the reduced nonlinear stress distribution in the construction joint vicinity and the stress concentrations between tendon and anchorage elements are investigated to assess the cyclic capacity of embedded tendon couplers. The basic problems of designing post-tensioned concrete bridge structures for cyclic loads both from the capacity as well as the load side are discussed.

INTRODUCTION

Large cracks and in one case even ruptured tendons found during routine bridge inspections in the Federal Republic of Germany in the vicinity of coupling joints of post-tensioned continuous bridge structures have led to a series of investigations into the behavior of coupling joints, [1] and [2].

Such coupling joints of post-tensioning tendons are common in segmental bridge construction, particularly in construction methods developed and used frequently in Europe, such as incremental launching and span by span erection with traveling self-supporting falsework. It is this latter group of bridge structures, however, which showed crack concentrations in the coupling joint vicinity. In the span by span construction method, Fig. 1, construction joints with couplers for the tendons are generally placed close to the theoretical point of inflection for dead load plus prestressing to minimize reinforcement requirements in the construction joint. Analytical investigations into the crack development of this construction joint vicinity, summarized in [1] and [2], showed that the initial crack development is caused by several factors such as highly nonlinear stress states due to concentrated anchorage forces, unaccounted differences in the actual dead load distribution, temperature gradients, and increased prestress losses in the tendon couplers. Once the section is cracked, the reduction in stiffness, and with it higher cyclic stress levels, must be evaluated very carefully since changes in tendon geometry due to lower grade steel of the anchorage-coupler assembly are cause for stress concentrations and lower fatigue limits.

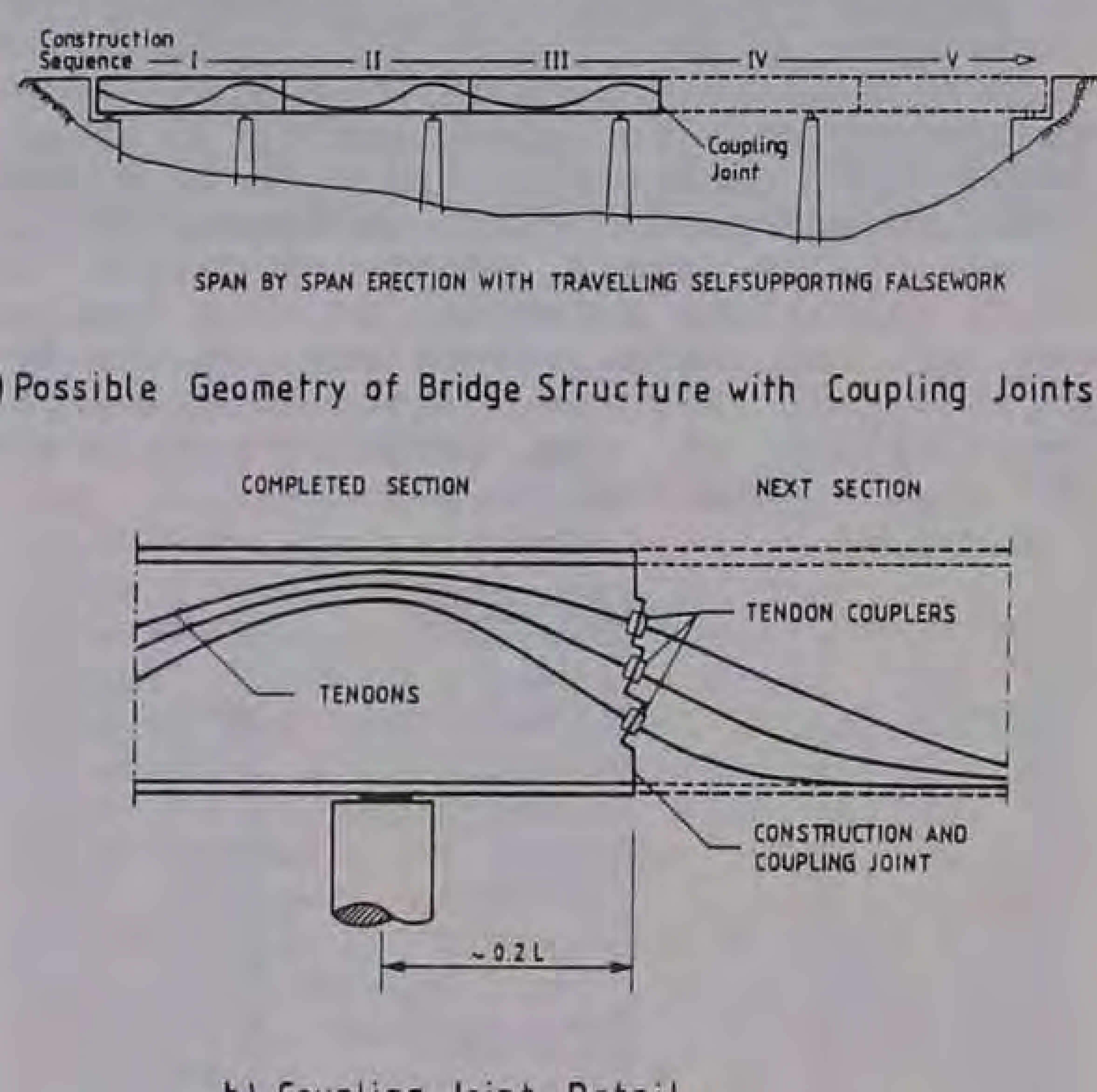

Fig. 1 Coupling of Post-tensioning Tendons

To verify the possible causes for crack initiation in the coupling joint vicinity and to assess the actual fatigue limit state of such a construction joint requires full-scale prototype investigations. Since in-situ measurements of the actual stress state in the coupling joint vicinity would require not only specialized field equipment for collecting the necessary data points, the large number of required measurements would also directly impact the construction progress and is therefore not feasible. Thus, only full scale (due to the hardware dependency) laboratory testing can provide the necessary experimental verification for the stipulated theories concerning the coupling joint behavior.

PRELIMINARY STUDIES AND OBJECTIVES

Analytical investigations into the stress state of segmentally erected, coupled and post-tensioned concrete members have shown that the stress state in the construction joint vicinity is far from uniform and can be significantly reduced from the constant state of precompression typically assumed in the design process. To illustrate this phenomena a simple plane stress model was employed to analyze the longitudinal stress state in a segmentally erected and centrically loaded (post-tensioned) concrete member, as shown in Fig. 2. The resulting stress state is a superposition of the two load cases shown and the contour lines represent the longitudinal stress field in multiples of the resulting uniform prestress-f_{po}. This analytical model clearly shows the reduction of the state of precompression with values of 0.5 f_{po} in the construction joint and as low as 0.3 f_{po} in the direct vicinity of the construction joint along the edge of the investigated element. Since this decreased state of precompression superimposed with uniform tensile stress fields are a clear indication for crack initiation potential in this region, one of the main objectives of the described test program was the experimental verification of the actual stress state due to the construction sequence.

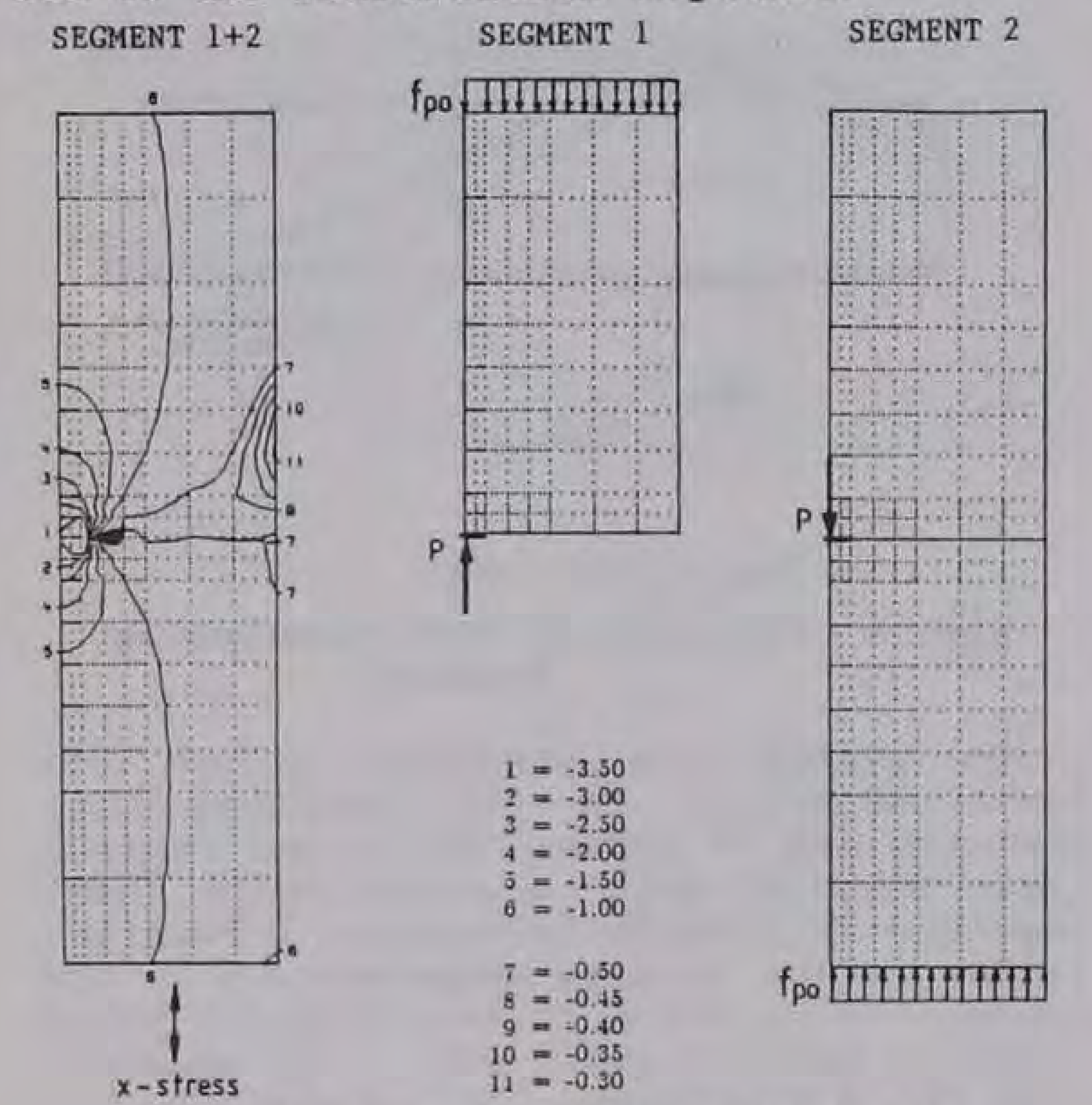

Fig. 2 Example of Longitudinal Stress State

The second objective for the full scale experimental program arises from the observed but not well quantified reduced fatigue behavior of embedded tendon anchorage assemblies. The FIP Commission on Prestressing Steels recommended stress range limits for tendon-anchorage systems of less than half the recommended stress range limits for the tendons. The fatigue problem is most likely to be amplified in coupling joints where not only tendon anchorage systems are present, but the weak construction joint zone enhances the development of isolated discrete cracks which in turn concentrate the mechanical action of cyclic loading directly in the coupler-tendon transition zone. Since the tendon-coupler interface seems to be a major influencing factor, different hardware configurations for tendon-couplers need to be investigated.

EXPERIMENTAL PROGRAM

Subject of the full-scale experimental program are both, coupled bar and strand systems. The two basic coupling elements for the two systems are shown in Fig. 3. The coupler for the bar system is a threaded sleeve which accepts thread bars from both ends and the strand coupler consists of a heavy anchor disk with conical holes in both directions which hold three-piece wedge action grips for anchoring 7 wire strands. The assembled tendon-coupler systems in the actual construction joint are shown in Figs. 4 and 5, respectively. The spring loaded conical wedges which accept the strands for the subsequent tendon segment can be seen along the perimeter of the coupling disk in Figure 5. Since some of the fatigue problems with tendon couplers have been associated with the concentrated heavy coupling disk, Fig. 6 shows the staggered arrangement of individual strand couplers investigated as an alternative to the coupling disk.

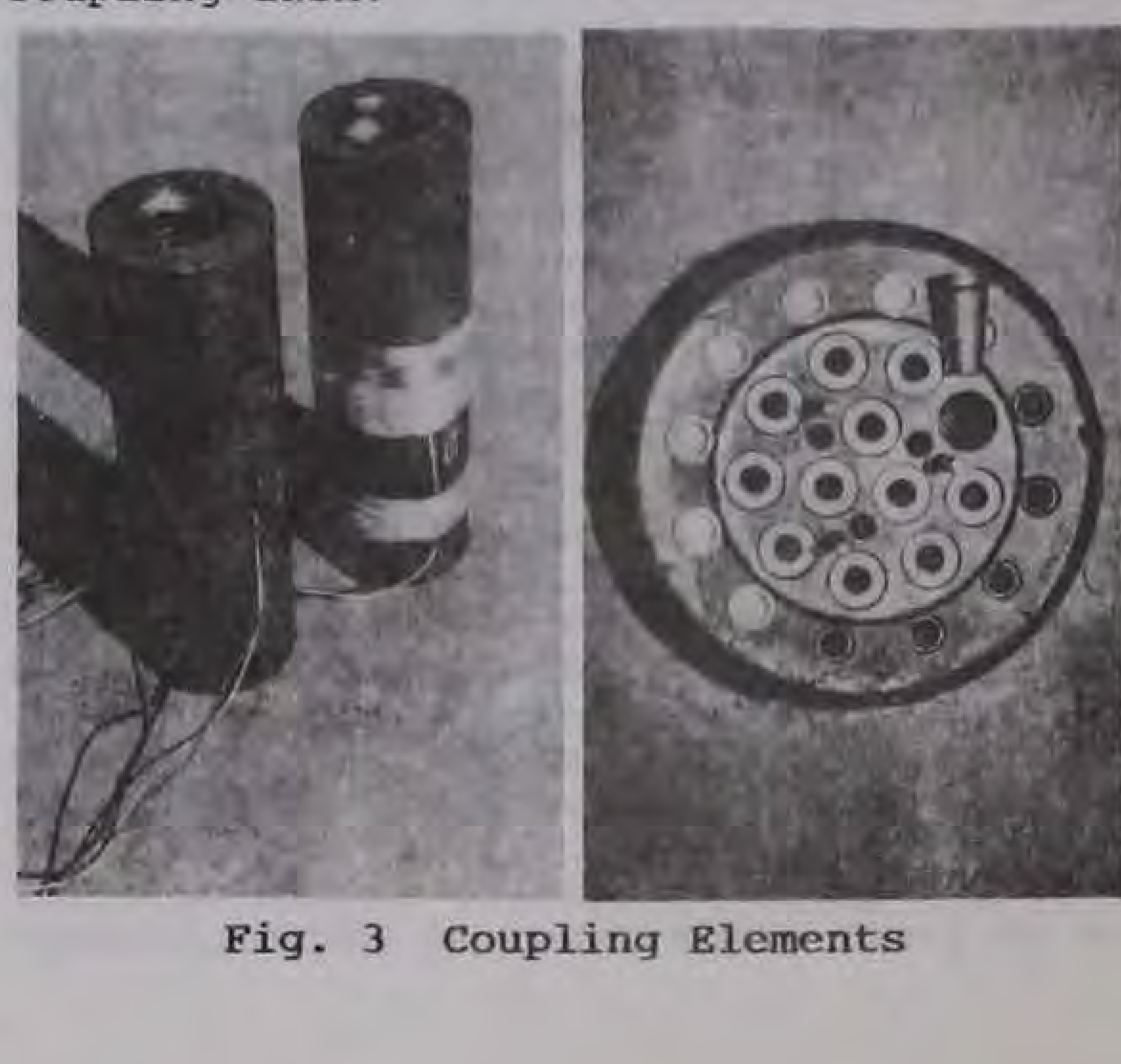

Fig. 3 Coupling Elements

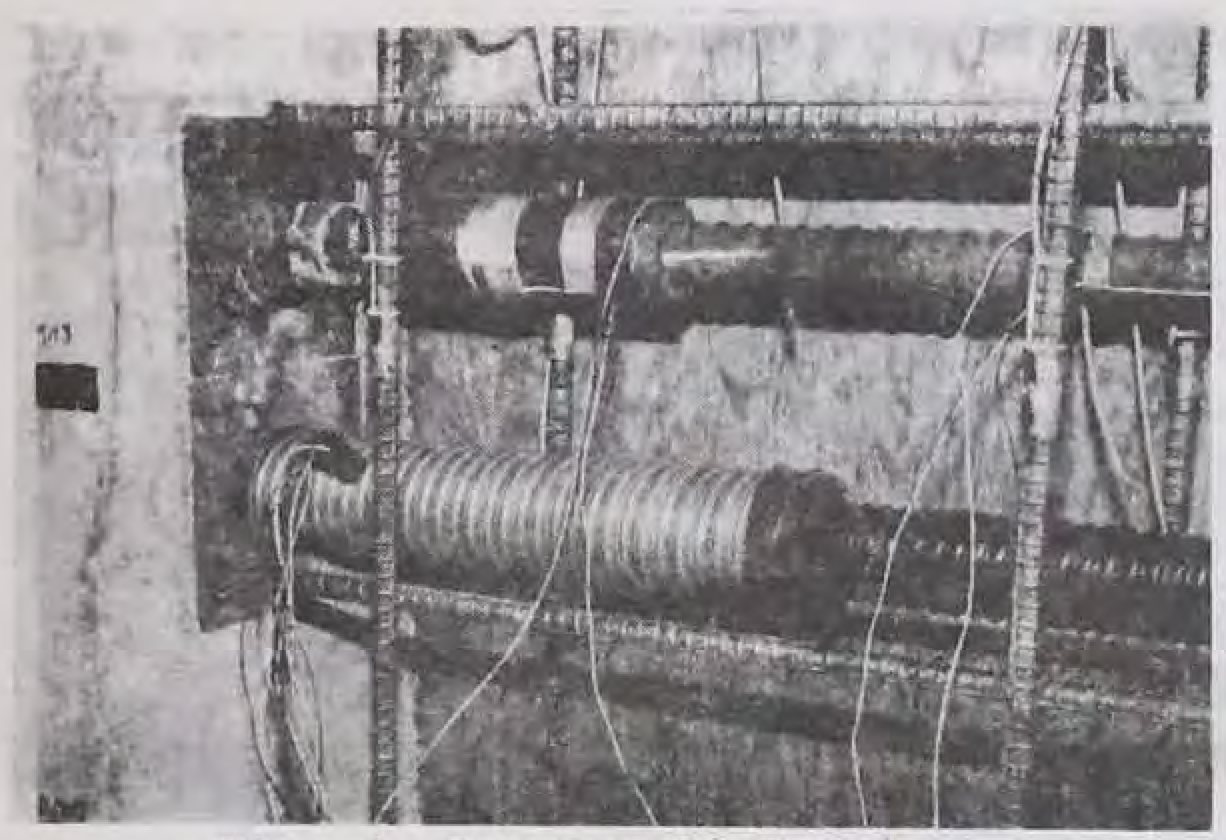

Fig. 4 Bar Coupling Joint

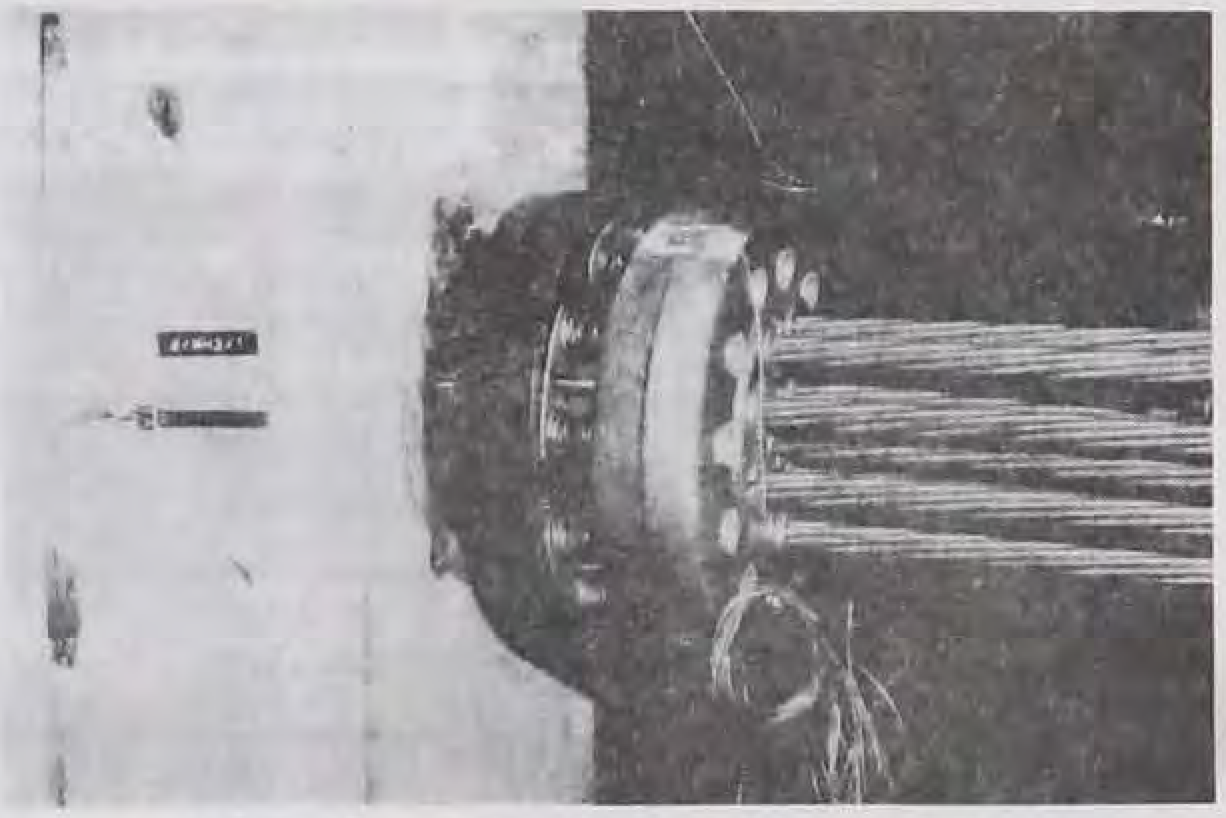

Fig. 5 Strand Coupling Disk

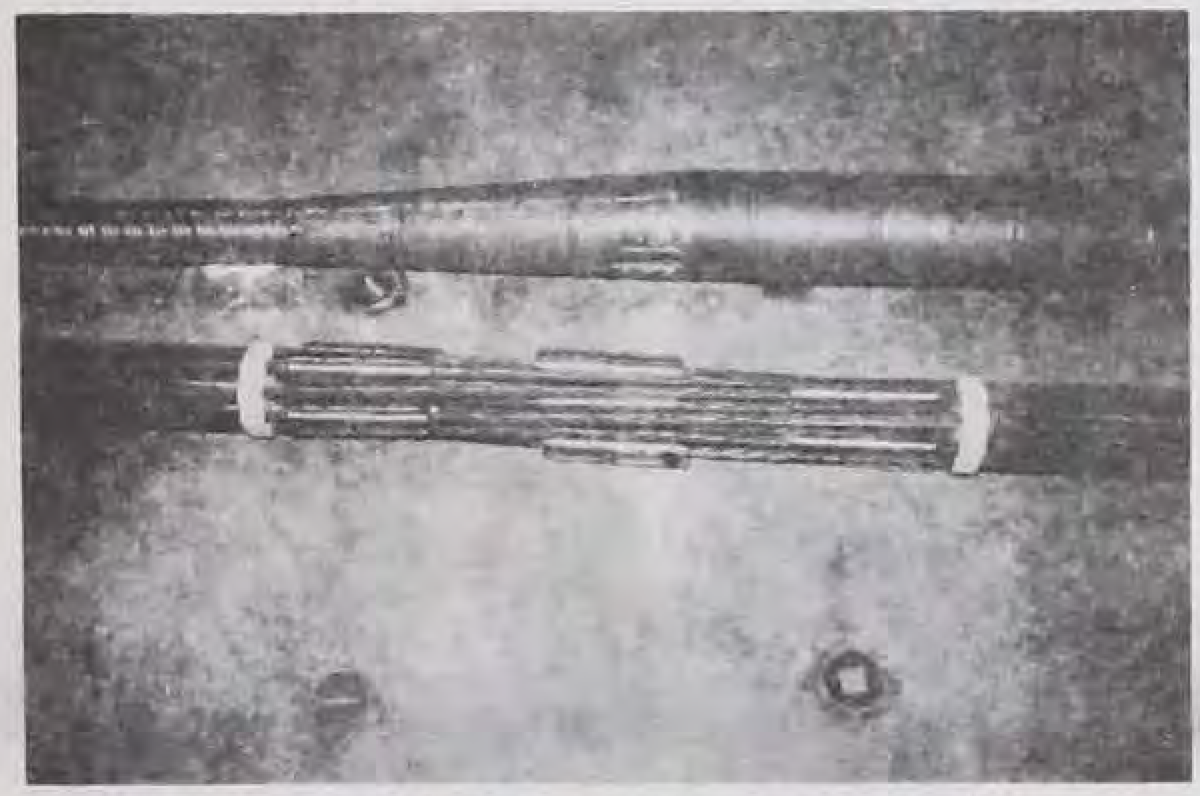

Fig. 6 Individual Strand Couplers

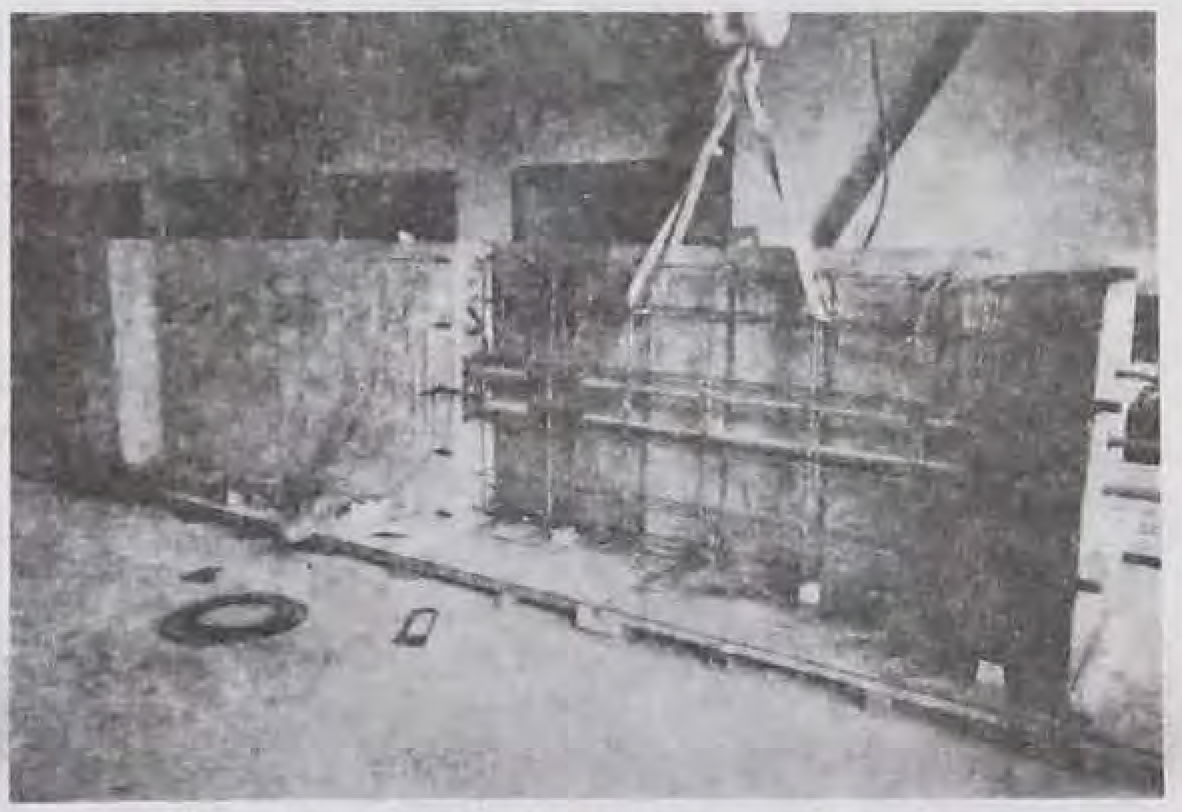

Fig. 7 Test Specimen Overview

The total test matrix consists of 4 segmentally erected and post-tensioned specimens, see Fig. 7, with the following post-tensioning details:

SP-1: Two bar system with thread bar couplers (Fig. 4)
SP-2: 12 strand (0.6" Ø ea.) system with coupling disk (Fig. 5)
SP-3: 12 strand (0.6" Ø ea.) system with individual strand couplers (Fig. 6)
SP-4: 9 strand (0.6" Ø ea.) system with coupling disk (similar to SP-2)

The test program consists of monitoring the actual stress state due to the construction procedure by means of DEMEC (Demountable Mechanical Gage) measurements, 2" concrete surface electrical resistance gages and electrical resistance rebar strain gages. This stress state is monitored with time to capture possible time-dependent stress relief. Subsequently each specimen will be subjected to cyclic stress levels of about 0.15 f_{pu} in the tendon to determine the fatigue limit state.

PRELIMINARY RESULTS

The construction and post-tensioning phase of the 4 full-scale test specimens is summarized in Table 1. Casting and stressing dates as well as lock-in force and average prestress levels show the overall state of each test specimen. A comparison of analytically predicted and experimentally observed longitudinal prestress levels (~500psi) along the top edge of the test specimens after the construction and post-tensioning of both segments is depicted in Fig. 8 for the first segment of specimen SP-3. Due to the inherent problems of stress-strain conversions of experimental strain measurements analytical strain values were matched to experimental strain values at a reference point, see Fig. 8, which is the St. Venant distance of 1.0 × d away from the construction joint. The experimental strain values clearly show the same reduced compressive stress state as predicted by the analytical model.

Time-dependent strain measurements, as depicted for specimen SP-2 in Fig. 9, show that the basic shape of the longitudinal strain profile along the top edge of the test specimens does not significantly change with time which indicates the continued presence of the reduced compressive stress state in the construction joint vicinity.

Table 1: Summary of Segmental Post-Tensioning

Specimen	Segment	Cast Data	Post-Tension Date	Max Force During Stressing	Lock-In Force P_o	Lock-In Force as % F_u	Average Comp. Stress $f_c=P_o/bd$
1	1	01/26/87	02/02/87	379.2kip	363.4kip	77	505psi
	2	02/04/87	02/11/87	379.2	363.4	77	505
2	1	07/08/87	07/15/87	492.2	407.8	58	566
	2	07/20/87	07/27/87	457.0	393.7	56	546
3	1	08/25/87	09/01/87	421.9	337.5	48	478
	2	09/03/87	09/10/87	464.0	379.7	54	527
4	1	09/14/87	09/21/87	421.9	342.7	65	476
	2	09/23/87	09/30/87	421.9	363.9	69	498

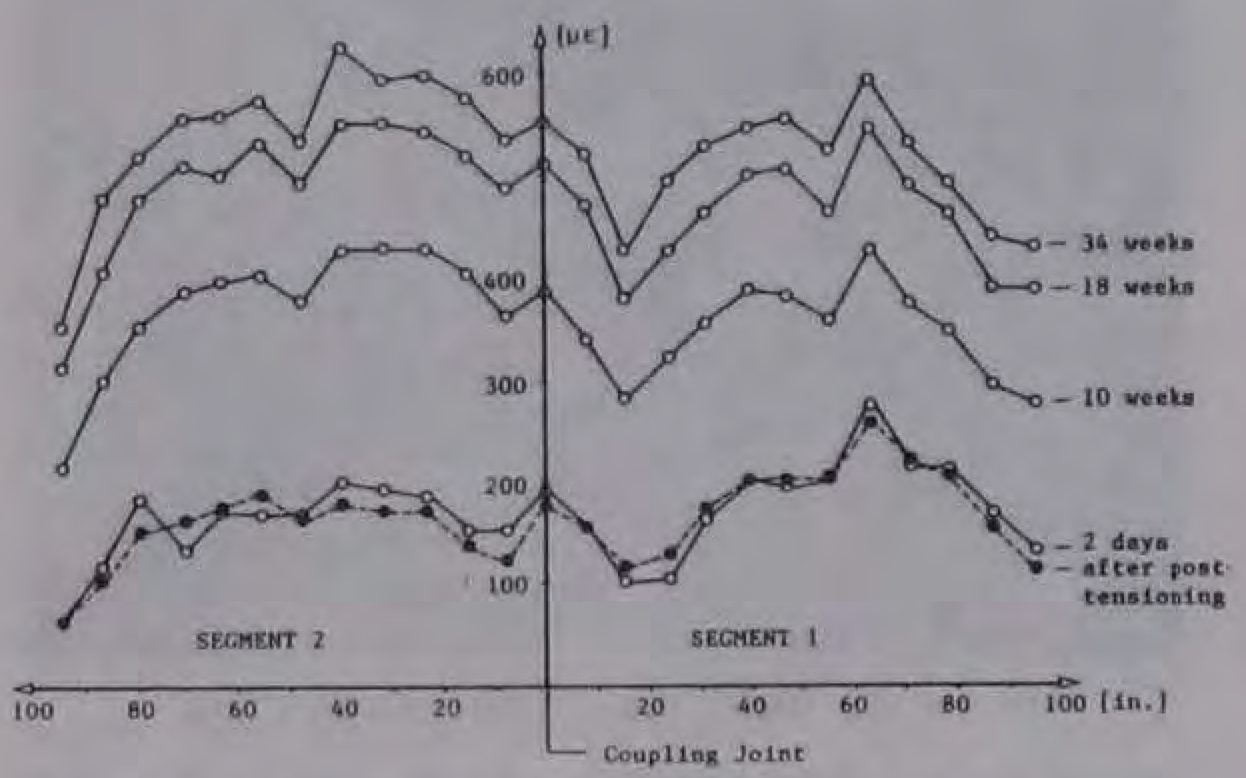

Fig. 8 Longitudinal Edge Strain State

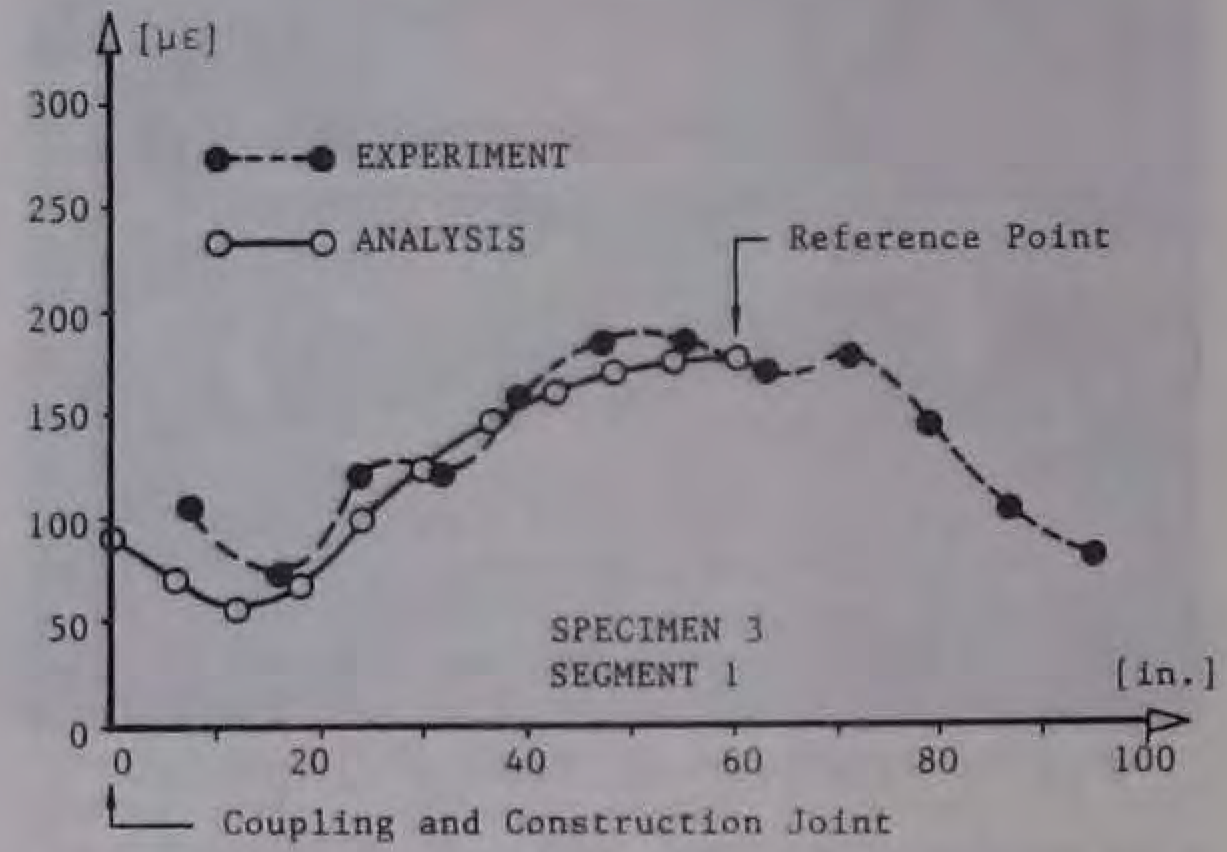

Fig. 9 Strain Variation with Time, SP-2

SUMMARY AND CONCLUSIONS

Experimental investigations to date have confirmed non-uniform reduced compressive stress states in segmentally erected and post-tensioned concrete specimens, where the tendons are stressed and fully grouted for each segment and subsequently coupled for the next segment. This stress state in the coupling joint vicinity does not significantly change with time which leaves regions of reduced prestress which are predominant for crack initiation with superimposed tensile stress states due to moment redistribution, temperature effects, etc. The cyclic behavior of coupling joints still needs to be investigated and design criteria need to be developed which not only address the fatigue limit state of embedded couplers but also the load side with realistic repetitive cyclic load and stress level assumptions.

ACKNOWLEDGEMENTS

The described research project has been funded by the National Science Foundation through Dr. J.B. Scalzi, under Grant No. CES-8552672. The post-tensioning hardware and equipment, as well as engineering support, was donated to the project by Dywidag Systems International, USA, Inc.

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POST-TENSION STRENGTHENING OF A THREE-SPAN CONTINUOUS BRIDGE

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SYNOPSIS

Based on previous success with post-tension strengthening of single span composite bridges, the authors have begun a research program to extend the strengthening method to continuous bridges. In the first phase of the research completed in July 1987, the authors analyzed and tested a laboratory bridge model. Strengthening of a similar, three-span continuous, composite bridge in Fonda, Iowa is in progress during the second phase of the research.

INTRODUCTION

The concept of strengthening single span, composite steel beam and concrete deck bridges has been developed by the authors through several Iowa DOT research projects. The single span projects were conducted in the following phases:

Phase I: Literature review, laboratory model testing, and analysis [5]

Phase II: Field bridge strengthening and analysis [4]

Phase III: Field bridge observation [1]

Phase IV: Design manual preparation [3]

Since the authors prepared the design manual in 1985, the Iowa DOT has used the allowable stress design methodology for strengthening of several bridges by post-tensioning. Post-tensioning is now one of the methods in regular use by the Iowa DOT for single-span bridge strengthening.

In 1986, the authors began a research program for continuous, composite bridges similar to the program for single-span bridges. Phase I, "Strengthening of Existing Continuous Composite Bridges," was completed in July 1987 [2]. Phase I verified that continuous bridges can be strengthened by post-tensioning.

In most bridges, the desired stress reduction in the positive moment regions as well as in the negative moment regions can be obtained by post-tensioning only the positive moment regions. This was determined theoretically using a finite element analysis and experimentally by testing various post-tensioning schemes on a one-third scale model bridge. In addition to testing the post-tensioning schemes on the model bridge, several of the schemes were tested on a full scale composite beam which was supported to simulate the negative moment region of a bridge beam.

OBJECTIVES AND SCOPE

The objectives of Phases II, "Strengthening of an Existing Continuous Span Steel Beam-Concrete Deck Bridge by Post-Tensioning," which is in progress, are:

to design and install a post-tensioning strengthening system on a continuous span, steel beam-concrete deck bridge, and

to instrument and test the bridge for determination of deflections and strains.

The objective of Phase III will be to document the behavior of the bridge for one year following the strengthening.

The bridge which is being strengthened was selected by a team consisting of members of the Office of Bridge Design of the Iowa DOT, county engineers, and the authors. The bridge is of the standard, Iowa DOT V12 Series. The laboratory model constructed in Phase I was based on a V12 bridge 125 feet in length; the bridge near Fonda, Iowa in Pocohontas County presently being strengthened is 150 feet in length. The similarity in the two bridges will permit comparison of laboratory and field data.

For the V12 and similar V14 Series of bridges constructed in Iowa between 1950 and 1975, there are about 90 bridges on the primary system and about 360 on the secondary road system. The Fonda, Iowa bridge is on the secondary road system and is typical of the many V12 bridges which need strengthening in order to avoid load posting.

Section and framing plan views of the Fonda, Iowa bridge are given in Figure 1. The bridge has four beams which are spliced at the dead load inflection points of the center span. The two A7 steel exterior beams are W 21x62, and the two A7 steel interior beams are W 24x76. Because all beams bear at approximately the same elevation, the difference in beam heights provides a deck crown. The bridge deck varies in thickness from 6 7/16 inches over each beam to 6 3/4 inches between beams. The bridge has steel channel diaphragms located at abutments and piers and wide flange diaphragms in each span. (See Figure 1.)

RESEARCH PROGRAM

Phase II of the research program currently is in progress. The bridge has been analyzed for dead and Iowa legal truck live loads using a computer program for continuous span bridges developed by the Iowa DOT. The analysis indicated stresses above the allowable operating stress, and the bridge presently is posted for reduced loads.

The envelope of the bottom flange stresses for the exterior bridge beam is shown in Figure 2b. Under most conditions, the beam would be considered composite only within positive moment regions for dead load. Because the post-tensioning to be applied will add positive moment near the piers, the exterior beam was considered to be composite except where coverplates are effective. Figure 2a shows the extent of the assumed composite action. Because of the extended region of composite action, Figure 2b indicates only a small change in stress at the transition between the composite beam and non-composite beam with coverplates.

The basic plan for the post-tensioning is to apply eccentric forces to all positive moment regions of all beams. Overstresses in negative moment regions at piers will therefore be alleviated by the longitudinal distribution of post-tensioning moments. Figure 2d shows stresses which would be applied to the bottom flange of the exterior beam with all beams post-tensioned with 100 kips as illustrated in Figure 2c. For the finite element analysis on which the stresses are based, the tendons for end spans were located 3 1/4 inches above the bottom flange, and the tendons for center spans were located 5 inches below the bottom flange. Based on further analysis, the post-tensioning forces will be adjusted to achieve the desired stress reduction.

During bridge analysis, a crew of graduate and undergraduate students has been preparing the bridge for strengthening and testing. Brackets for post-tensioning tendons, welded strain gages, and double-nutted, high strength steel bolt shear connectors [4] are currently being installed. Before testing in August 1988, instrumentation will also be installed for measuring deflections on the bridge.

During testing, the bridge will be subjected to the following loading conditions to determine its response, strains, and longitudinal and vertical displacements:

(1) An overloaded truck at various predetermined locations on the bridge.

(2) Various stages of the post-tensioning sequence.

(3) The same overloaded truck at the same locations after post-tensioning in order to determine the effectiveness of the strengthening system.

During the future Phase III, the bridge will be inspected every three months for one year to monitor the bridge's behavior. At the end of the one year period, the bridge will again be service load tested to determine any behavior change from the initial testing.

SUMMARY

A three-span continuous, composite bridge in Fonda, Iowa has been selected for strengthening by post-tensioning. Previous testing and analysis of a three-span laboratory bridge model indicated that it was feasible to strengthen such a bridge by post-tensioning only the positive moment regions. The strengthening of the Fonda, Iowa bridge is proceeding on that concept.

To date, the Fonda, Iowa bridge has been analyzed and is being prepared for post-tensioning and testing during August 1988. After testing, the bridge will be monitored for one year, at which time it will be retested to determine any changes in behavior. If the strengthening is successful, the authors propose to develop a design manual for strengthening of similar continuous span Iowa bridges.

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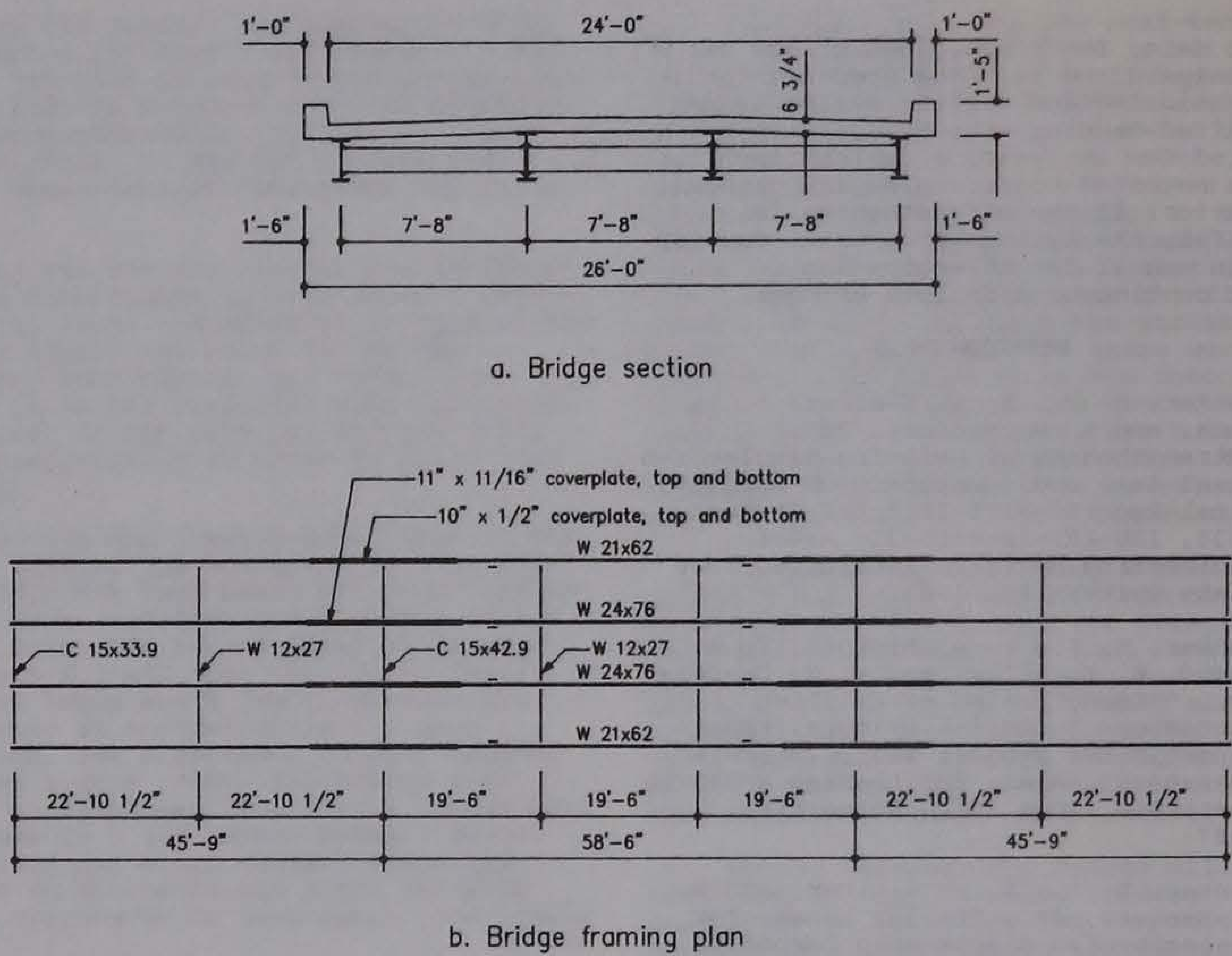

Figure 1. Three-span, 150 ft. bridge near Fonda, Iowa

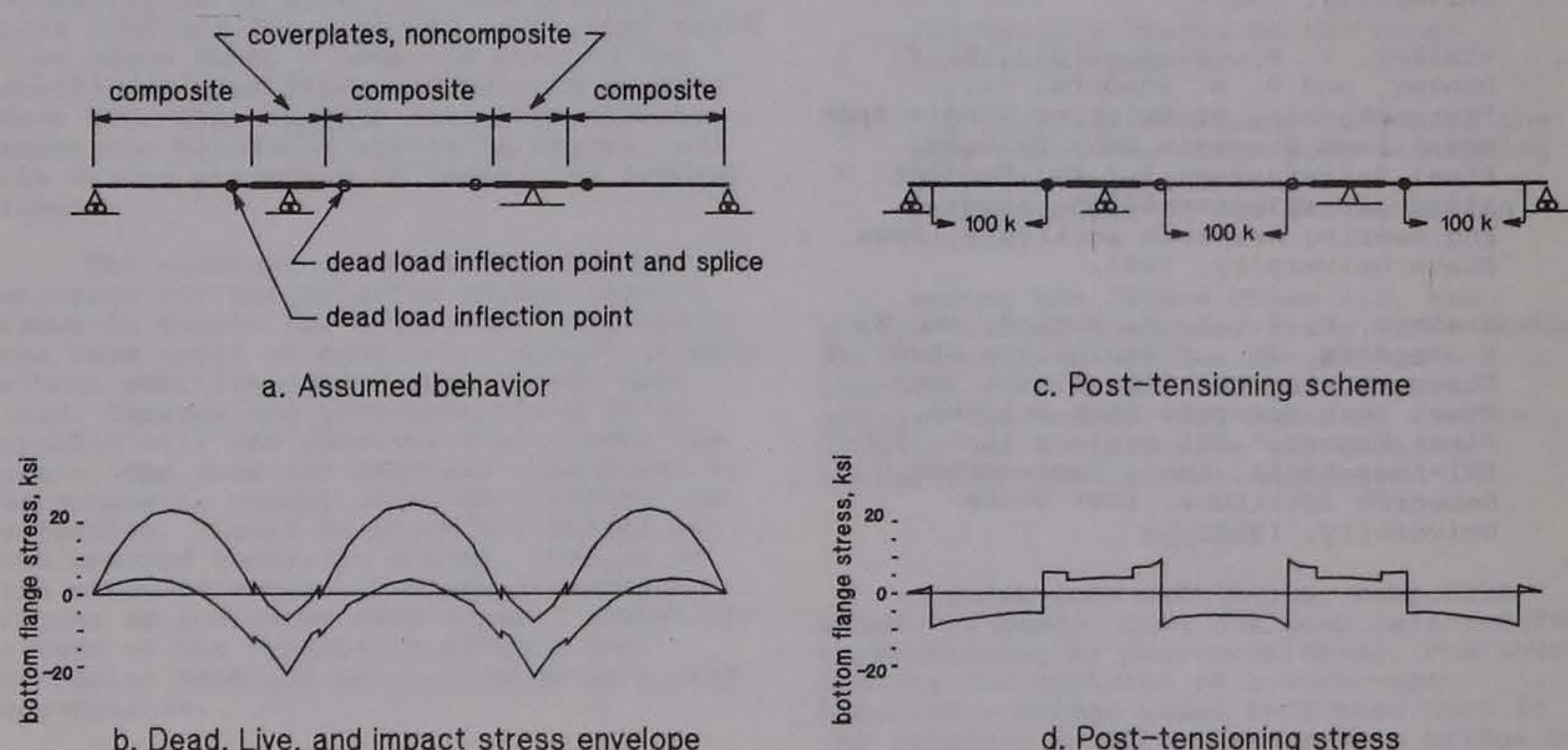

Figure 2. Exterior beam behavior and stresses

PRESTRESSING OF A COMPOSITE STEEL BRIDGE

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SYNOPSIS

Design and construction aspects of prestressed and non-prestressed composite steel bridges are studied in this report. The study evaluates the potential for increasing live load capacity by prestressing composite steel girders. The report considers bridge construction methods both with and without shoring as well as stage prestressing. It is shown that prestressing technique could enhance the live load capacity of a bridge by as much as 280 %.in shored and stage prestressed construction. The result is only about 40 %, in case of no-shoring. The live load capacity could not be increased in no-shored case, as more dead loads are to be carried by steel stringer with no composite action.

INTRODUCTION

Prestressed composite beams have been used in both bridge and building construction but they are not so commonly used as prestressed beam. Their rare use could be due to lack of design procedures in manuals and codes or insufficient test data. There is also inadequate information on maximum carrying capacity of the section and the effect of type of construction when prestressed. The primary objective of this study is to find possible advantages of a prestressed composite steel bridge over a conventional composite steel bridge. Construction methods using shoring or no-shoring and stage prestressing are studied.

Prestressed composite steel beams have been studied by several authors. In 1949, F. Dischinger (1) published a series of articles proposing the prestressing of entire bridges by means of high-strength cables. A United States patent for a composite beam and concrete slab system prestressed by means of draped cables was granted to L. Coff in 1950 (2). The analysis of prestressed composite beams using both elastic assumptions and approximate ultimate strength methods were discussed in papers by R.Szilard (3) and P.G.Hoadley (4). Hoadley's method of ultimate strength analysis was approximate,as tendon force was not determined from consideration of equilibrium and compatibility of deformation, but assumed to be equal to the tendon force occuring at first yield of the beam. Behavior of prestressed composite beams are also studied by R.S. Regan and N.W. Krahl (5).

It is believed, that since steel is strong both compression and tension, prestressing cause unnecesary induced stresses. Therefore this may not be economical or advantageous to strength. However this is not true,as the induced stresses cause either low tensile or compressive stress in top flange and only compressive stress in bottom flange. Low stress caused by prestress is an advantage for concrete decks. Bottom compressive stress increases the stress range available to live loads.

BEAM CROSS SECTIONS

Non symmetric sections are very efficient for external prestressing since a greater lever arm will be available for the prestressing force. The asymmetric sections could be formed either from geometry or composite material. The section properties may be obtained which satisfy the allowble boundry stresses. The top flange is always under- stressed in a composite or asymmetric section as the neutral axis is close to the top flange. By prestressing, the allowable stresses can be attained in both top and bottom chords. However, in a symmetric section designed as a prestressed beam, the top chord reaches the allowable stress first and bottom remains under-stressed. In order to utilize the section efficiently the prestressed beam should be an asymmetric section.

LOCATION OF CABLES

The behavior of the beam and its

economic performance depend to a great degree on the location of the prestressing cables. The cables may be built within or out-side of the beam. There is less direct stress and more bending stress due to prestressing, when the cable is moved away from the section. This leads to tensile stress on top flange and larger stress is available for live load moments. Thus, effectiveness of the section increases for live load capacity. This type of cable location and prestressing force need complicated anchor design and can cause local stress concentrations. Therefore the cables may be located just below the bottom tension flange for span moment and above the top flange for support moment. This arrangement gives sufficient room for stressing and efficient anchorage for cables. The number of cables and their cut-off lengths may be established from the bending moment diagram.

ANALYSIS

Structural engineers constantly search to find better and more economical structures. These attempts include combining the composite steel girder and prestressing to achieve an economic and efficient section. It is shown in the following ,the improvement in the load carrying capicity if the composite section is prestressed. The degree of improvement of the section also depends upon the type of construction.

A typical section of composite steel stringer bridge is taken from 'Highway Structures' Design Handbook by United States Steel (6). It is a 100',two span continious bridge. The size of plate girder and concrete deck thickness are shown in the Fig. 1. The loads considered and section properties are shown in Tables 1 & 2. The granular flux-filled studs of 7/8"x4" size are used to resist horizontal shear.

The following asumptions are made for this analysis:
1 Stresses are in the elastic range,
2)All moments are taken about the neutral axis,
3)Centroid and neutral axis of the section coincide,
4)Shear studs are capable of transfering the full horizontal shear.

(a) COMPOSITE STEEL BRIDGE WITH NO-SHORING

The stresses caused by dead loads and live loads are shown in the Fig.2., the compressive and tensile stresses are denoted by (+) and (-) respectively and their units are in ksi.

(b) PRESTRESSED BRIDGE WITH NO-SHORING

A prestressing force of 277.6 kips satisfies the top allowable stress condition. The calculated stresses for (Prestreess+D.L) and (Prestress+D.L+L.L) are shown in Figs. 3 and 4. The live load moments calculated for this case from the available stress is(20.0-17.27) x 6989/12 = 1590 ft-kips. Thus the live load capacity is increased by 40 % .

Higher compressive stresses are developed in top flange when concrete is placed. This is due to the non-composite action of steel and concrete,the neutral axis is close to the bottom flange. Allowable stresses are reached in top flange before the bottom flange reaches its allowable stress. Thus the section is not fully utilized by prestressing. Better utilization can be achieved by providing shoring or stage prestressing which reduce stresses in top flange.

(c) PRESTREESED BRIDGE WITH SHORING

In this case a prestressing force of 653.5 kips is calculated. This satisfies the allowable stress boundary conditions. The stress diagrams of (Prestress+D.L) and (Prestress+D.L+L.L) are shown in the Figs. 5 and 6. Here the D.L moment of the stringer is taken as 119.0 ft-kips . The live load moment calculated for this case from the available stress is 40.0x1303)/12 = 4343 ft-kips. Thus the live load capacity is increased by 280 %.

(d) PRESTRESSED BRIDGE: STAGE PRESTRESSING

Prestressing force of 471.4 and 186.65 kips is applied in two stages satisfying allowable stress condition. The stress diagrams are shown in the Figs. 7 through 10. In this case the available stress is 40.0 ksi, so the live load moment is 4343 ft-kips . Therefore capacity is also increased by 280 % .

ADVANTAGES OF PRESTRESSING

1) Live load capacity can be increased.
2) Fatigue life of the stucture can be increased by maintaining low stress range.
3) Lighter sections can be used thus it is more economical and easier to construct.
4) The catastropic failures can be reduced as cables act together with steel girders providing redundency.
5) Deflections can be decreased.
6) Span lengths can be incresed.

CONCLUSIONS

1) The prestressing steel areas required for increased capacity are only about 11%. of the steel section area.
2) Live load capacity of the section is increased by 280%. by prestressing.
3) This tecnique can be extended to continuous bridge also but the deck must be prestressed to take up tensile stress.

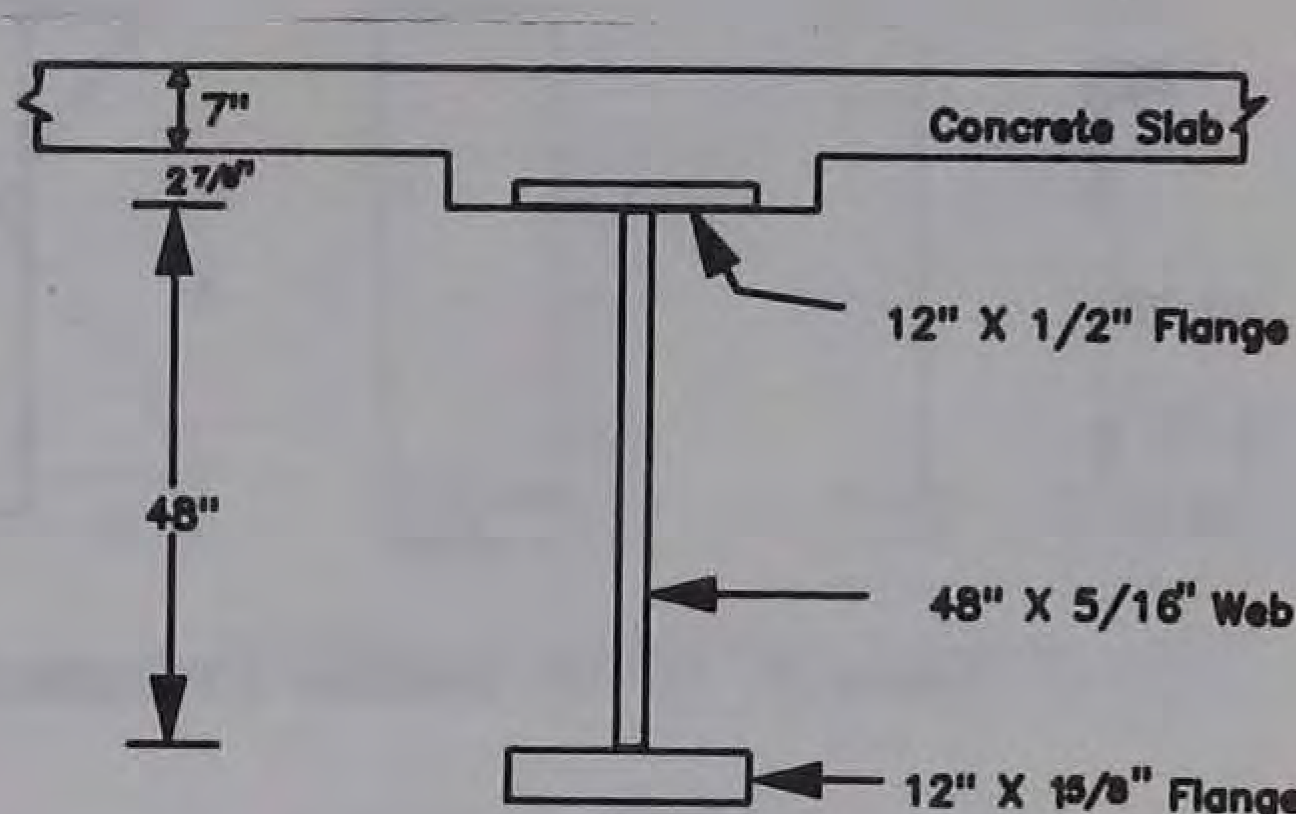

Figure 1 COMPOSITE SECTION

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DESCRIPTION	DL1	DL2	L.L + IMPACT
MOMENT Ft-Kips	630	116	1136

Table 1 Dead Load And Live Load Moments

DESCRIPTION	AREA SQ.IN.	MOMENT OF INERTIA. IN^4	SECTION MODULUS (BOTTOM) IN^3	SECTION MODULUS (TOP) IN^3
Steel Section	40.5	15600	903	475
Composite Section 3N = 24	65.0	38590	1210	2115
Composite Section N = 8	114.0	55070	1303	6969

Table 2 Composite Sectional Properties

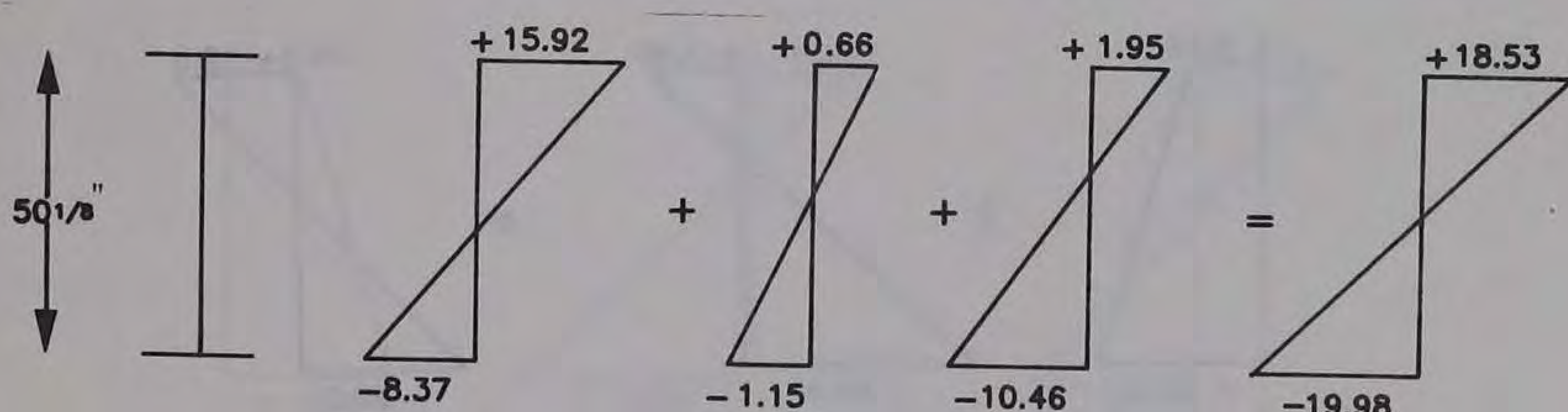

Figure 2 STRESS DIAGRAM (D.L + L.L): NO-SHORING

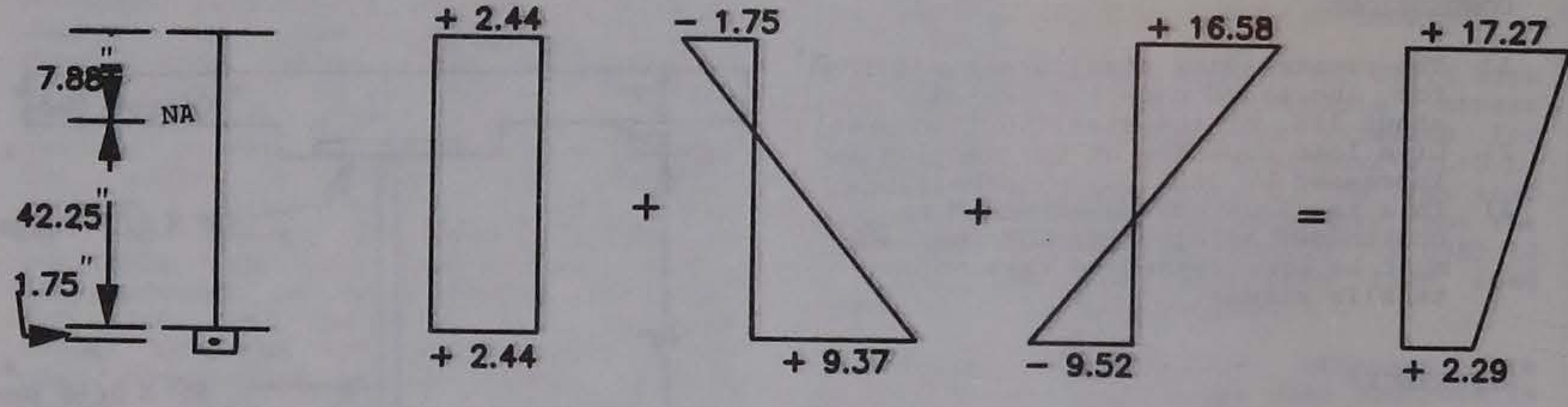

Figure 3 STRESS DIAGRAM (PRESTRESS + D.L): NO–SHORING

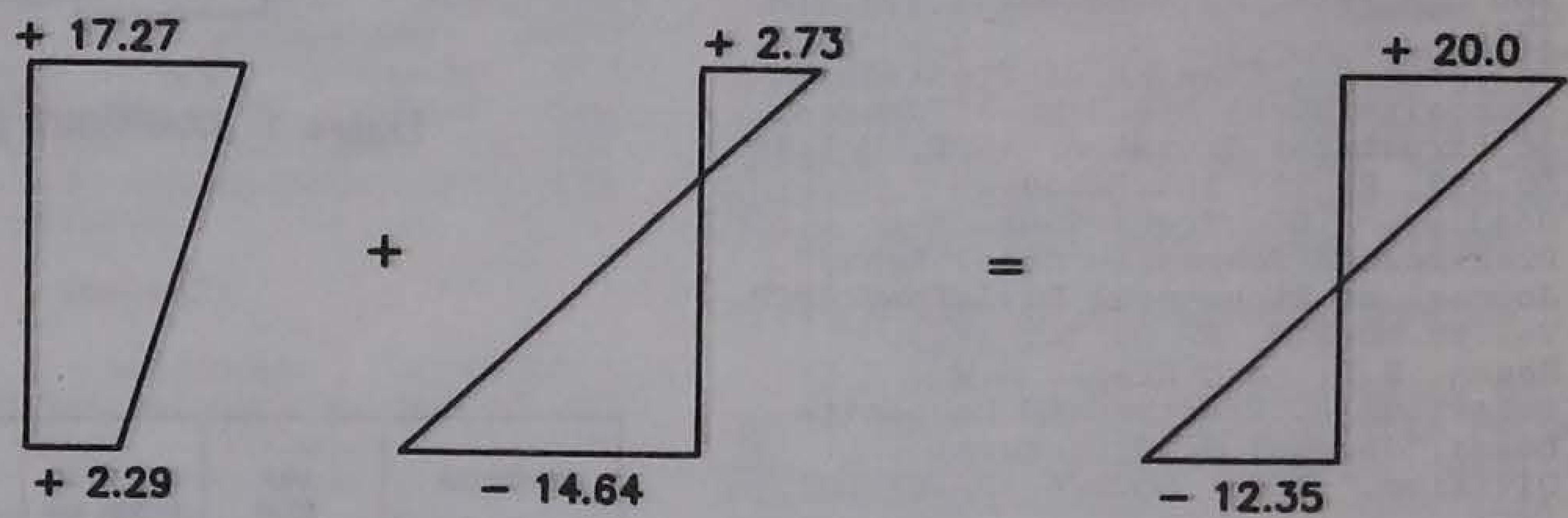

Figure 4 STRESS DIAGRAM (PRESTRESS + D.L + L.L): NO–SHORING

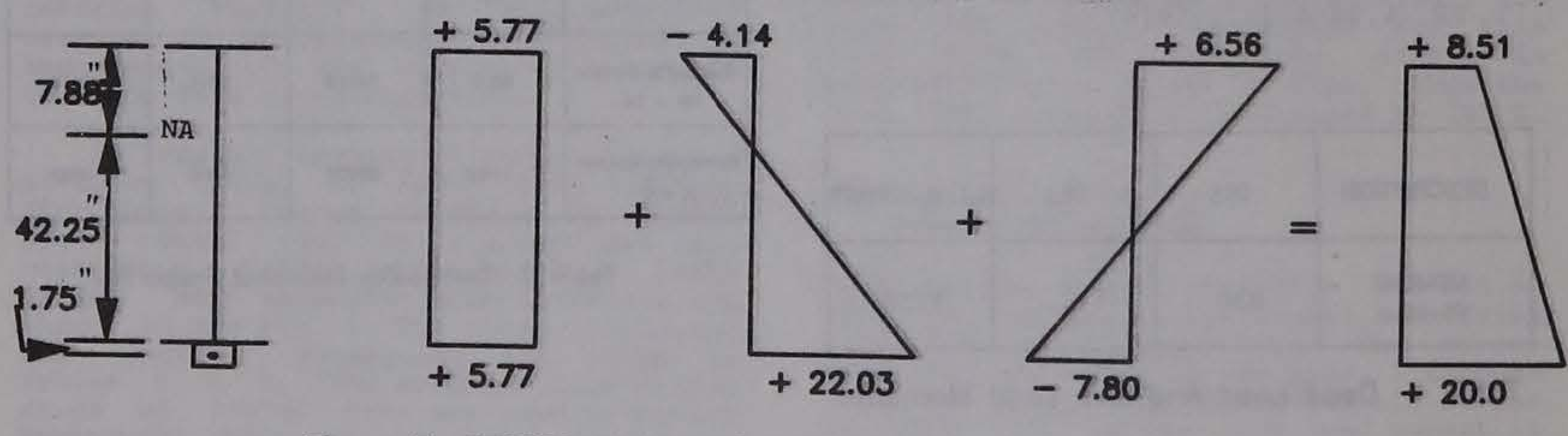

Figure 5 STRESS DIAGRAM (PRESTRESS + D.L): SHORING

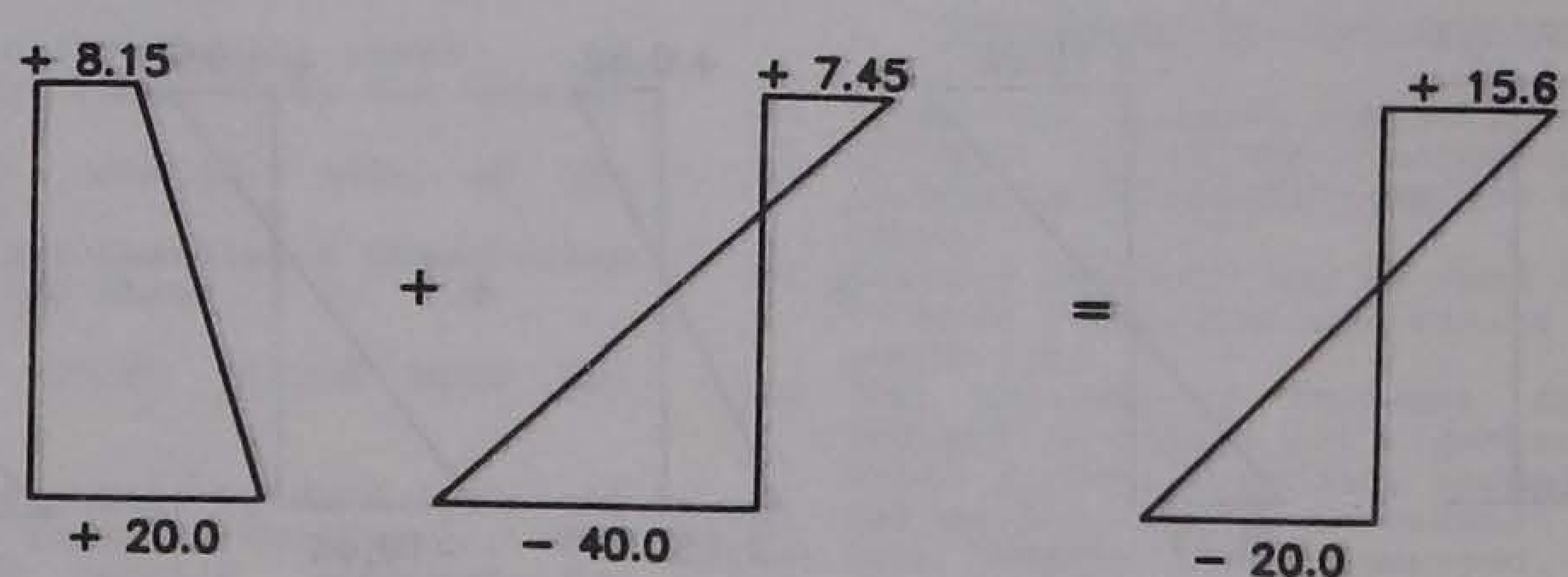

Figure 6 STRESS DIAGRAM (PRESTRESS + D.L + L.L): SHORING

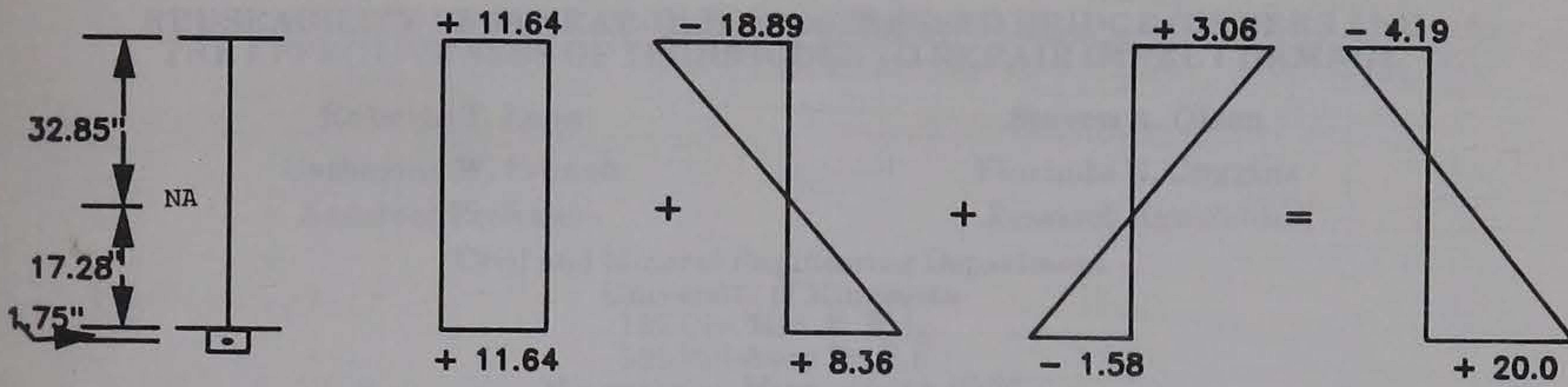

Figure 7 STRESS DIAGRAM (PRESTRESS + D.L OF STRINGER): FIRST STAGE PRESTRESSING

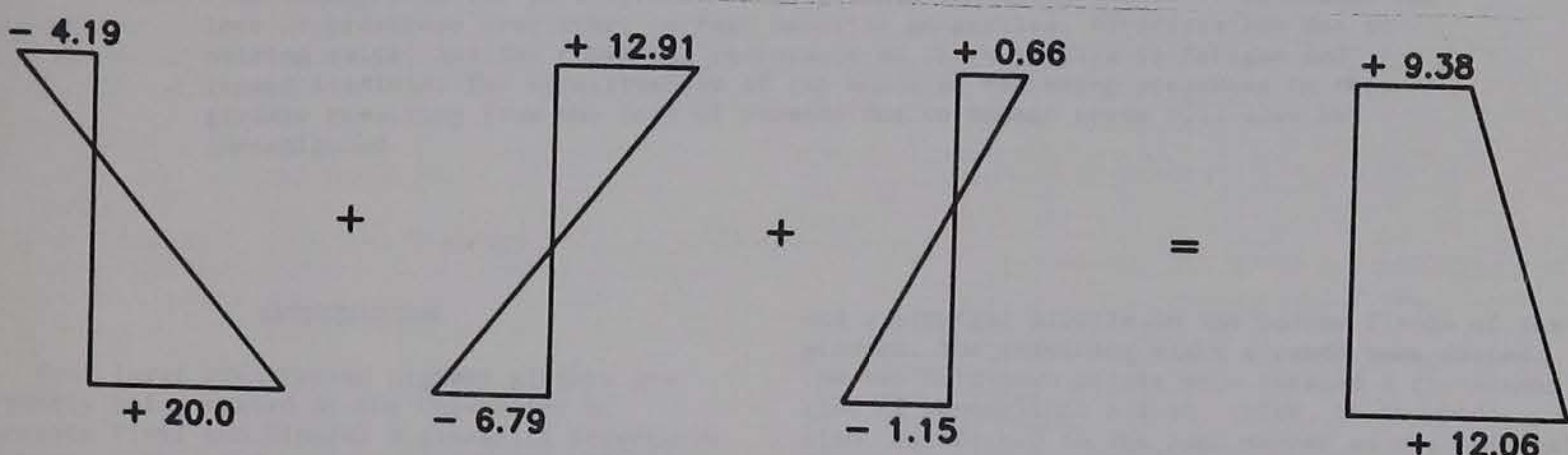

Figure 8 STRESS DIAGRAM (PRESTRESS + D.Ls): FIRST STAGE PRESTRESSING

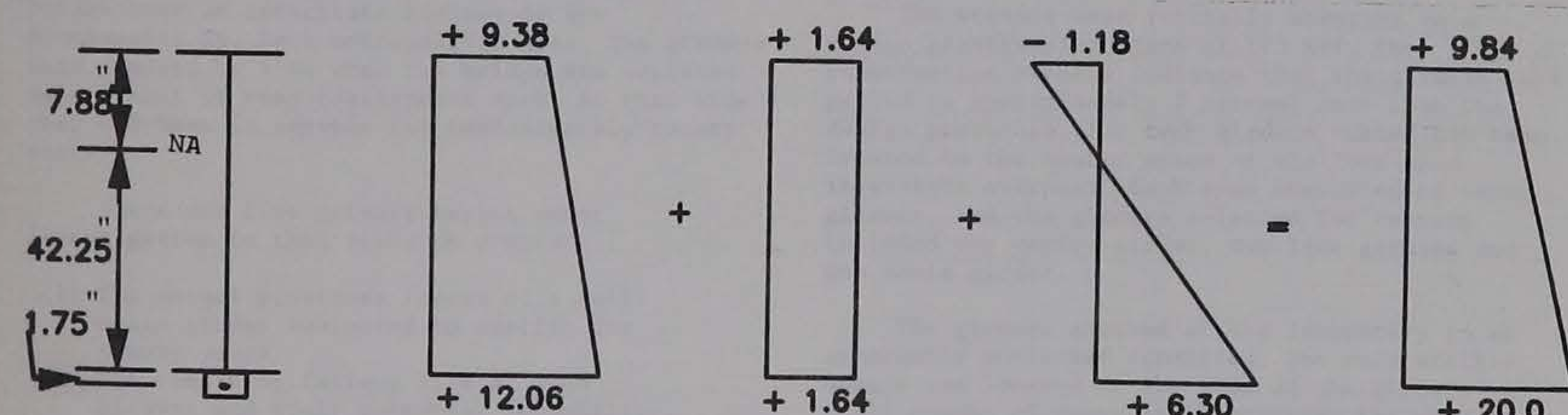

Figure 9 STRESS DIAGRAM (PRESTRESS + D.Ls): SECOND STAGE PRESTRESSING

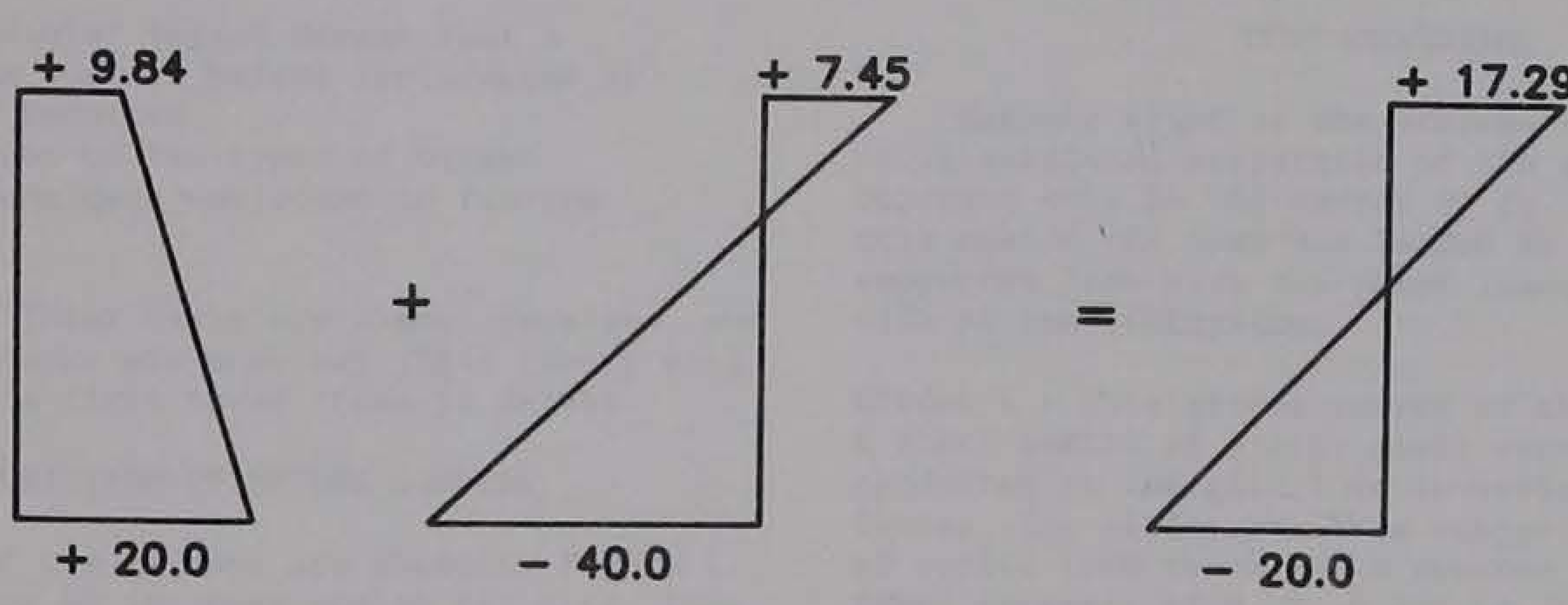

Figure 10 STRESS DIAGRAM (PRESTRESS + D.Ls + L.L): STAGE PRESTRESSING

REUSEABILITY OF 20-YEAR-OLD PRESTRESSED BRIDGE GIRDERS AND THE EFFECTIVENESS OF TECHNIQUES TO REPAIR IMPACT DAMAGE

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SYNOPSIS

Four twenty-year-old prestressed bridge girders are being tested to determine the loss of prestress over time, current material properties, deterioration due to deicing salts, and the remaining resistance of these girders to fatigue and impact loadings. The effectiveness of two means of restoring prestress in the girders resulting from the loss of strands due to impact tests will also be investigated.

INTRODUCTION

Four large prestressed highway girders are currently being tested at the University of Minnesota Civil and Mineral Engineering Structures Laboratory. The girders are standard AASHTO-PCI Type III girders fabricated in July of 1967. They were installed soon afterwards on a county road bridge over an interstate highway in the Minneapolis-St. Paul metropolitan area. The girders were removed in 1986 when the bridge was replaced as a result of road realignment work. At that time they had been in service for approximately twenty years.

There are five primary topics under investigation in this research project:

(1) The actual prestress losses of a full-scale girder subjected to traffic for twenty years.
(2) The remaining fatigue life of such girders and their potential reusability.
(3) The material properties of the girders and components after twenty years of service.
(4) The "allowable" impact damage that a girder can sustain before replacement or repair is required.
(5) The behavior of two types of strand repair techniques subjected to fatigue loading.

The first three tasks are almost complete, and the remaining tasks are underway. This report will address only the first three items in detail.

DESCRIPTION OF BRIDGE GIRDERS

Details of the girders are shown in Figure 1. The girders were 45 in. deep and 64 ft. 8 in. long, prestressed with thirty 1/2 in. diameter 250 ksi stress-relieved strands. Twenty-two of the strands had a straight profile in the bottom flange of the girders. The remaining eight strands were draped; the two hold-down points were located 5 ft. either side of centerline. A 6 in. thick, 64 in. wide slab, reinforced in the same manner as the original bridge slab, was added to each girder in the laboratory. Each girder was tested individually.

The strands were initially stressed to a design prestressing force of 175 ksi. The construction records indicate that the girders were pulled to approximately 2 percent more than the design prestress. The four girders tested had been located in the center spans of the four-span interstate overpass. Each span consisted of seven girders, and the girders selected for testing included one center girder, two lane girders and one facia girder.

The girders arrived at the laboratory in an apparently uncracked condition. The only visible damage was located at the ends of the girders: a small amount of epoxy paint covering the ends of the strands had spalled and rust was evident on the ends of the strands.

TEST PROCEDURE

Because eight of the strands were draped, the cross-sectional properties of the girders were constant only in the center 10 ft. of the span. For this reason the beam was loaded as a simply-supported beam with two point loads 5 ft on either side of the centerline.

Girder 1 - This girder served as the control model. A short series of static tests were initially conducted on the girder to investigate prestress losses. The girder was then subjected to a series of cyclic load tests which created bottom tensile fiber stresses of 0, 3, 6 and 12 $\sqrt{f'_c}$, respectively, at the centerline of the girder. These loadings produced stress ranges in the strand

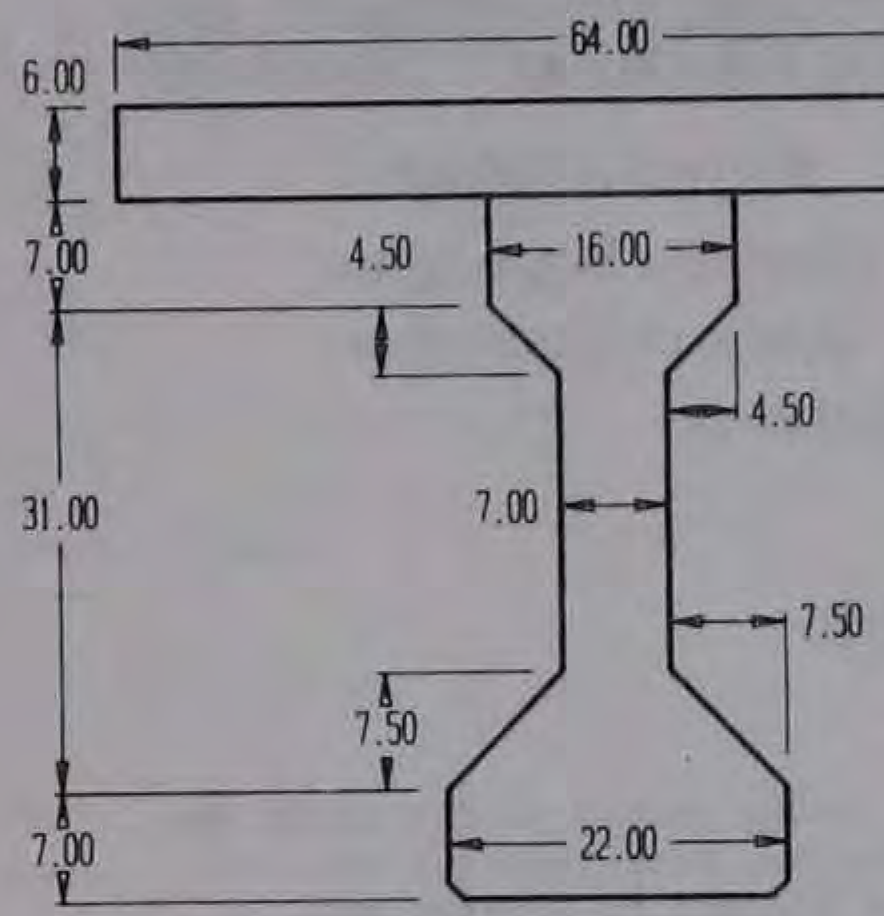

Figure 1 - Cross Section of Girder

corresponding to 8, 10, 13 and 30 ksi, respectively. Static tests were conducted intermittently to monitor the amount of damage accrued during the fatigue tests. After the fatigue tests, the girder was loaded to failure. Nondestructive tests were correlated with core samples to determine the material properties.

Girder 2 - Strands are to be cut in this girder to simulate impact damage which might occur from an oversized vehicle or piece of equipment travelling beneath the bridge. This test is being conducted to determine the "allowable" amount of impact damage which may be sustained before replacement or repair is necessary. The test of this girder was in progress as of July 1988. Prestress losses were first investigated as for Girder 1. The girder was then cycled at a bottom fiber tensile stress of $3\sqrt{f'c}$ for 500,000 cycles and 1,500,000 cycles at $6\sqrt{f'c}$. This was intended to simulate previous fatigue damage to the girder before impact.

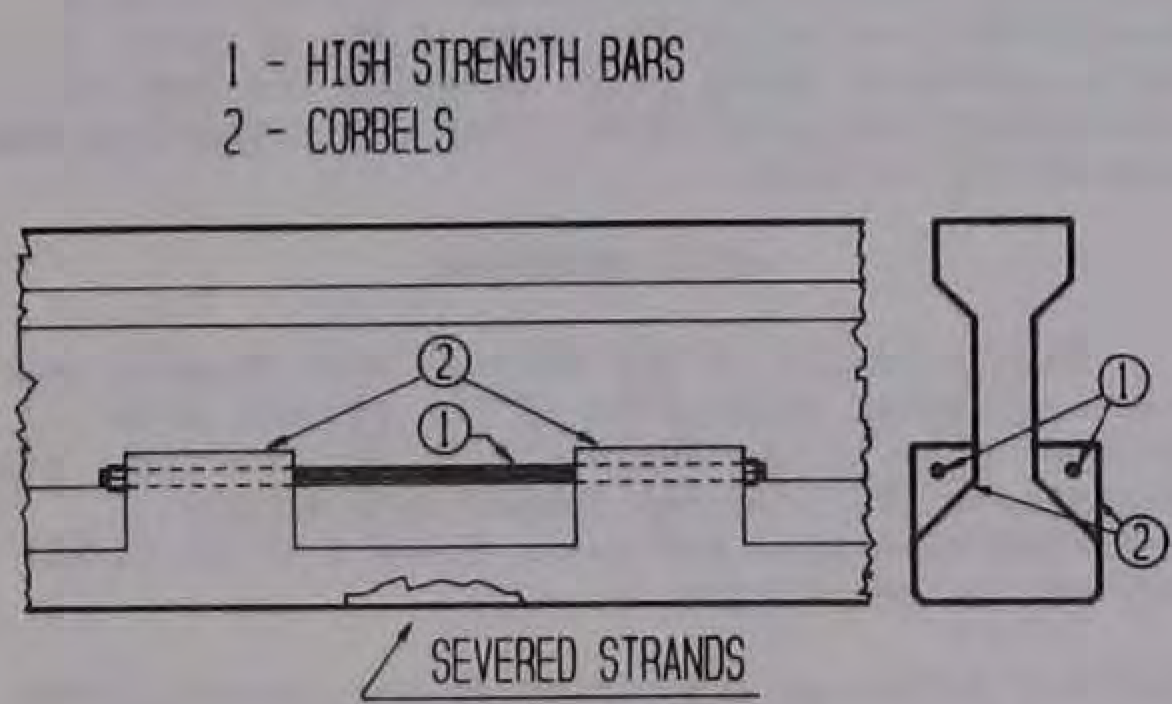

Figure 2 - External Post-tensioning Repair

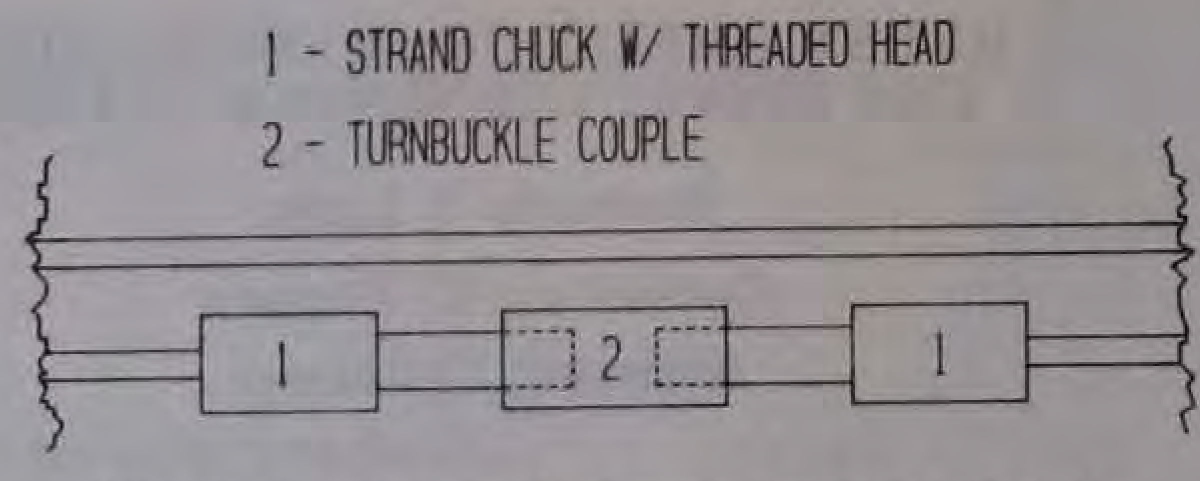

Figure 3 - Internal Strand Coupling Splice

Girders 3 & 4 - These girders will be used to evaluate two techniques to repair impact damage. First, an external post-tensioning system will be used to restore the prestress loss due to impact. Second, a strand splice using a coupling device will be investigated. The two connections have been investigated under static loading in a previous study by NCHRP. Figures 2 and 3 shows details of these devices.

PRESTRESS LOSSES

Four techniques were used to estimate the prestress losses which had occurred in Girders 1 and 2 since the time the girders were fabricated. The procedures are outlined below:

(a) The center 12 ft. of the bottom flange was instrumented with sensitive crack gages to determine the formation of the first crack. The prestressing force could then be calculated by assuming a tensile strength for the concrete or based on core samples taken after the end of the tests.
(b) Upon reloading from the initial cracking cycle, the loading progressed very slowly to determine the crack opening load. For this phase high-resolution LVDTs were used to monitor the strains over a 3 in. gage length across the crack already present.
(c) The load-deflection curve was carefully tracked by the computer-based data acquisition system, and a noticeable change of stiffness was evident as the crack reopened.
(d) After the tests on Girder 1 were completed, an undamaged section near the ends was carefully chipped away to expose the two strands closest to the flange edge. Strain gages were then applied to three of the wires. The gages were monitored to determine the change in strain as the strands were then severed with a torch. This technique will be repeated for Girder 2, except that in addition to the strain gages an extensometer will be used.

While it is difficult to state the initial prestress because the construction documents are the only source available, the in-situ prestress forces as given by methods (a) to (c) were very close to one another, indicating a total prestress of 130 ksi. Method (d) gave somewhat lower values (-15%). This can be explained by slight misalignment of the gages, transfer length and differences in prestsress from strand to strand. The total prestress losses calculated were about 45 ksi, very close to the lump sum prestress losses predicted by the current AASHTO specification.

FATIGUE LOADING

After the initial tests to determine prestress losses, Girder 1 was cycled through a very severe fatigue loading history. The initial cyclic loading, which produced decompression at the centerline of the beam, ranged between a seating load of 8 kips and a peak load of 81 kips (40.5 kips on each actuator). After 500,000 cycles, during which no damage could be observed, the fatigue loading was increased to produce $3\sqrt{f'c}$ nominal tension in the bottom fiber of the cross section (peak load of 94 kips). One million cycles of $3\sqrt{f'c}$ were performed with virtually no damage done to the specimen. The shorter cracks, which had formed during the initial static tests to investigate prestress losses, increased in length during the beginning of this cycling sequence. After approximately 50,000 cycles at this level, the cracks stabilized. No visible change in crack size was evident during the next 950,000 cycles. The loading was then increased to produce a bottom tensile fiber stress of $6\sqrt{f'c}$, the current AASHTO limit. The peak load at this level was 104 kips. Once again, the shorter cracks increased in depth during the initial cycling at the higher loading and then stabilized. After 1.2 million cycles at this level, the load was increased to produce a bottom tensile fiber stress of $12\sqrt{f'c}$ to determine the effect of modest overloads on the behavior of the specimen. Approximately 60,000 cycles were performed at this load level. There was noticeable permanent set observed in the specimen after loading at this level.

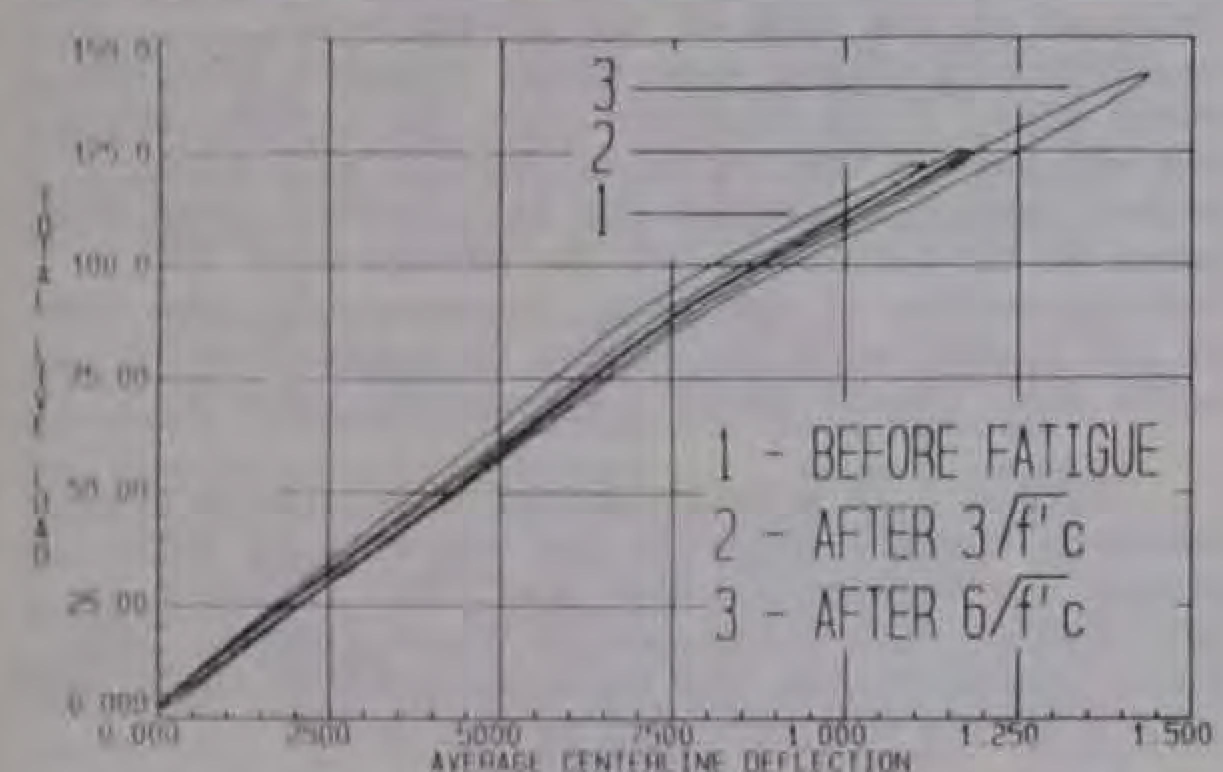

Figure 4 - Load-deflection curves for Girder 1.

Figure 4 shows the load-deflection responses obtained during some of the monotonic load tests conducted intermittently. During the fatigue tests of Girder 2, the structure exhibited only a minor loss of stiffness.

The results of the tests on the first two specimens indicated that fatigue loading which produces strand stress ranges of less than 15 ksi has little or no effect on the ultimate strength and ductility of the member. The size of these girders and their close spacing in the field means that very little damage could have been done by fatigue since the sections were uncracked during their service life. The stress range in service was probably less than 2 ksi.

TEST TO ULTIMATE

After a total of approximately three million cycles, Girder 1 was tested monotonically to failure. The failure, which was initiated by the upward buckling of the longitudinal slab reinforcement, continued to propagate by crushing of the deck and top flange, and concluded with an explosive outward failure of the poorly confined beam web.

The failure occurred at a load of 293 kips and a centerline deflection of 21 in. The load at ultimate constituted approximately 95 percent of the ultimate capacity of the beam based on nominal material properties and the assumption that all steel yielded. By the time failure was reached, the entire 10 ft at the center of the beam had formed a long plastic hinge, and inclined shear cracking had moved out from the constant moment region to the quarter points in the beam. The inclined shear cracks in this area had reached the top flange of the beam, while flexural cracking had progressed to the bottom of the slab. The fatigue loading did not appear to have affected the ultimate strength of the section.

IMPACT TESTS

Impact damage to highway overpasses due to oversize vehicles is quite common and very expensive to repair. Tests on Girder 2 intend to determine what amount of impact damage can be sustained in a large prestressed girder before major repair or replacement is undertaken. Groups of two strands will be cut successively on a bottom corner of the flange to simulate impact damage. Before cutting, the strands will be exposed and instrumented such that static and fatigue tests may be run to estimate the actual stress range in the strands subjected to fatigue. After each pair of strands are cut, the girder will be statically tested and then subjected to cyclic loading at a bottom fiber tensile stress of $6\sqrt{f'c}$. If no serviceability or strength limit is exceeded or strand failure witnessed after 500,000 cycles of fatigue loading, another pair of strands will be cut. This process will continue until strand failure or serviceability criteria dictate that the damage is too severe and that repair or replacement is necessary. At that point the girder will be loaded monotonically to failure.

MATERIAL PROPERTIES

A series of nondestructive tests has been conducted on Girder 1. The tests include a variety of uniformity/strength tests as well as chloride ion penetration investigations. Windsor probe (penetration test), Schmidt hammer (surface hardness), pulse velocity (compressive wave velocity) and breakoff tests (lateral pressure required to detach countersunk cylinder) have been conducted and are currently being correlated with two and four inch diameter cores drilled from the girder.

Concrete and Steel Properties

The nondestructive tests were performed in the web at twelve locations along the length of the girder. The locations were chosen to avoid web reinforcement, draped strands, and cracks which had developed during the testing of the girder.

The primary function of most of the nondestructive tests performed is to give an indication of the uniformity of the concrete. Manufacturer's curves and empirical relationships developed by researchers have attempted to correlate the information obtained from the instrumentation to concrete compressive strength. It is recommended, however, that the information be correlated with actual cores obtained from the specimen under investigation.

The results of the tests are given in Table 1. Compressive strengths were obtained from the pulse velocity measurements, given in parentheses, assuming the compression wave was moving in an infinite medium with lateral restraint and a material constant K = 0.8484 (based on a Poisson's ratio of 0.24). A lower bound Poisson's ratio (0.15) yields compressive strengths on the order of 13,500 psi. Manufacturer's tables were used to calculate the concrete compressive strengths from the other instrument readings.

TABLE 1 - MATERIAL TESTS

TEST	AVERAGE (psi)	MAXIMUM (psi)	MINIMUM (psi)	COV
Schmidt	7960	9800	6120	0.041
Windsor	6500	7880	5800	0.102
Pulse V.	10700	11900	9350	0.051
Breakoff	3660	4690	2090	0.206
4" core	7950	9590	6650	0.118
2" core	10185	12030	8460	0.088

In these tests, the concrete was found to be quite uniform throughout the girder; this is evident from the low coefficient of variation observed with all of the test methods with the exception of the break-off test which also indicated very low concrete compressive strengths. These findings may have resulted from damage done to the concrete during the preparation of the cores for the break-off test. For this method, 2 in. cores are counterbored into the concrete. The lateral pressure required to detach these cores is the break-off reading. Damage could have been done to these cores during the drilling process which would increase the variability and decrease the apparent compressive strengths.

The results from the 4 in. cores are quite variable. Some of the variability may have resulted from a poor capping procedure. The cores which indicated the best failures and stress strain curves yielded compressive strengths between 8000 and 9600 psi. As a result of insufficient test data from the 4 in. cores, additional tests were conducted on the 2 in. cylinders obtained from the break-off tests. The variation in results obtained from the 2 in. cores may be attributed to the small diameter of the core in relation to the aggregate size.

Stress-strain curves are being carried out on the prestressing strands removed from Girder 1 to obtain their current material properties.

Strand Corrosion and Chloride Ion Penetration

The strands which were exposed to check the effective prestress of Girder 1 were located in the flange near the end of the girder in an uncracked region. Evidence of rust appeared on the strand within 4 in. of the end of the girder. Otherwise, the strands appeared to be in excellent condition.

Chloride ion concentrations were determined at several locations in the web and bottom flange of Girder 1 (interior girder). Measurements were taken at 1/2 in. increments to a depth of 2-1/2 in. All of the readings were within the threshold limits of 250-350 ppm by weight of concrete. The highest reading of 270 ppm, obtained at a depth of 2-1/2 in., was suspect because it exceeded the readings obtained at shallower depths. The next highest readings were on the order of 120 ppm. It is interesting to note that the readings obtained from the bottom flange were consistently higher than those obtained from the web, and the readings obtained on one side of the girder were consistently higher than those obtained from the other side. It is expected that the girders which are exposed to the incoming traffic will have higher readings because the salt-concentrated mist tends to be carried under the bridge with the forward motion of the cars. Consequently, the side of the girder with the higher concentration of salts most likely faced the incoming traffic.

Girder 3 is a facia girder which faced the incoming traffic chloride ion contents of this girder are currently under analysis. If there is any problem with corrosion of these girders, the facia girder facing the incoming traffic would most likely be the one affected by the salts.

SUMMARY

The tests carried out so far indicate that the prestressed girders were in excellent condition after twenty years of service in an aggressive environment. There did not appear to be problems associated with corrosion of the girders, most likely because of the excellent concrete quality and the depth of cover. The prestress losses, estimated to be on the order of 45 ksi, correlated well those predicted by the current AASHTO lump-sum method. The fatigue loading imposed, indicated that for loadings up to HS20-44 or Type 3S2 vehicles, the stress range was probably in the infinite life region of the fatigue curves. Thus the girders could be reused in other bridges if they are removed with care and NDT techniques indicate adequate material properties and no evidence of strand corrosion.

STRENGTHENING OF CONCRETE GIRDERS WITH EPOXY-BONDED FIBER-COMPOSITE PLATES

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SYNOPSIS

This paper presents a new method for strengthening of existing bridges. The load carrying capacity of bridge girders can be substantially increased by bonding high-strength fiber-composite plates to the tension flange of the cambered girders. Six upgraded girders will be tested to investigate their flexural strength. Two additional tests will be performed to examine the shear capacity of girders with fiber-composite plates attached to their webs.

INTRODUCTION

A great number of the bridges in the United States are old and rapidly deteriorating. In many case, the reinforcing steel in the concrete girders and deck is corroded, resulting in a reduction of the load-carrying capacity. Even among those structures which are not extensively corroded, many require deck widening and/or strengthening in order to accommodate larger and heavier trucks.

External post-tensioning and the addition of epoxy-bonded steel plates to girders have been used to strengthen bridge superstructures in recent years. External post-tensioning has been applied to both steel and concrete girders to increase their ultimate capacity and to improve their fatigue life by reducing the tension part of the stress range cycle (1,2,3). This method does present some practical difficulties such as providing anchorage for the post-tensioning strands, maintaining the lateral stability of girders during post-tensioning, and protecting the exposed strands against corrosion.

The addition of epoxy-bonded steel plates to the tension face of the girders has been used effectively in Europe, Japan, New Zealand and Russia (4,5,6). The plates can be attached to both steel and concrete girders, resulting in a significant increase in the load carrying capacity of the member. A major shortcoming of this system is the deterioration of the bond between the plate and the girder caused by the corrosion of steel. An effective method to eliminate the corrosion problem would be to replace the steel with a corrosion resistant material such as a fiber-composite. Fiber composites have been successfully used in the aerospace industry for many years. Their superior tensile and fatigue strengths and their resistance to corrosion makes them particularly suitable for use in bridge superstructures.

STRENGTHENING WITH FIBER-COMPOSITE PLATES

This paper presents a new approach for strengthening of existing reinforced or prestressed concrete bridge girders. The technique can be applied to single or continuous-span bridges as well as to other structural systems such as composite steel-concrete bridges.

In this technique, the girders are externally prestressed by epoxy bonding high-strength fiber-composite plates to the tension flange of cambered girders. Adding a moment couple, consisting of the tension force in the composite plate and an equal compression force in the deck, increases the ultimate capacity. At the same time, precompressing the bottom (tension) flange should close the existing cracks and prevent or reduce cracking under service loads.

The steps involved are as follows: First, the girder, in loose contact with an epoxy-coated plate is deflected upward by means of hydraulic jacks as shown in Fig. 1a. In bridges with deteriorated decks, this step can be performed when the old deck is removed and before the new deck is cast to prevent the deck from cracking in tension. When the deck of a bridge is in good condition and does not require replacement, the magnitude of the jacking forces should be limited so as to prevent tension cracking of the deck.

Second, the girder is held in its deflected position until the epoxy is completely cured. The jacks are then removed, as shown in Fig. 1b. The composite plate, placed in tension, prevents a complete elastic return of the girder. This results in initial tension and compression stresses in the top and bottom flanges, respectively. These stresses oppose those caused by gravity loads. A concrete deck is then cast (Fig. 1c). Cambering the girder may result in tension cracks in the top flange. However, these cracks will close under the action of the dead load when the deck is cast.

EXPERIMENTAL STUDY

The static strength of concrete girders externally prestressed with epoxy-bonded fiber-composite plates is currently being investigated at the University of Arizona. Seven 8 in. x 18 in. rectangular beams and one T-beam with a 24 in.-wide flange have been constructed. These 15-feet long beams will be tested under two concentrated load points, symmetrically placed at 12 inches from the mid-span. The design concrete strength for all beams is 4000 psi; Grade 60 steel has been used for all longitudinal and shear reinforcement.

Six tests will be performed to investigate the flexural strength of the upgraded girders (F- series). The variables in these tests include the steel reinforcement ratio, and the initial camber. In addition, two girders will be tested to examine the increase in shear carrying capacity by bonding composite plates to the web of the girder (S-series).

Each F- girder will be strengthened with a 1/4 in. x 6 in. glass-fiber-reinforced composite plate bonded along the full girder length. The composite plates used for this project have linear stress-strain behavior up to failure. According to the manufacturer, the modulus of elasticity of these plates is 7000 psi with a minimum tensile strength of 60 ksi. Coupons of the plates are scheduled to be tested to determine the exact values of their modulus of elasticity and tensile strength. The structural properties of composite plates could be substantially enhanced by using stronger fibers in the plates. However, the manufacturing costs of such fibers are relatively high at this time.

The S- girders will be strengthened using the same type of plates as in F-girders. Each S- girder will be strengthened for shear by bonding 1/4 in. x 3 in. x 18 in. plates to both sides of the web at regular intervals. A summary of specimens to be tested is given in Table 1.

TABLE 1 - Test Specimens

SPEC.	REINFORCEMENT TENS.	COMP.	SHEAR	INITIAL CAMBER
FH1	2#8	2#4	No. 4 @6 in.	No
FH2	2#8	2#4	No. 4 @6 in.	Yes
FL1	2#4	2#4	No. 4 @6 in.	No
FL2	2#4	2#4	No. 4 @6 in.	Yes
FZ	---	2#4	No. 4 @6 in.	No
FT	2#8	3#4	No. 4 @6 in.	Yes
S1	3#8	2#4	---	No
S2	3#8	2#4	---	No

H: High reinforcement ratio
L: Low reinforcement ratio
Z: Zero reinforcement ratio
T: T-beam

The test program is currently in progress and girder FH1 has already been tested. The load vs. deflection curve of this girder is shown with solid lines in Fig. 2. The calculated ultimate loads for the girder without the composite plate and with the composite plate attached are shown with dashed lines in the same figure. The test was stopped after the bond between the concrete girder and the epoxy plate failed at a load of 57 kips. The bond failure was sudden and caused by the brittle nature of the adhesive. Currently, different types of adhesives are being evaluated to select an adhesive with more ductile behavior. This adhesive will be used for the remainder of the tests.

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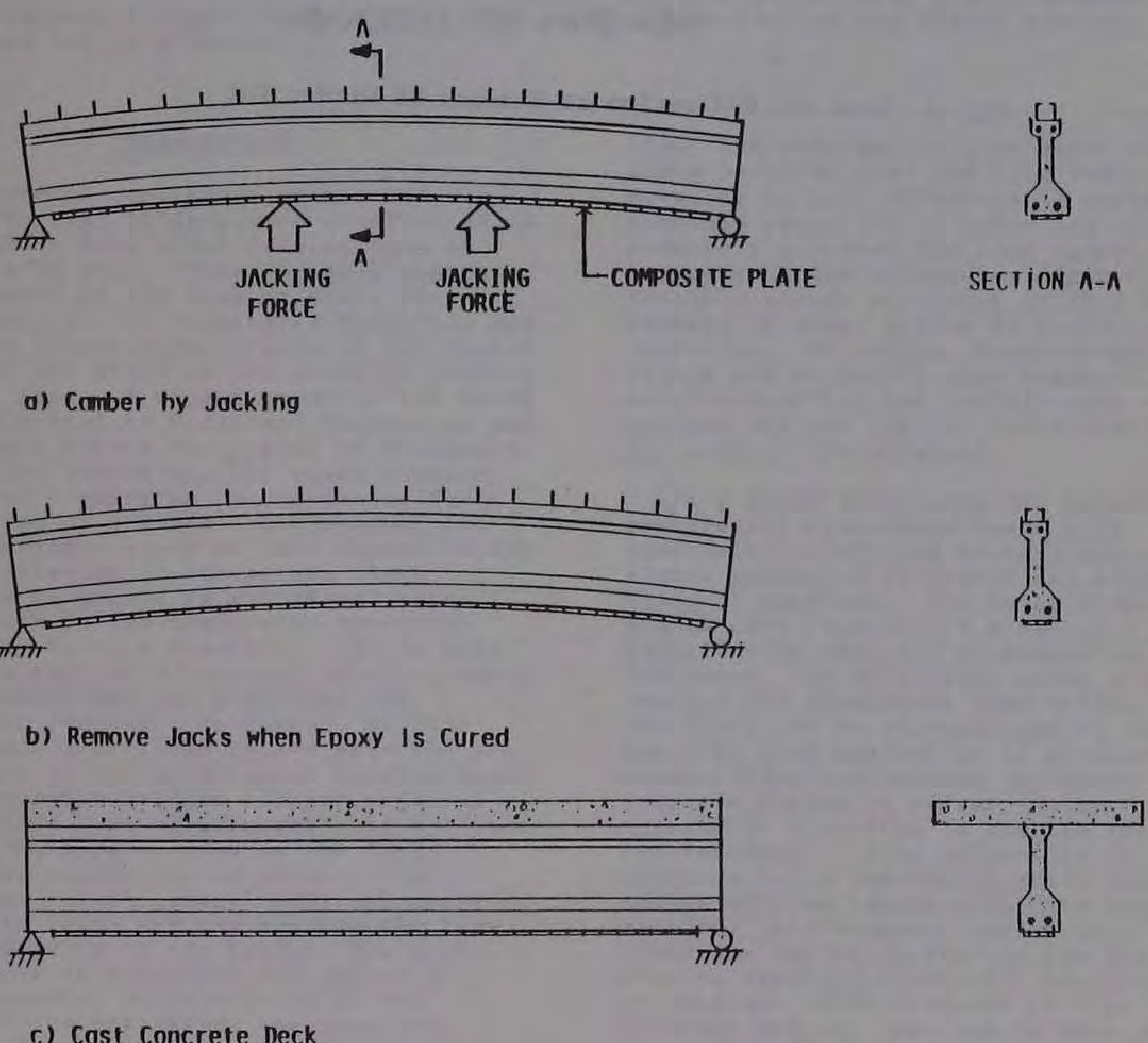

Fig. 1 Prestressing Concrete Girder by Bonding High-Strength Fiber-Composite Plate to Tension Flange of Cambered Beam

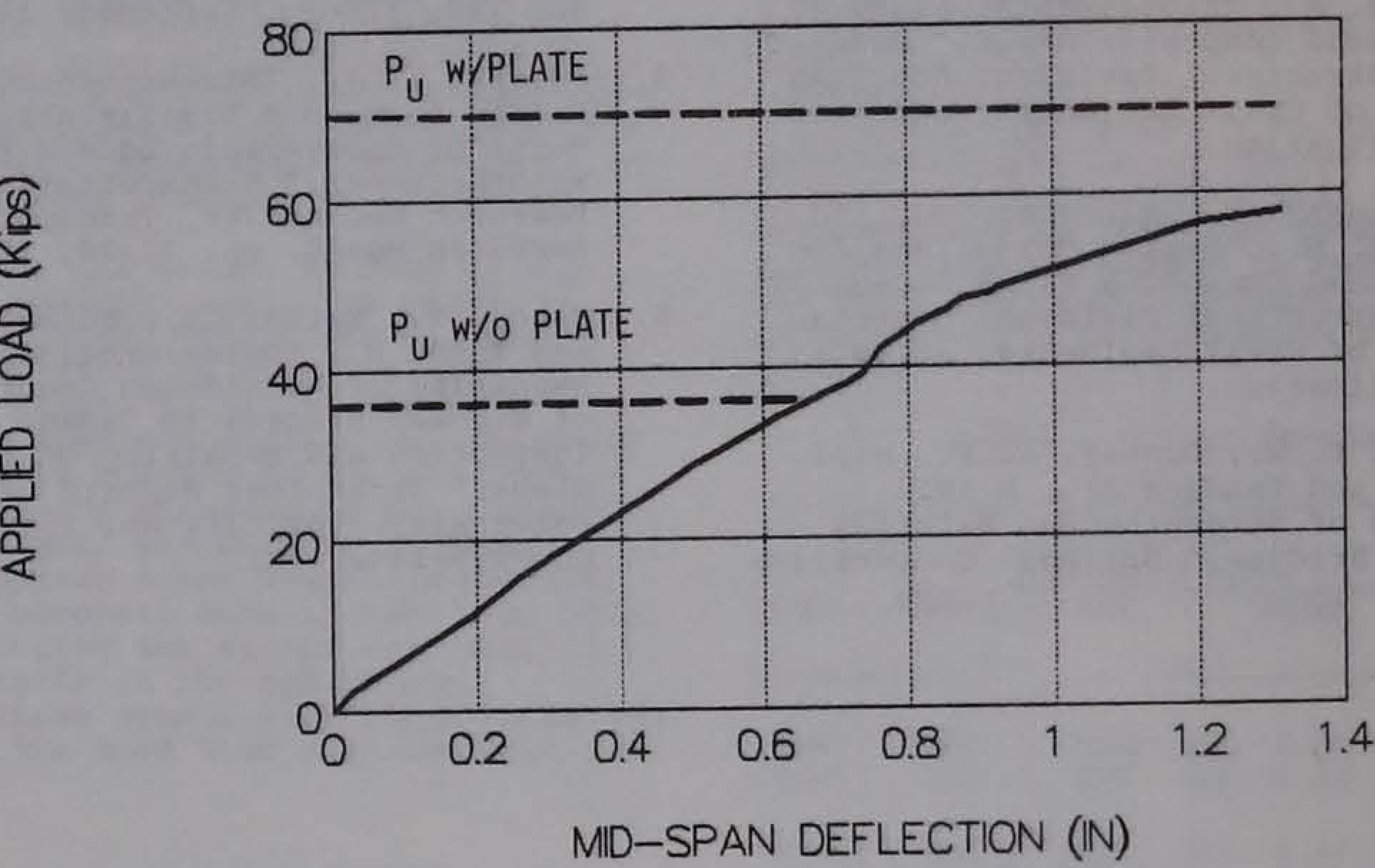

Fig. 2 Load vs. Deflection at Midspan of Girder FH1

STRENGTHENING OF REINFORCED CONCRETE BRIDGES WITH EXTERNAL REINFORCEMENTS

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SYNOPSIS

Several continuous reinforced concrete slab bridges were constructed in the 1950s with spans ranging from 30 ft. to 60 ft. A recent study by South Dakota School of Mines and Technology and South Dakota Department of Transportation has revealed a deficiency in the area of steel reinforcements in the transverse direction to cause longitudinal cracks. Rehabilitation of these bridges was studied by jacking the slab bridges (to provide partial prestress) and bonding advanced composite sheets to the bottom of the slab to retain the prestressing and to provide external reinforcements in the transverse direction. This process has been tested on reinforced concrete beams and will be tried on one-fourth scale model bridge to develop a computer model using finite element method to strengthen bridges in the field. The model bridge testing and finite element analysis are in progress.

INTRODUCTION :-

Several continuous reinforced concrete slab bridges were constructed in the 1950's with spans ranging from 30 feet to 60 feet. These bridges show a deficiency in the area of steel reinforcement in the transverse direction and develop longitudinal cracks on the bottom side of the slabs in the positive bending moment zone. A recent study by the South Dakota School of Mines and Technology and the South Dakota Department of Transportation has determined the exact requirements of transverse reinforcement for this type of bridges. Replacement of these bridges would be very expensive and rehabilitation to strengthen these bridges by providing additional reinforcement in the transverse direction seems to be more practical. It is also obvious that only external reinforcements can be provided for practical and economic reasons. In Europe, studies have been undertaken to evaluate the use of steel plates as external reinforcement to strengthen bridges without prestressing. This type of strengthening will not close the gaps created by the longitudinal cracks, or in effect, the external reinforcements will not carry any dead loads and may enhance the live load capacity of the bridge. The presence of cracks is sometimes the source of environmental deterioration of the reinforcing bar inside the concrete slab. This study presents a method of prestressing the deck so that the cracks can be closed effectively and at the same time, the external reinforcement can share both the dead and live loads. This seems to be more effective in preventing possible corrosion of rebar and considerably increase the load carrying capacity of the structure. Epoxy-Graphite sheets were used in this study instead of steel plates to avoid corrosion. Of course, Epoxy-Graphite plates are expensive when compared to steel plates but the overall cost of the project may not reflect the difference in the cost of the material.

This study deals with the behavior of prestressed precracked beams with externally reinforced Epoxy-Graphite plates subjected to static and cyclic flexural loadings. The Epoxy-Graphite plates were bonded with structural adhesives in addition to mechanical fasteners. If this study shows promising results the continuous slab bridges in the field can be strengthened by jacking the slab from the bottom to produce prestressing and bonding the Epoxy-Graphite plates in strips in the transverse direction to provide external reinforcement. This project is in progress and a one-fourth scale model bridge will be tested with this type of external reinforcement before this technique can be applied in the field. A similar separate study with steel plates as external reinforcements is also in progress and not reported in this paper. The cost effectiveness and practical application of the technique to strengthen this type of bridges will be compared in the future.

TEST PROCEDURE :-

In order to study the effect of externally reinforced beams with prestressing on precracked sections, four beams 6 in. by 6 in. by 40 in. long were cast with two rods of number five 60 grade steel. The reinforcements were placed with their center at one inch from the bottom of the slab. These beams were loaded at the center with two concentrated loads six inches apart on a simply supported span of 36 inches. Electrical strain gages were used to monitor strain in steel reinforcement and on concrete (top of the beam) . All the four beams were subjected to an incremental static load up to 8000 lbs. (87 percent of the ultimate load). All these beams were subsequently subjected to a cyclic flexural load with the same span and the same load position, with the load varying from 1000 to 8000 lbs. (sinusoidal variation) at 8 Hz. Strain gage measurements were monitored at regular intervals using the Micromeasurements 220 system with a Peak Indicator. Photoelastic sheets were attached to the side of the beam to monitor crack propagation during the static and cyclic load tests. After one million cycles of loading, two of the beams were again loaded (incremental static loads) to 8000 lbs. to compare the results and to study the influence of cyclic loading. These two beams were then loaded to failure (incremental static loads). Most of the microcracks were not visible to the naked eye even though crack lines could be seen through the photoelastic sheet after the loading was removed. More cracks were visible both by naked eye and through the photoelastic sheet during the cyclic loading. And it is evident that the cyclic loading induces permanent damage in the beam very close to the cracks. After cyclic loading many cracks did not close even after the removal of the load. The static tests after cyclic loading show larger strain in steel and concrete, indicating the effect of cyclic loading on these reinforced concrete beams. In other words, cyclic loads with the same magnitude as static loads produce more damage in concrete beams. Of course 87 percent of the ultimate load is higher for a practical structure subjected to service loads, but in bridge decks, there is a possibility of reaching that equivalent condition with many years of repeated lower level of load to provide cracks in the tension zone.

Two of the remaining beams were subjected to an eccentric force using threaded rods, plates and nuts to produce compression in the bottom of the beam (where the reinforcements are placed) . The magnitude and eccentricity of the prestressing force were adjusted such that maximum allowable tensile stress was induced at the top. This has partially closed some of the microcracks as observed through the photoelastic sheet. A 6 inch width and 36 inch specimen and 0.03 inch thickness (6-ply unidirectional) Epoxy-graphite laminate was prepared using prepreg supplied by AMOCO*. These Epoxy-Graphite plates were prepared according to the directions of the manufacturer for curing. Threaded steel rods of one half inch diameter were used to provide mechanical fastening between the bottom of the beam and the Epoxy-Graphite in addition to the structural adhesive used for bonding the plate to the beam. A Hysol film adhesive was used in this particular study for bonding the Epoxy-Graphite plate to the beam. After the epoxy graphite plates were installed, the prestressing force was released. Strains were monitored in the reinforcing rod, Epoxy-Graphite sheet and concrete, to determine loss of prestress. These two beams were subjected to static incremental load with the same span and the same load up to 8000 lbs. Again, these beams were subjected to cyclic loading of load varying sinusoidally from 1000 to 8000 lbs. at 8 Hz. At the end of one million cycles, these beams were subjected to static incremental load to failure. The strain readings in the steel reinforcement, concrete surface at the top, and Epoxy-Graphite were monitored.

CONCLUSIONS:-

On the basis of the limited tests conducted on beams externally reinforced with Epoxy-Graphite laminates, the following conclusions were drawn.

1) The technique of externally reinforcing the cracked concrete beams was successful.

2) The beams provided with external reinforcement deflected much less than ordinary reinforced concrete beams. This implies that the stiffness of the beams was increased by Epoxy-Graphite laminates and less cracks were produced in the tension zone.

3) The strains at the compression zone of the beam and at the rebar were considerably reduced by introduction of external reinforcement.

* Amoco Performance Products, Inc., Itasca, Illinois

4) A major effect of introduction of the Epoxy-Graphite laminates was the control of cracks and crack widths. Photoelastic sheets conclusively proved that cracks can be controlled by external reinforcement and prestressing the reinforced concrete members.

5) The externally reinforced concrete beams showed very little effect due to cyclic loading of a million cycles.

6) The structural adhesive together with the steel bolts formed an effective shear connection between the Epoxy-Graphite laminate and the reinforced concrete beam.

The tests conducted are a part of a larger project to upgrade existing reinforced concrete continuous slab bridges by external reinforcement. A finite element model is being developed to represent the above mentioned experimental procedure and behavior. The same finite element technique will be applied to study analytically, the effects of prestressing and external reinforcement on an already existing one-fourth scale model bridge before conducting field tests.

STRENGTHENING TECHNIQUES FOR REINFORCED CONCRETE BRIDGE STRUCTURES

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SYNOPSIS

Increased legal loads and special permit overload requirements on the Nation's highway network requires a reevaluation and strengthening of a large number of bridge superstructures. Two types of strengthening measures for RC bridges are proposed and investigated; one consisting of traditional external tendons and the other of a precast, prestressed bottom soffit panel. Both strengthening measures are evaluated analytically and experimentally by means of a full-scale prototype test specimen.

INTRODUCTION

In California alone 353 bridge structures have been identified in need of strengthening at an estimated cost of 103 million dollars to meet uniform rating criteria for increased legal and overloads. However, strengthening measures are often designed and applied in a rather empirical fashion without detailed knowledge of the actual state of the bridge and understanding of the effect of secondary structural components such as rails, curbs, barriers and parapets, as well as actual support constraints. The large volume of required strengthening projects not only justifies but necessitates a comprehensive evaluation of strengthening measures and with it a validated means of state determination of existing bridge structures. In a first step this can only be accomplished on full-scale test structures under controlled laboratory conditions prior to increasing the complexity with actual in-situ measurements.

As part of an ongoing project with Caltrans on "Structural Concrete Overlays in Bridge Deck Rehabilitation" [1], a 12 ft wide and 60 ft long section of an existing 25 year old cast-in-place concrete T-beam bridge was brought from Fresno, California to the Charles Lee Powell Structural Systems Laboratory at the University of California, San Diego for full-scale testing of a structural concrete overlay (see Figs. 1 and 2). Upon completion of the overlay tests, this fully instrumented, full-scale bridge section presents an unique opportunity to investigate the effectiveness of strengthening measures for existing bridge decks under con-

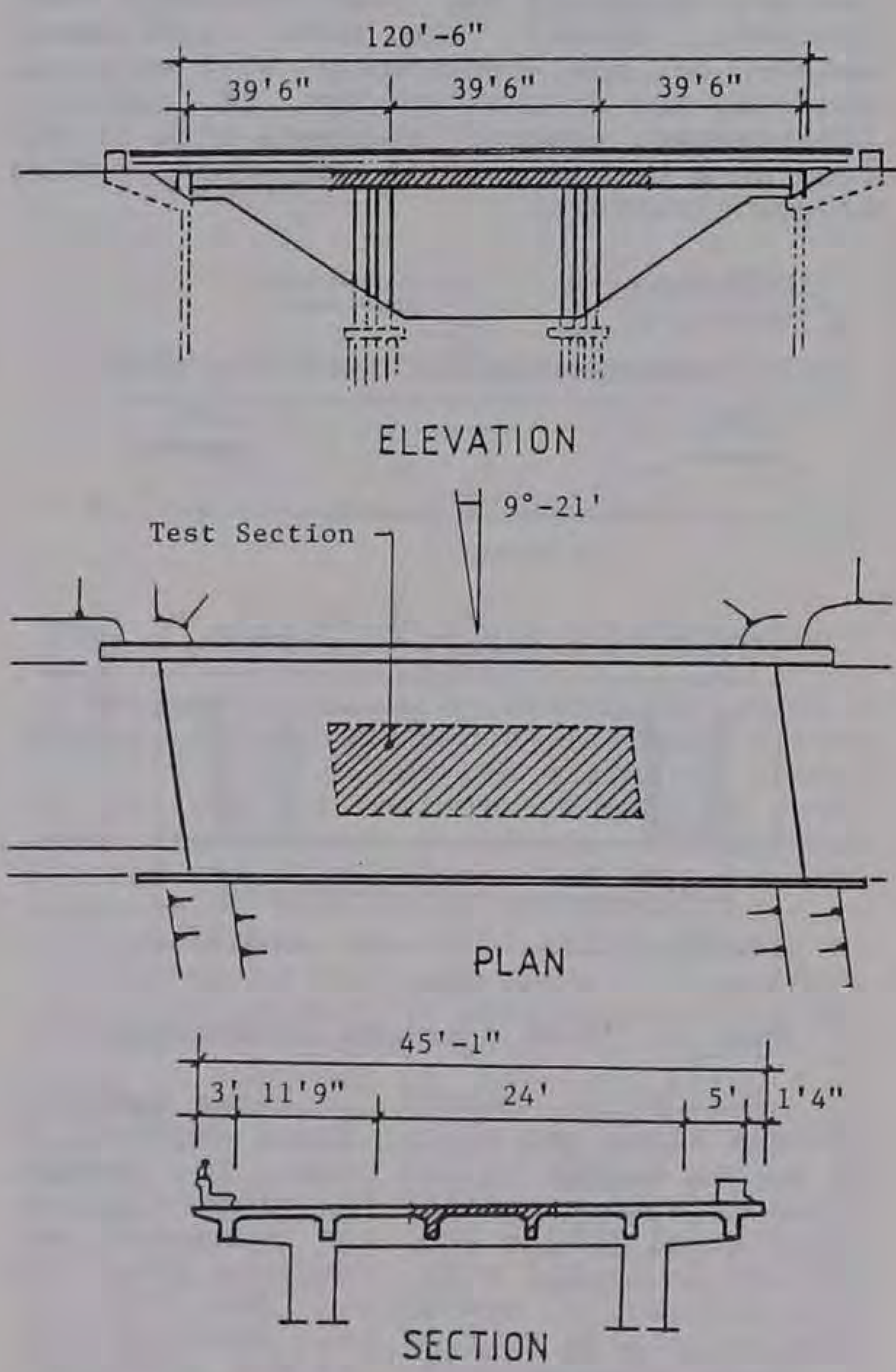

Fig. 1 Gepford Overhead

trolled laboratory conditions.

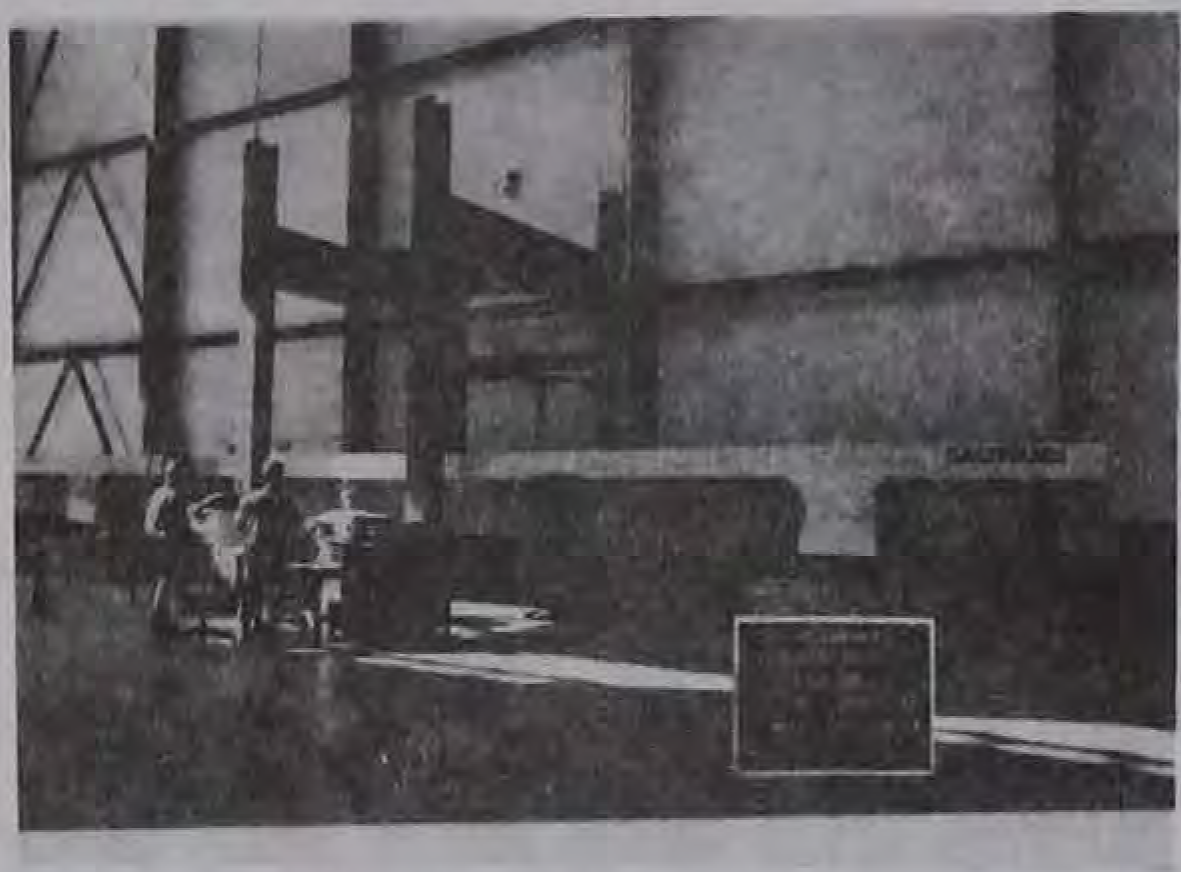

Fig. 2. Test Set-Up

STRENGTHENING MEASURES

The detailed geometry of the instrumented Gepford Overhead test section is shown in Fig. 3. Two strengthening methods are proposed for this prototype test specimen; namely the more traditional method of post-tensioning with external tendons, and a new concept of adding a prestressed, precast concrete slab in the form of a bottom soffit to the existing bridge structure.

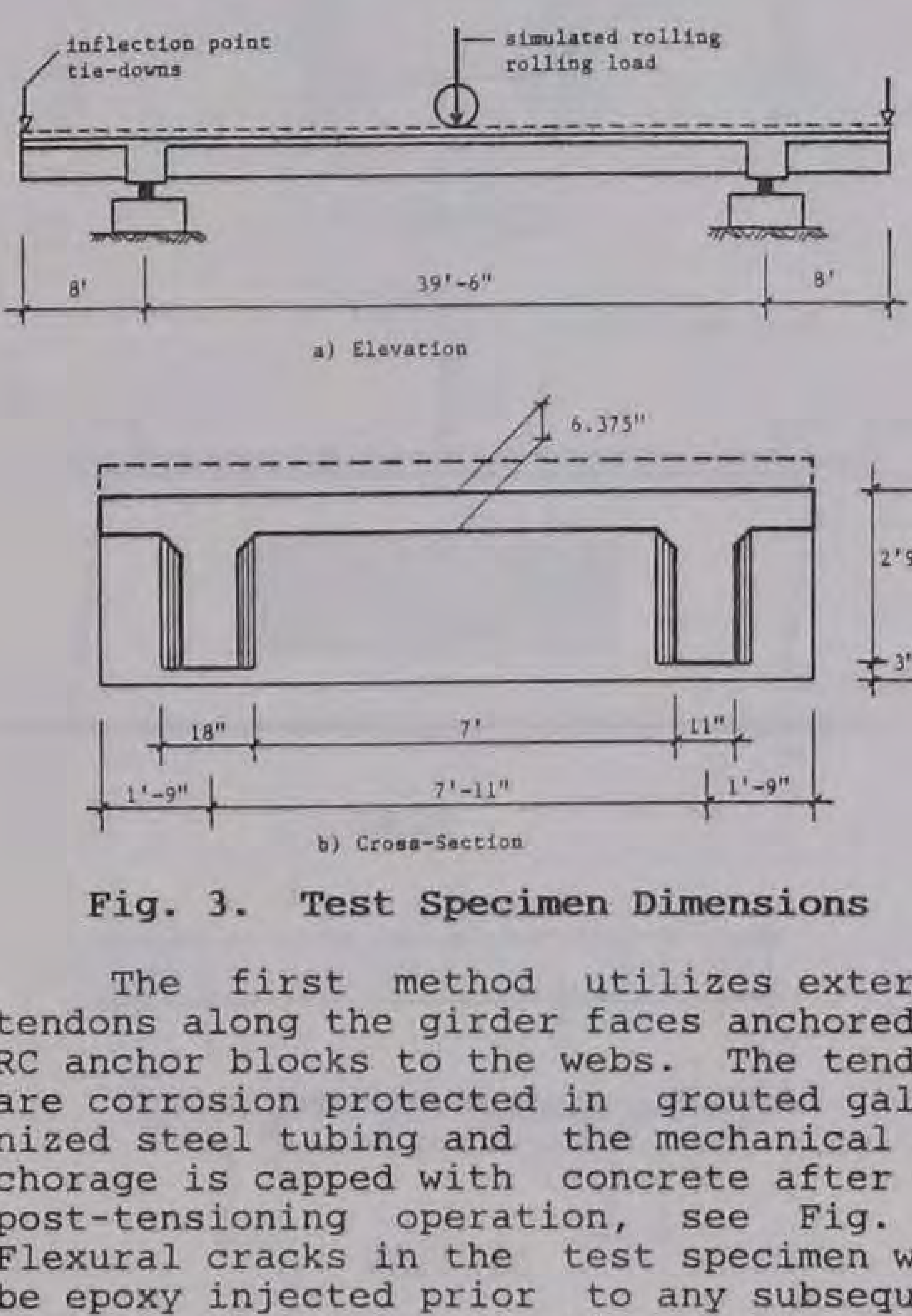

Fig. 3. Test Specimen Dimensions

The first method utilizes external tendons along the girder faces anchored in RC anchor blocks to the webs. The tendons are corrosion protected in grouted galvanized steel tubing and the mechanical anchorage is capped with concrete after the post-tensioning operation, see Fig. 4. Flexural cracks in the test specimen will be epoxy injected prior to any subsequent testing. Special attention needs to be given to the shear transfer behind the anchorage zone where secondary effects due to post-tensioning cause significant positive moments in an area of high shear transfer where additional tensile stress states behind the anchor blocks are introduced in the bridge girder. Detailed analytical modeling of this critical zone between the support diaphragm and the anchor block will be performed to compare the theoretical stress state with experimentally obtained values during the post-tensioning operation and at the flexural midspan yield limit state.

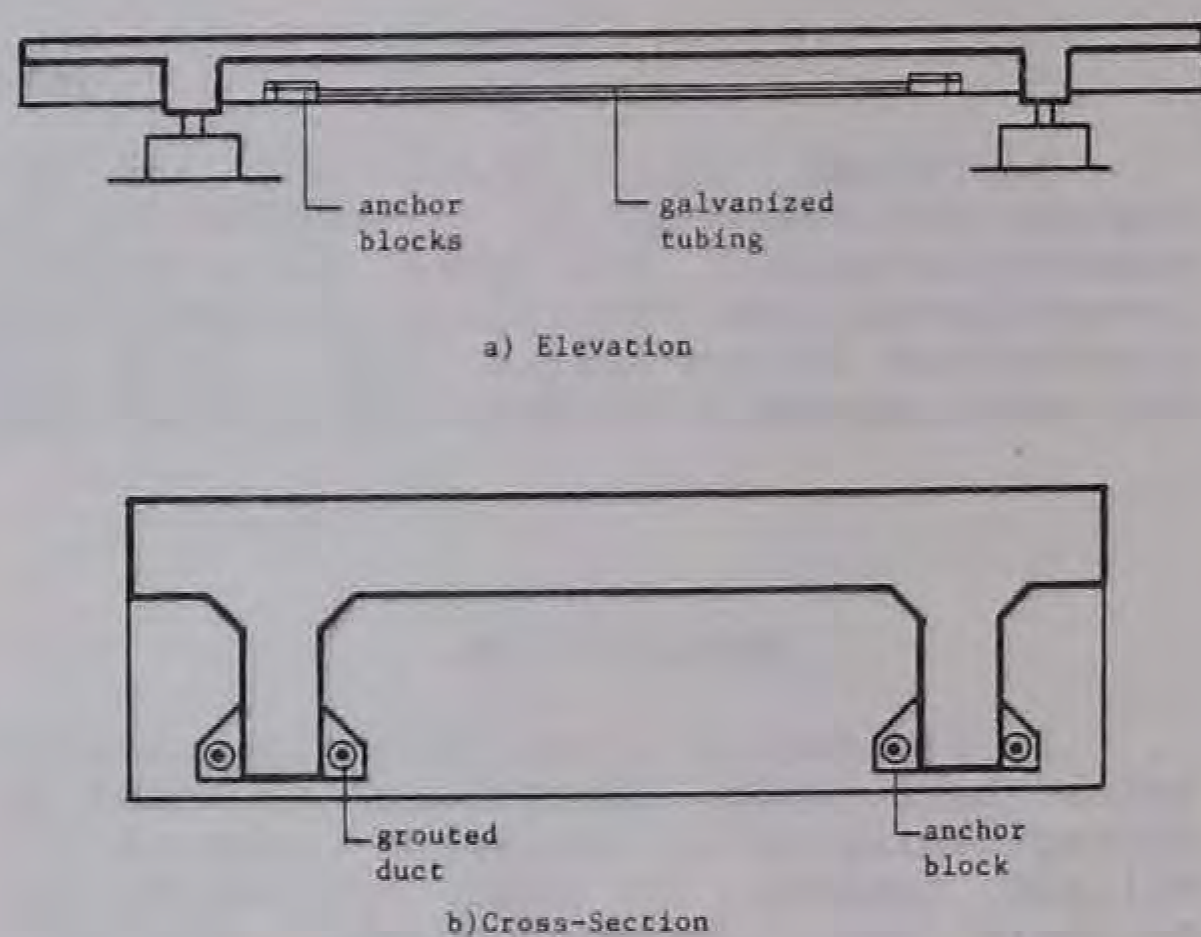

Fig. 4 Strengthening with External Tendons

Strengthening of bridge decks by gluing steel plates to the bottom flange of concrete girders has shown problems with the bond of the epoxy to the steel plates under cyclic loading. In addition to the bond problem the corrosion problem, and with it continued maintenance of the steel plates, still exists. These problems can be eliminated and additional benefits achieved by replacing the steel plates with thin precast highly prestressed, high strength concrete soffit slabs which are positioned with galvanized dowel bolts and epoxy grouted to the girder bottoms (see Fig. 5). The principal advantages of this second method are in the easy manufacturing of the precast and prestressed slabs to required tight tolerances under factory conditions, the transfer of prestress from the soffit slab to the girders due to creep and shrinkage, the improvement in transverse load distribution capabilities due to the formation of a box section, as well as the aesthetic appearance of a closed bottom soffit. The good bonding characteristics between the concrete and the epoxy mortar will ensure non-slip monolithic behavior and the corrosion protection maintenance problem is eliminated.

Testing of the strengthened bridge section will be carried out in both cases to the onset of yield in the main flexural reinforcement to establish yield limit state performance characteristics over and above the service load limit state behavior.

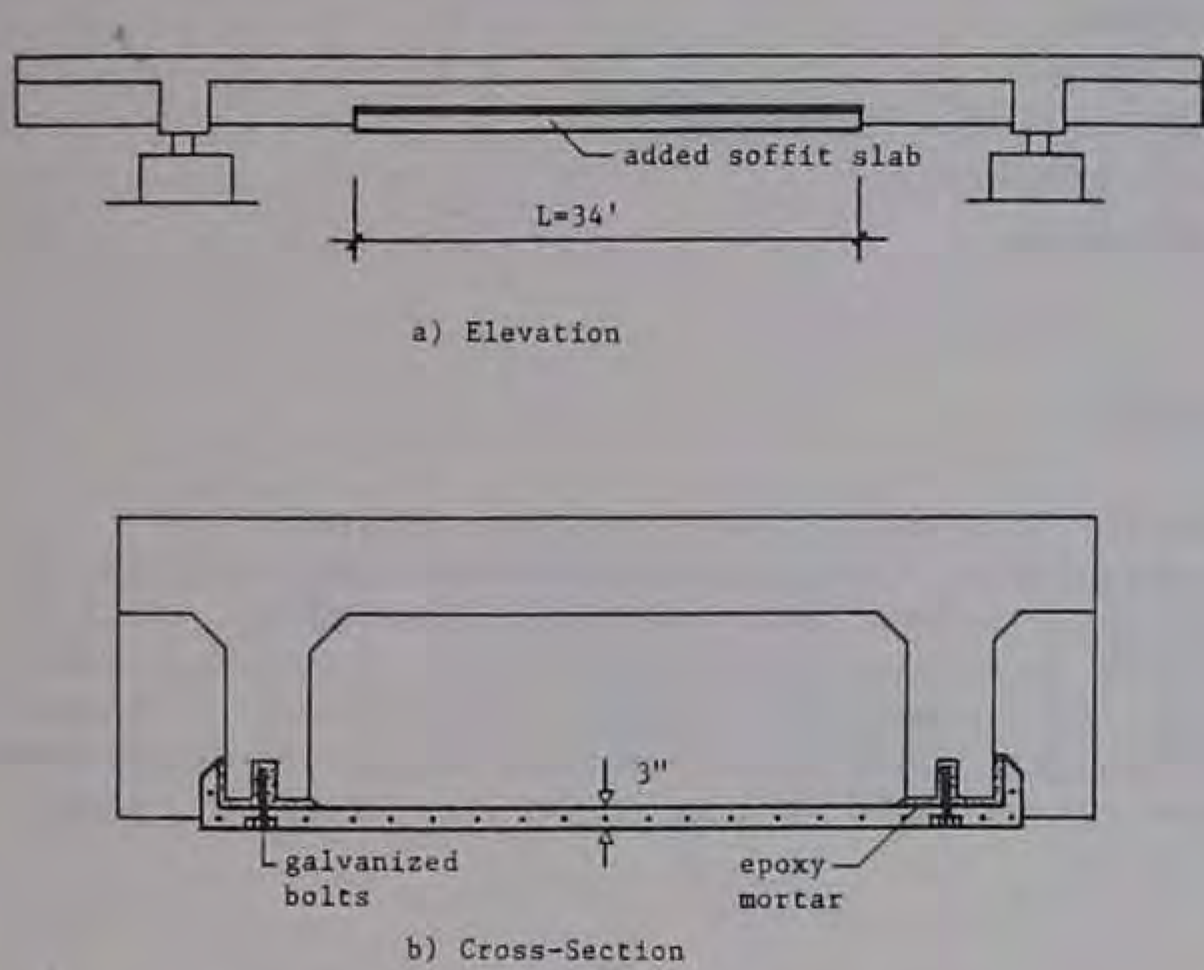

Fig. 5 Strengthening with Precast Soffit Slab

ROLLING TRAFFIC LOAD SIMULATION

The advantages of testing full or large scale test specimens under controlled laboratory conditions are often lost due to simplifications and approximations necessary in the load application. Rolling traffic loads and their effect on the structural behavior cannot be modeled realistically by a push or pull type point load used in most laboratory experiments.

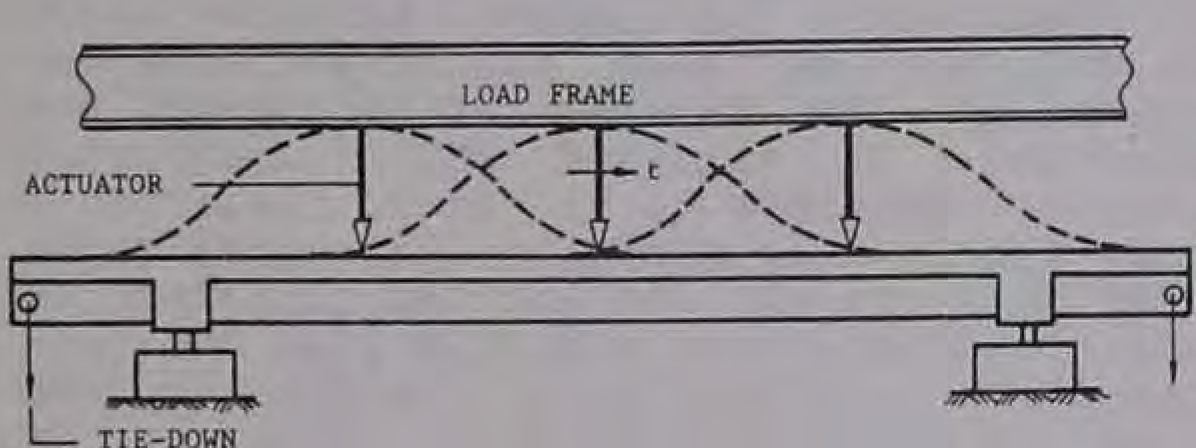

Fig. 6 Moving Load Simulation

A servo controlled multi-actuator system allows the modeling of rolling loads if a series of actuators is computer controlled in a time sequence as schematically outlined in Fig. 6. This type of load application allows the realistic representation of combined and changing moment and shear force states encountered in the prototype under rolling traffic loads.

STATE DETERMINATION

The assessment of the effectiveness of strengthening measures depends largely on the availability of analytical tools to model the actual state of the bridge structure and the changes introduced by the rehabilitation measure. This basic problem is demonstrated in Fig. 7, where a sophisticated nonlinear analytical model of the Gepford Overhead test section is compared with the experimentally measured midspan load-deflection behavior.

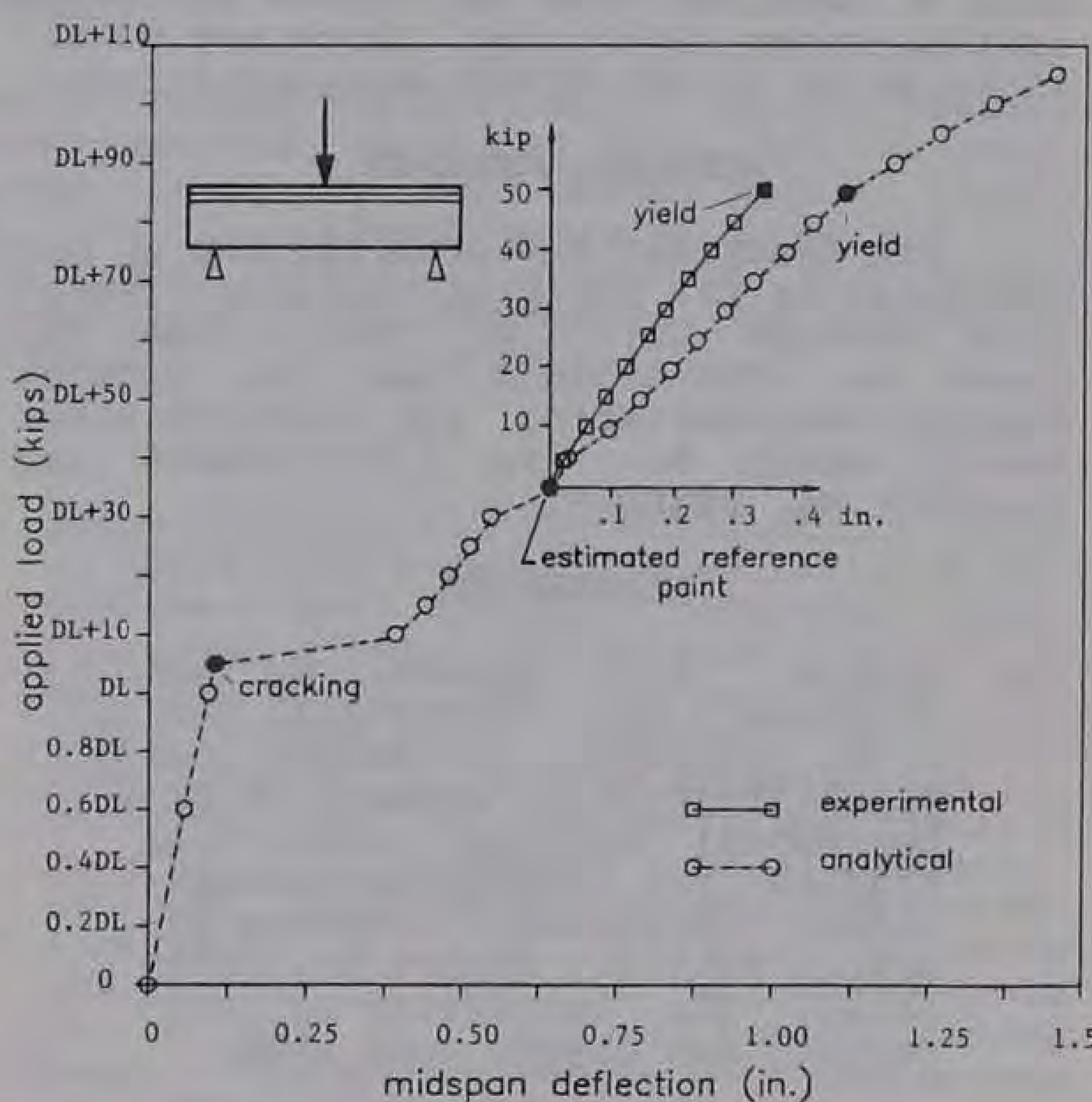

Fig. 7 Analytical State Determination

Not only is the determination of a common reference point (current bridge state) a problem but the different slopes in the load-deflection curve also indicate significant stiffness differences between the analytical and experimental model which have to be explained prior to any assessment of the effectiveness of strengthening measures. It is therefore important to develop analytical state determination models which are partially based on observed or measured field data such as crack patterns, crack widths and the dynamic response behavior due to forced vibration tests. Since forced or ambient vibration measurements are typically the easiest way to obtain in-situ bridge response data, a series of forced vibration tests on the Gepford Overhead specimen will be employed to establish input data under controlled laboratory conditions for input into state determination models.

CONCLUSION

For the large number of bridge structures in need of strengthening to meet uniform rating criteria, proven rehabilitation methods are needed to ensure the required performance. To properly assess the effectiveness and consequences of strengthening measures a state determination of the actual bridge structure is required. This state determination requires the combination of field observations and measurements with state-of-the-art analytical tools. Both the validation of strengthening measures and the verification of state determination models can only be obtained from full scale prototype testing under controlled laboratory conditions prior to any field implementation.

ACKNOWLEDGEMENTS

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STRUCTURAL CONCRETE OVERLAYS IN BRIDGE DECK REHABILITATION

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SYNOPSIS

Full depth structural concrete overlays in bridge rehabilitation require special interface and construction joint preparations between the old bridge deck and the overlay in order to ensure monolithic action under increased legal loads, special permit overloads and long-term environmental effects. The (in-)effectiveness of interface dowels for horizontal shear transfer in connection with "specially roughened" and "clean" contact surfaces is investigated and design recommendations concerning the horizontal shear transfer in overlaid bridge decks are presented.

INTRODUCTION

Increased legal loads and the demand for higher permissible overloads necessitate a reevaluation of the performance and capacities of existing bridge structures. Rehabilitation of bridge decks damaged by continuous wear, as well as by time-dependent and environmental effects is a major concern for a large number of short and medium span bridge structures.

In many cases of bridge rehabilitation the existing concrete deck exhibits damage from general wear, chain beating, deicing salts, freeze-thaw cycles, temperature cycles, etc. in the form of surface cracks, deck delamination and/or scaling, which requires repair or replacement of the existing bridge deck in addition to the strengthening measures of the bridge structure. The benefits of increased structural depth of an additional (overlaid) structural concrete slab placed very economically on top of the existing deck slab can only be taken into account in the design process if monolithic structural action up to critical structural limit states can be assured. In conjunction with the appropriate strengthening measures in the longitudinal superstructure, such as external post-tensioning (where needed), and in the substructures, such as possible pier walls to account for increased gravity and seismic loads, these full depth structural (reinforced) concrete overlays can contribute significantly to increased traffic and overload capacities.

The critical design question in this rehabilitation measure is the horizontal shear transfer in the construction joint or interlayer between the existing "old" and the "new" overlaid deck slab. Questions concerning the surface preparation of the "old" deck slab and the amount, effectiveness and necessity of shear reinforcement or dowels crossing the interlayer led to the described research program. In completed overlay projects, the cost associated with the dowel reinforcement (drilling, placing, grouting and materials) amounted to approximately 15 to 20% of the total rehabilitation cost. The effectiveness of these dowels in conjunction with different surface preparations of the interlayer is the subject of the described research project with Caltrans. The objective of the overlay research project is the development of comprehensive design guidelines to ensure monolithic bridge deck behavior.

EXPERIMENTAL OVERLAY STUDIES

The performance of overlaid bridge decks with different types of surface preparation was determined in a three phase experimental test program. The objective of the test program was threefold: first, to establish distinct performance differences for different surface preparations; second, to develop constitutive information for the interlayer slip behavior for the nonlinear analytical models; and third, to verify proposed design recommendations derived from the analytical studies and the experimental phase I and II (shear block and slab panel) results by means of a full scale prototype test. Surface preparations of the "old" concrete surface creasing load levels in the corner region and both delamination regions grew without a change in the load-deflection behavior. Only when the two delamination zones met the slab changed the flexural behavior from "monolithic" to "lubricated." Thus, due to the in-plane stiffness of both the "old" deck and the overlay, relative interlayer slip in the construction joint can only occur when large regions of delamination grow together. Since a complete delamination is very unlikely in a bridge deck under traffic loads, relative interlayer slip in the construction joint will not occur which renders dowel reinforcement ineffective.

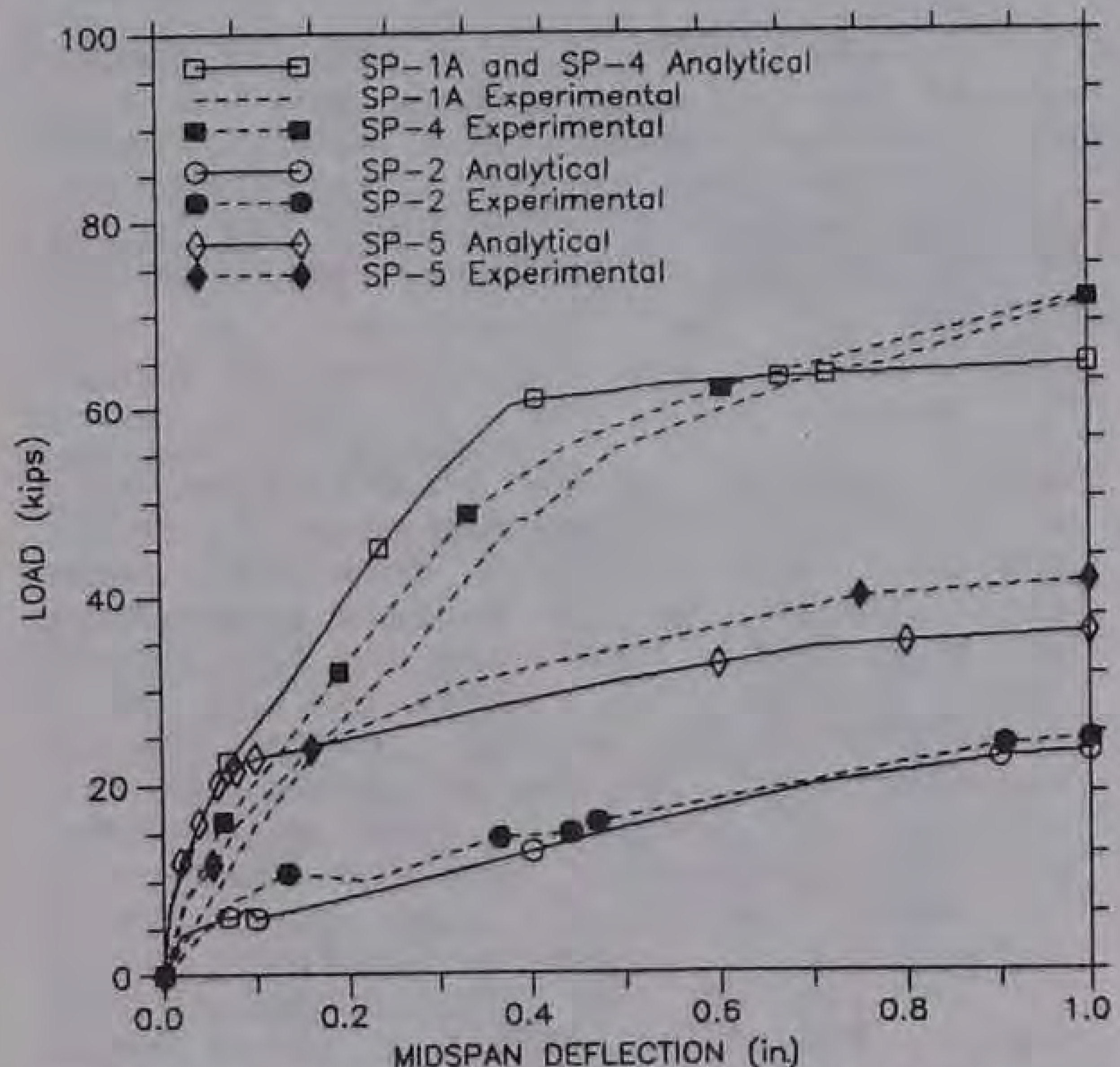

Fig. 6 Analytical Model Validations

The nonlinear interlayer slip model for reinforced concrete was then combined with a layered flexural concrete and smeared steel element and applied to model the analytical response of the simply supported slab panel tests under increasing midspan point loads all the way to the ultimate load limit state. A comparison of analytical results and experimental measurements for three representative slab panels SP-2, SP-4 and SP-5 is given in Fig. 6. The lubricated slab specimen SP-2 which was modeled with zero interlayer stiffness shows excellent agreement in the load-deflection behavior which indicates that the assumption of neglecting any interlayer shear transfer in the analytical model is justified. The behavior is clearly that of two independent slab panels in flexure. The addition of 10 #4 dowels in SP-5, introduced in the analytical model with their equivalent interlayer stiffness v_d, shows reasonable agreement with the experimental values even though the ultimate load level and the pre-yield behavior are slightly different. A similar difference can be observed for the specimens behaving monolithically, represented in Fig. 6 by SP-4 and SP-1. However, the overall correspondence between the theory and experiment in the load-deflection behavior, Fig. 6, and in the crack pattern, Fig. 4, is close enough for validation of the analytical modeling approach.

CONCLUSIONS AND DESIGN RECOMMENDATIONS

Since, based on the above results, only large areas of delamination will cause interlayer slip and with it deviations from the flexural monolithic behavior and since for "rough" and "clean" interfaces slip does not occur unless a nominal horizontal interface shear stress value of at least $2.0\sqrt{f_c'}$ is present, it is proposed that interface dowel reinforcement for full shear transfer is only provided if factored horizontal shear stress values exceed this nominal value of $2.0\sqrt{f_c'}$.

Unless a detailed horizontal shear analysis is performed, a simple capacity design check is recommended based on the flexural collapse limit state load in a critical position for shear. In most design cases the positive (M_n^+) and the negative (M_n^-) section moment capacities at midspan (L/2) and at the supports are readily available, both in the transverse and in the longitudinal direction. The required ultimate shear capacity V_u to ensure monolithic behavior up to the flexural design capacity can be determined as

$$V_u = \Phi_v \Phi_f (M_n^+ - M_n^-)/(L/2) \qquad (1)$$

where Φ_f is the flexural capacity reduction factor (0.9) and Φ_v a shear force enhancement factor which shifts the critical flexural collapse load to a critical shear force position close to the support region but limited by a distance 4 × structural depth due to arching action.

$$\Phi_v = 2 - 8h/L \geq 1.0 \qquad (2)$$

If the so determined horizontal shear stress does not exceed $2.0\sqrt{f_c'}$ for intentionally roughened contact areas no additional shear design is required. In all other areas full shear transfer by interface dowels is proposed.

ACKNOWLEDGEMENTS

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continuous slab panel tests showed the same behavioral characteristics described above.

These test results strongly suggest that the provided dowel reinforcement does not influence the crack pattern development and thus the flexural behavior of the slab panels, since a "rough" and "clean" construction joint provides for a monolithic flexural behavior. Also the ultimate delamination behavior cannot be controlled by the provided minimum dowel reinforcement and significantly higher reinforcement ratios would be required to influence the ultimate capacity.

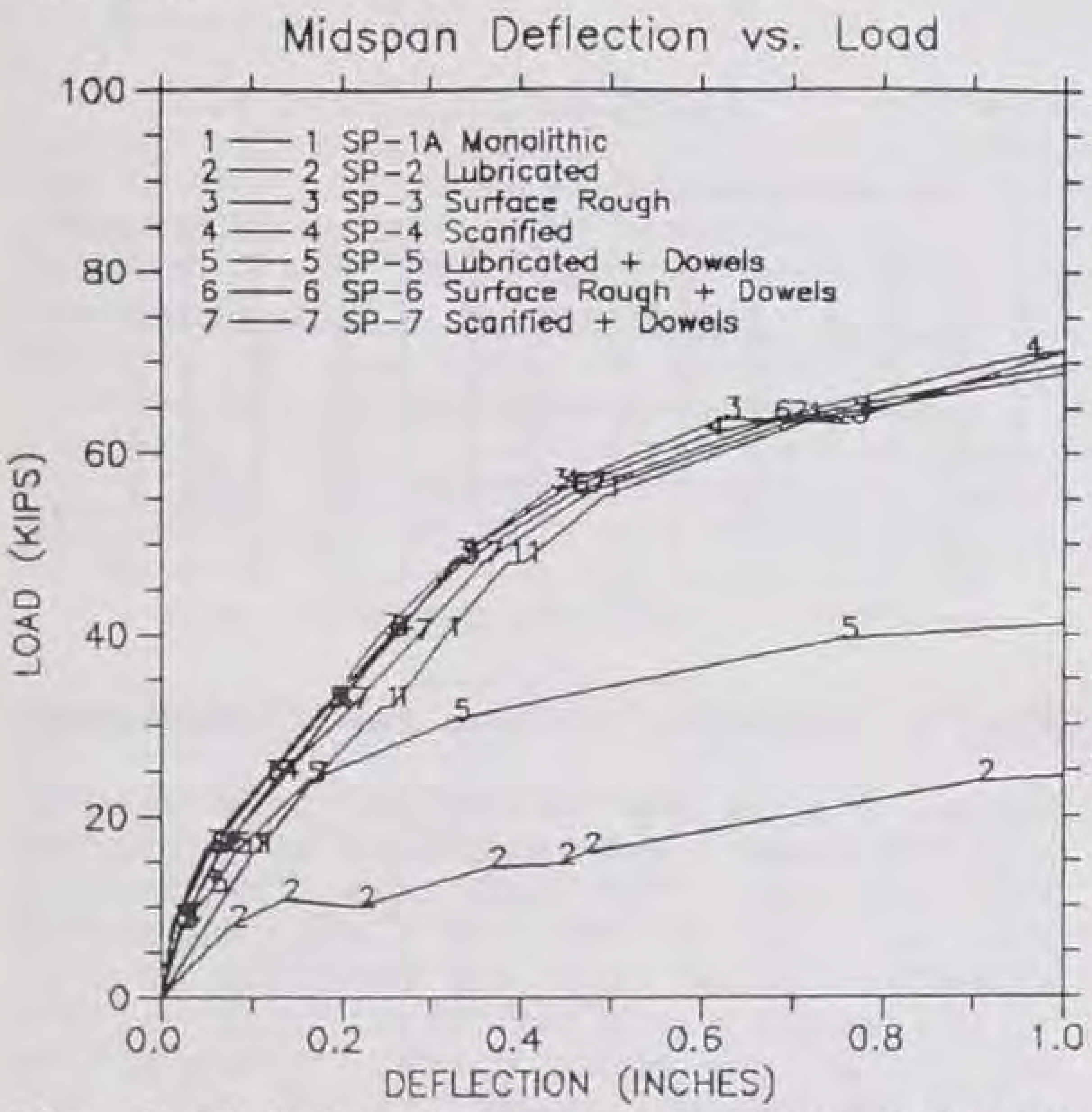

Fig. 3 Slab Panel Load-Deflection Behavior

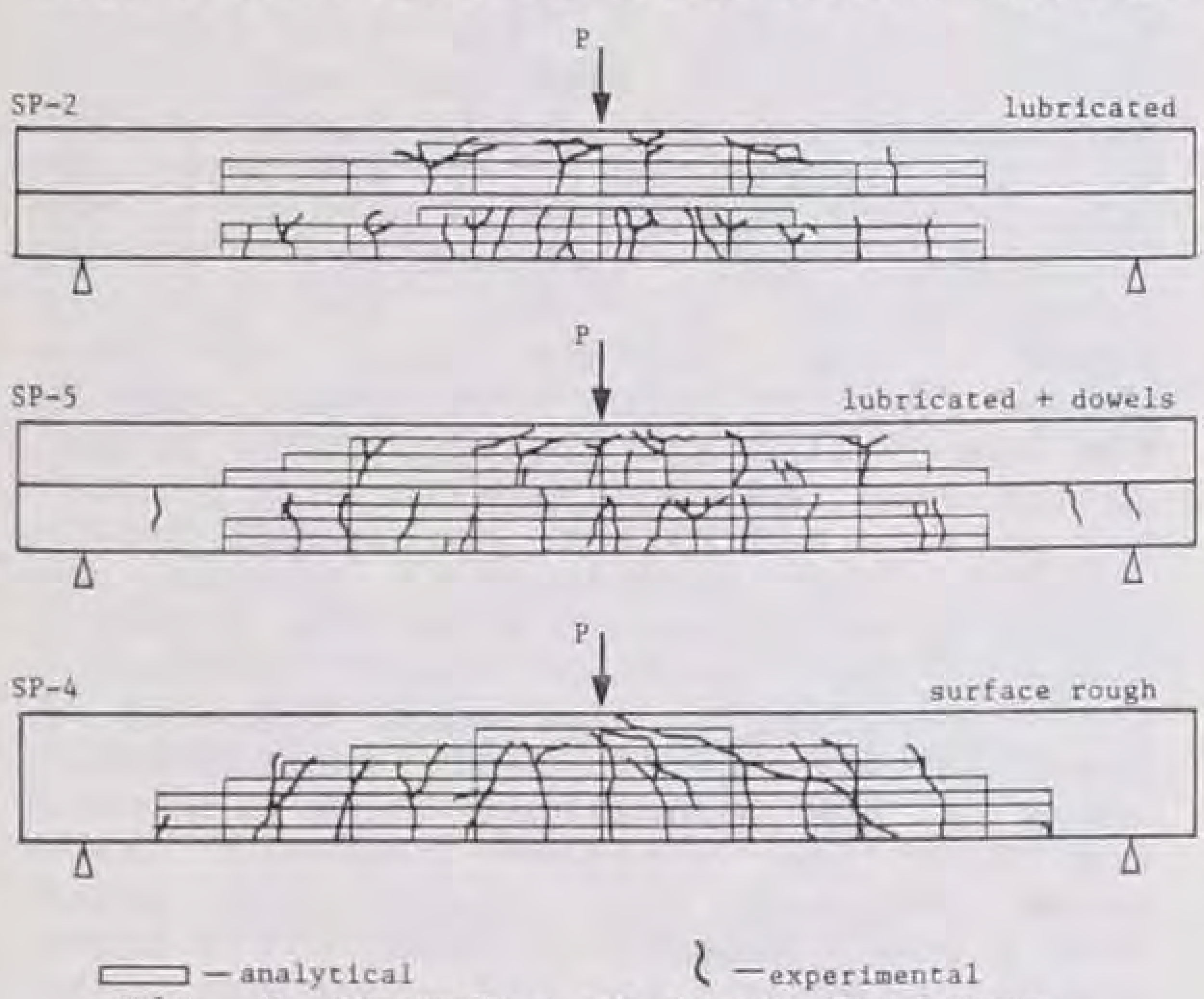

Fig. 4 Examples of Crack Patterns

Full-Scale Prototype Test

The above findings were incorporated in phase III of the experimental test program, where a 60 ft long x 12 ft wide section of a 3 span continuous T-girder RC highway bridge was removed from California State Highway 41 south of Fresno (Gepford Overhead) and brought to the Charles Lee Powell Structural Systems Laboratory for full scale testing under controlled laboratory conditions. Repair measures consisted of a 6 in. overlay with sandblasted and scarified interlayer (no dowels) over half of the bridge deck, respectively. An overview of the repaired Gepford Overhead section is shown in Fig. 5. The 8 ft overhangs past the bents allow for tie-downs and adjacent continuous span simulation.

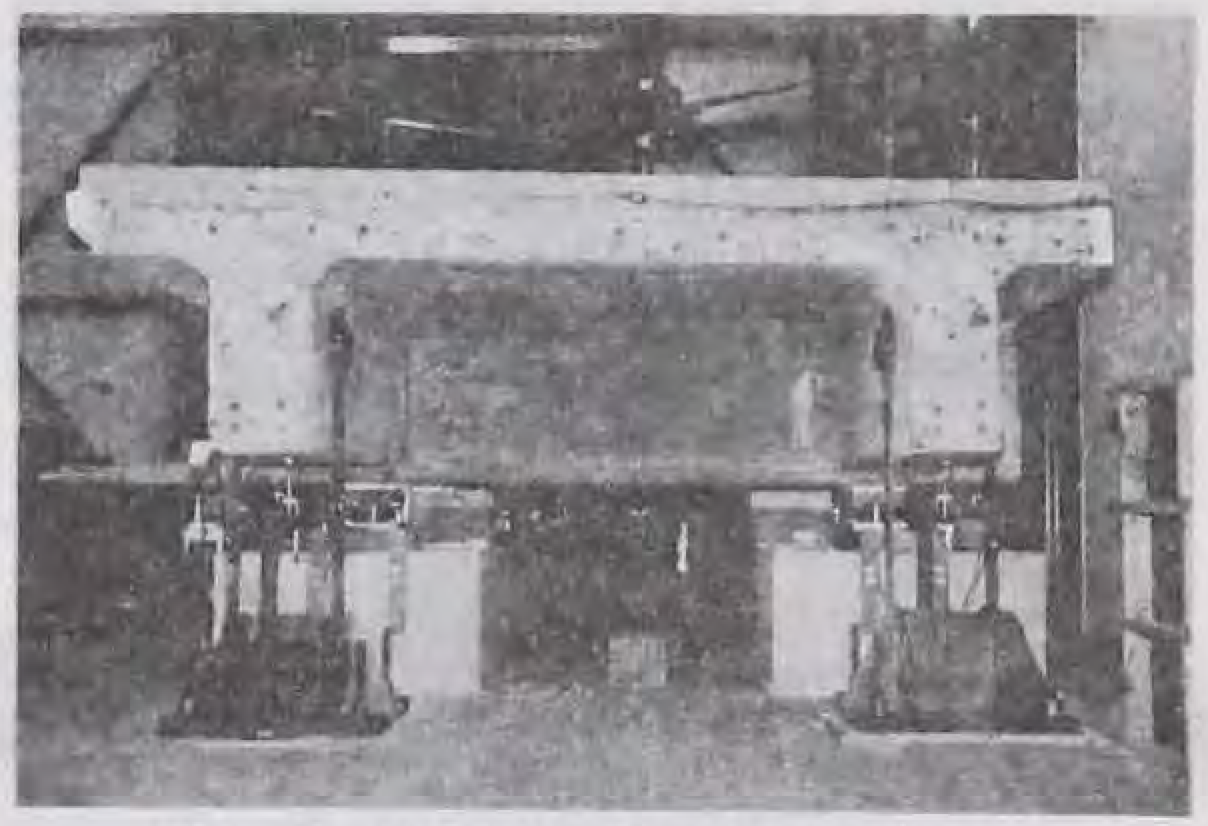

Fig. 5 Full-Scale Bridge Test

A series of point load (wheel load) and two point load (axle load) tests at midspan and the quarter span points was conducted in a monotonic loading scheme all the way up to the onset of yield in the main flexural midspan reinforcement. Subsequently, the bridge section was subjected to 200,000 dynamic (5Hz) quarter point traffic load cycles of ± 10 kips or ΔP = 20 kip about a 32 kip mean load level which corresponds to the 16 kip maximum wheel load and impact in addition to a uniform lane load of 640 lbs/ft. Again, no interlayer delamination was noticeable. Thus, the phase III prototype test confirmed the effectiveness of a "clean" and "rough" construction joint surface to achieve monolithic behavior.

ANALYTICAL OVERLAY STUDIES

To study the effect of interlayer slip on the overall flexural behavior of bridge decks, a two layer plate element with nonlinear interlayer slip was developed. This finite element model was then applied to an overlaid slab with a rigid-perfectly plastic interlayer constitutive behavior. The progressive delamination of the interlayer was traced starting with a delamination region under the point of load application. A second delamination region developed with in-prior to the overlay application consisted of the following interlayer types:

1. **monolithic:** specimen was cast monolithically to establish a basis for comparison.
2. **lubricated:** a "surface rough" (see below) construction joint was sprayed with a bond breaking agent (form oil) to eliminate chemical bonding between the "old" and the "new" concrete.
3. **surface rough:** wood trowel finished and lightly sandblasted interlayer surface.
4. **scarified:** grooves ≥ 1/8" deep @1" were cut into the "old" concrete surface with a jack hammer. The laitance layer was effectively removed and the coarse aggregate exposed.
5. **lubricated and dowels:** same as No. 2 but with #4 dowels set in 3/4" Ø drilled holes 4" deep with modified type II (expansive) portland cement grout, $\rho_d = 0.07\%$.
6. **surface rough and dowels:** same as No. 3 with dowels as described above.
7. **scarified and dowels:** same as No. 4 with dowels described above.

Examples of surface preparations can be seen in Fig. 1.

Fig. 1 Surface Preparation Examples

Shear Block and Slab Panel Test Results

The most important finding from the triplet shear block tests with vertical construction joints was the direct relationship between the dilation and the relative slip in the interlayer. Any measurable slip was always accompanied or preceded by a significant dilation of the construction joint.

The slab panel tests, which simulated transverse strips of bridge deck spanning 8 ft between longitudinal girders, were tested as simply supported and continuous slabs as shown in Fig. 2 and featured the typical horizontal construction joint with

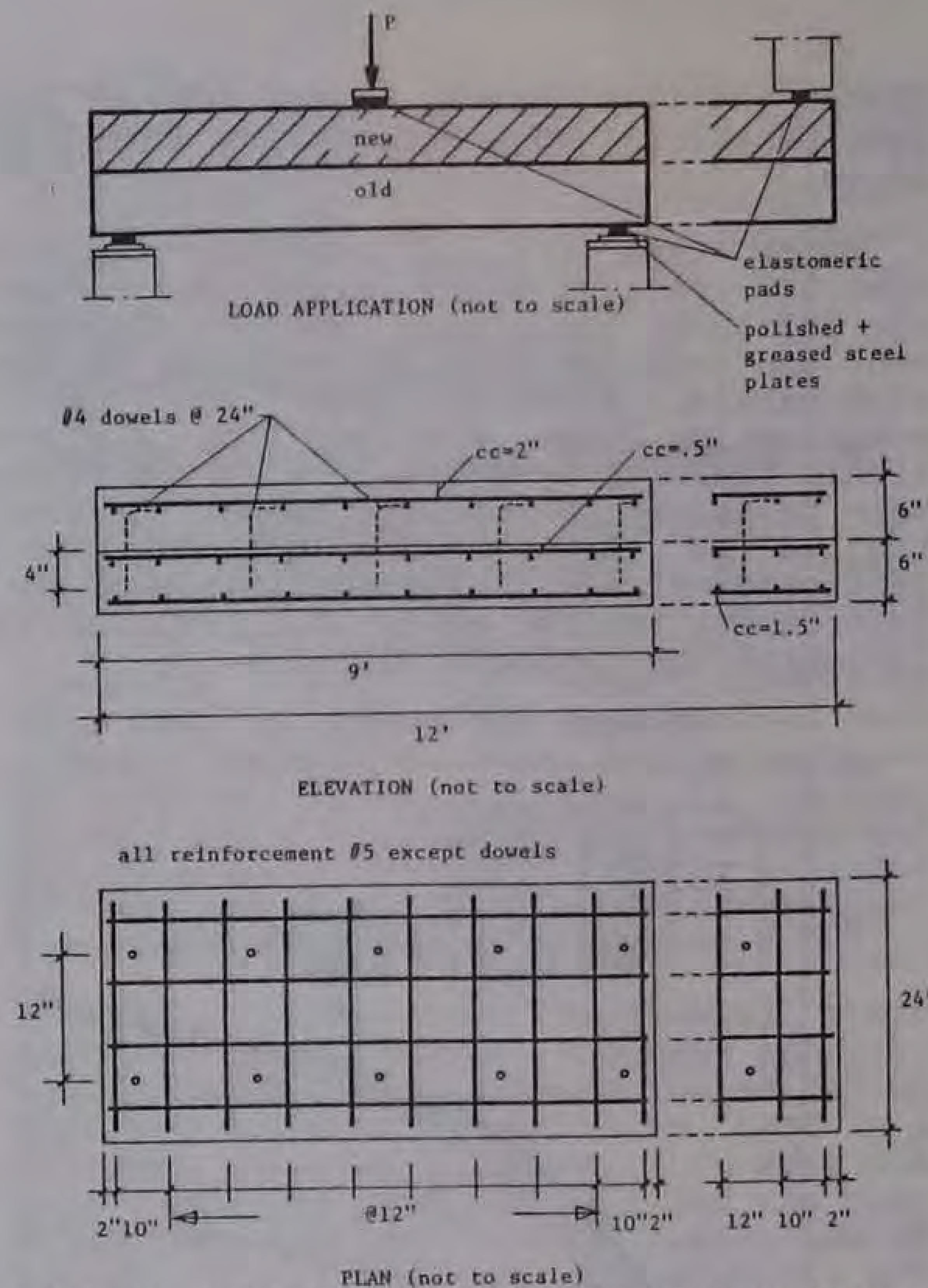

Fig. 2 Overview of Slab Panel Specimens

normal forces due to overlay self weight, applied loads and reactions in different slab regions. The load versus deflection plot in Fig. 3 for the simply supported slabs indicates that the behavior of the "lubricated" slabs was distinctly different from the other surface preparations which basically showed "monolithic" action beyond the yield in the flexural deck reinforcement. The same behavior can be deducted from the crack pattern development and propagation plots in Fig. 4. The "lubricated" test specimens exhibited very early delamination of the interlayer and independent flexural cracks in the "old" and "new" concrete slabs. The "surface rough" specimens showed flexural crack development only in the lower slab and flexural crack propagation into the overlay with only temporary arrests and slight horizontal deviations at the interlayer. Failure occurred by delamination after the flexural yield of the "surface rough" specimens. The "scarified" specimens featured a crack pattern development exactly analogous to the monolithic test specimens and only the ultimate flexural shear crack followed the construction joint over a short distance. The 0.07% dowel reinforcement did not influence the crack pattern development in the non-lubricated specimens and was not sufficient to control the ultimate delamination crack (SP-6 surface rough and dowels). The

THE WHITTIER EARTHQUAKE: FREEWAY BRIDGE PERFORMANCE, ANALYSIS, AND REPAIR

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SYNOPSIS

This paper describes a coordinated study of the performance of freeway bridges in the Whittier earthquake, involving research personnel at the University of California, San Diego; California Department of Transportation (Caltrans); and Imbsen & Associates, Sacramento. The study involves four stages: observed damage assessment, component strength and deformation analysis, dynamic analysis of selected bridge structures, recommendations on assessment, evaluation and prioritization for retrofitting. The intention of the study is to obtain maximum benefit from the lessons emphasized by bridge damage in the earthquake in the form of improved understanding of the vulnerability of existing bridge structures to seismic damage, calibration and assessment of analytical techniques for analysis of seismic response of bridge structures, and in the development and assessment of retrofit techniques to reduce the seismic risk of freeway bridges.

INTRODUCTION

Damage to freeway bridges in the Whittier Earthquake of October 1, 1987 and the major aftershock of October 4, 1987 was one of the most significant aspects of performance of engineered structures designed for seismic lateral loads. Of particular importance was the performance of the I-5/I-605 separator, a major freeway overpass that was extensively damaged during the earthquake, and came perilously close to collapse. According to a California Highway Patrol officer who was close to the scene at the time of the earthquake, morning rush-hour traffic was at its heaviest, and both the I-605 bridge and the I-5 freeway below were carrying maximum density traffic at crawling speed. It is estimated that if the damaged section had collapse, between 70 and 100 cars would have fallen with the bridge, or been crushed underneath, with inevitable extensive loss of life.

In this paper, the structural performance of the bridge is briefly discussed, and details of a current coordinated study of the performance of freeway bridges in the Whittier earthquake are outlined.

DAMAGE TO THE I-5/I-605 SEPARATOR

Structural Description

The I-5/I-605 separator is a complex nine span bridge carrying eight lanes of the I-605 over the six lanes of the Santa Ana (I-5) Freeway, and an additional four feeder lanes. This major bridge structure is located in northwest Orange County some six miles from the Whittier earthquake epicenter. The bridge was designed and constructed in the early to mid 1960's.

As is illustrated by the plan view of the bridge in Fig. 1, the geometric constraints imposed by the I-5 and feeder lanes have resulted in considerable structural complexity. The axes of the I-605 and I-5 are skewed 37.5° from perpendicular, which is reflected in column support lines on Bents 5, 6 and 7, but the skew of Bents 2-4 and Bents 8 and 9 are based on the geometries of the adjacent feeder lanes. The variable skew of the different support lines results in Bents 4 and 5 (and Bents 7 and 8) sharing one common column.

The structural complexity is continued in the superstructure. From Abutment 1 to Bent 5, the superstructure is a continuous 5'-2" deep multicell reinforced concrete box with 8" wide webs at approximately 7'-10" centers, a 6.5" thick deck slab, and 5.5" thick soffit slabs. The construction from Bent 7 to Abutment 10 is similar. Between Bents 5 and 6, and 6 and 7, precast, prestressed I-Beams are simply supported on cap beams supported by columns. The total depth of the I-Beams and in situ deck slab of 6" is 5'-2", and the beams are at nominal 6.18' centers (20 beams across the width).

Thus there are movement gaps along the center lines of Bents 5, 6, and 7. These are carried through the deck slab, and typically allow 1" movement. Despite fairly generous seatings for the I-Beam (the continuous reinforced concrete box sections are monolithic with the cap beams at 5 and 7), the joints have been retrofitted with restrainer cables to avoid excessive movement during seismic attack. These cables were fitted by the California Department of Transportation (Caltrans) as part of a major program to provide restraint across movement joints following extensive bridge damage in the 1971 San Fernando earthquake.

As shown in Fig. 2, this results in columns framed into stiff beams top and bottom, and with an average clear height of about 12'-6", are much shorter than the columns at the adjacent Bents 5 and 7. The combined effects of base fixity and reduced height make Bent 6 about 15-20 times more stiff than the other bents of columns. This clearly contributed greatly to the damage sustained by Bent 6 which is sketched on Fig. 2 and discussed in more detail later. Note that the scale of Fig. 2 has been deliberately distorted to aid clarity.

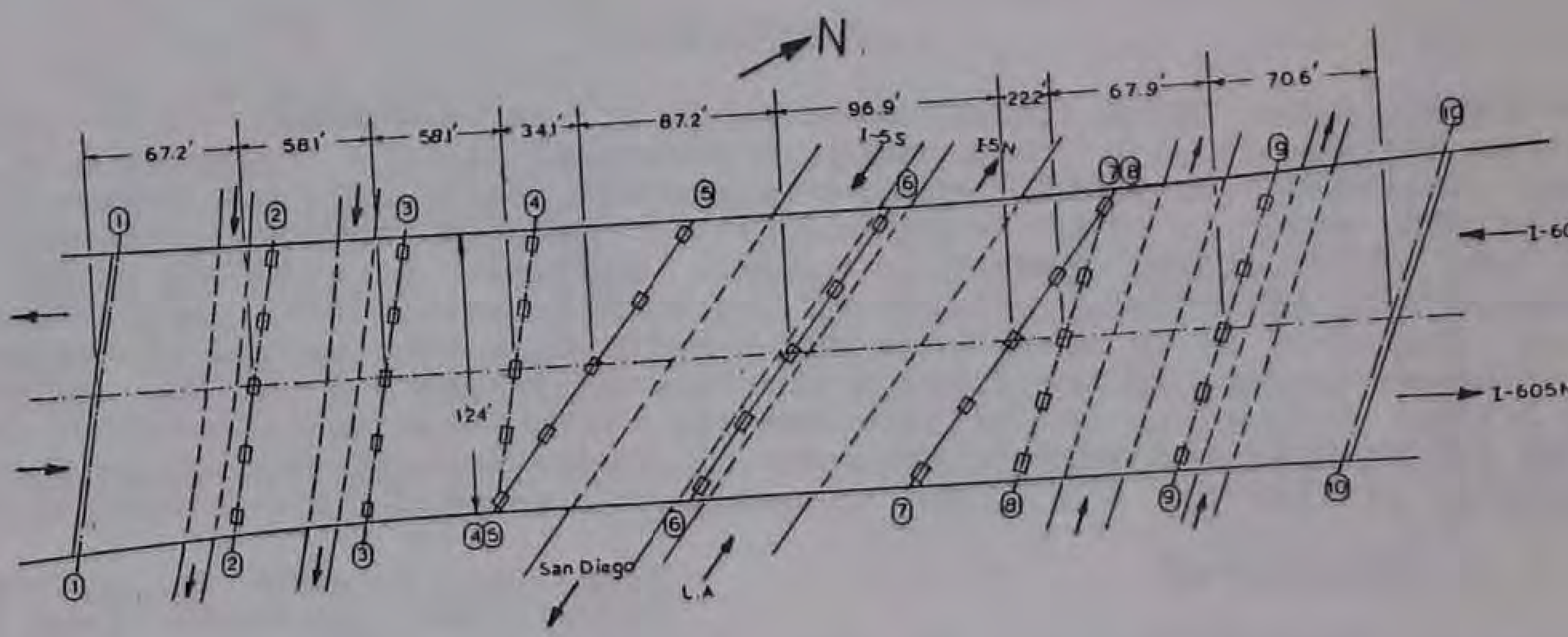

Fig. 1 Plan View of I-605 Overpass Structural Configuration

Abutments 1 and 10 (see Fig. 1) are supported by steel rocker plates allowing free longitudinal movement. Transverse displacement at these abutments is restrained by friction and by keeper plates on either side of the rockers.

Bents 2, 3, 4, 8, and 9 each consist of 5 columns 48" x 36" in section, framing into diaphragms in the continuous box-section superstructure. The column bases are pinned to individual footing pads supported on concrete piles. The pinned-base detail is commonly adopted by Caltrans for multi-column bents to reduce design moments for footings and piles. Typical column height varies from 21' at Bent 2 to 28'at Bent 4.

Bents 5 and 7 frame into the cap beams supporting the ends of the prestressed I-Beams. The column bases are pinned to individual footing pads as with Bents 2, 3, etc. above.

At Bent 6, the columns also frame into the cap beam at the top. However, at the base, the columns frame into a continuous footing pedestal, which is founded on cast-in-drilled-hole concrete piles located in the median strip of the Santa Ana Freeway.

Observed Damage

The most obvious damage sustained by the bridge, and that primarily recorded immediately after the earthquake, was the shear failures of all five columns on Bent 6, as sketched in Fig. 2, which represents the appearance of the NE side of Bent 6. It is apparent that major damage was associated with a response pulse of the bridge to the right, or NW, in Fig. 2, though severe shear cracking on the opposite diagonals indicated strong response in the SE direction also.

Column 1 (see Fig. 2) showed less damage than the other columns, possibly due to the flexural cracking of the cap beam, which would have reduced the stiffness of this column relative to the others in the bent. Shear failure of the other four columns was severe, with crack widths of more than 1.0" being recorded. At the center column, two #4 transverse rebar stirrups fractured, and a third had a region of imminent failure, indicated by visible necking.

Damage to other parts of the bridge was rather minor and included some spalling of concrete at Abutments 1 and 10 due to abutment pounding, anchorage failure above Columns 1 and 5 in Bent 5, and some

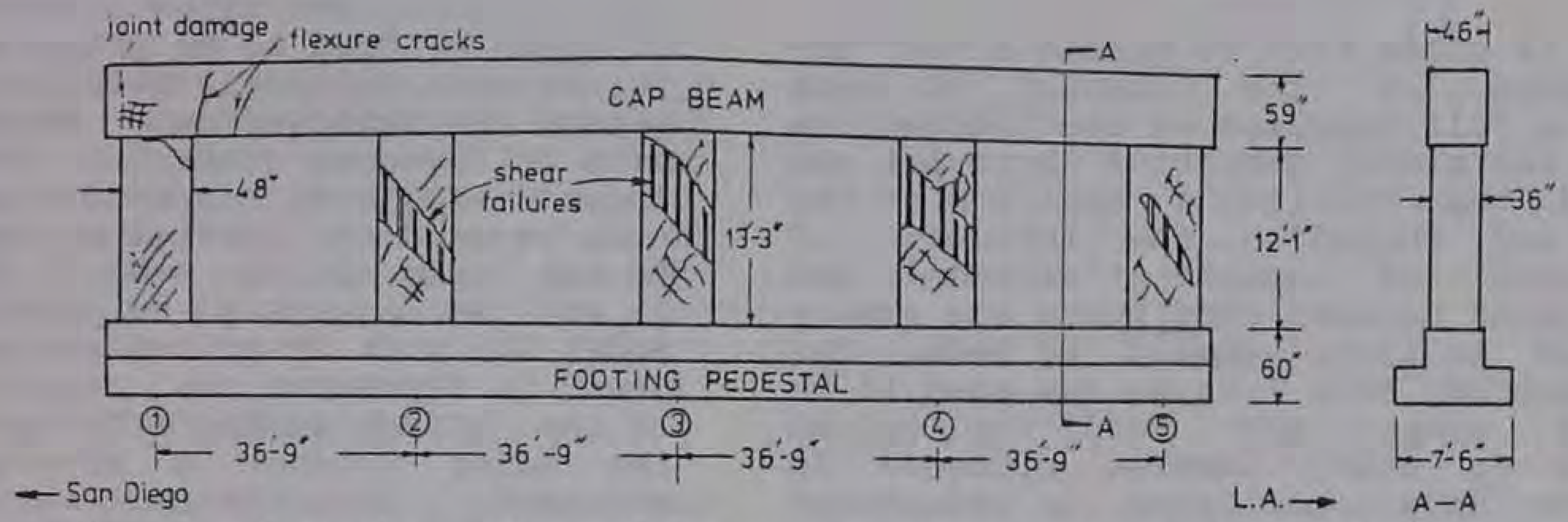

Fig. 2 Dimensions of, and Damage to Bent 6, October 1, 1986

minor damage to keeper plates at the support rockers at each abutment. This damage was not recorded nor observed by the author during this initial visit but was reported by Caltrans staff.

The major M5.5 aftershock of 4 a.m. on October 4 caused further damage to the bridge. All columns of Bent 6 showed considerable extension to cover spalling, but no further fractures of shear reinforcement. It is clear that by this stage the lateral stiffness of these columns had effectively reduced to zero, and thus Bent 6 was not contributing to seismic resistance of the bridge as a whole.

Additional spalling was observed at the top S corner of columns on Bents 2 and 3, apparently associated with incipient development of flexural plastic hinges at the column top. The damage was commensurate with a flexural displacement ductility factor of about 1.5 to 2.0 caused by displacement to the N or NE.

Discussion of Damage

Preliminary calculations indicate that the shear failures of Bent 6 were a consequence of the much greater stiffness of this bent relative to adjacent bents, and due to elastic design philosophy in use at the time that the bridge was designed. Fig. 3 shows the reinforcement details for the exterior columns (1 and 5) and the interior columns (2, 3, 4) of Bent 6. Longitudinal reinforcement ratios were very high: 6.02% and 4.43% for the exterior and internal columns respectively. Transverse reinforcement, at two #4 legs at 12" centers is nominal. Based on these details, and a reasonable range of possible material strength, shears corresponding to flexural cracking, flexural strength, and shear strength have been calculated and are summarized in Table 1.

It is clear from these data that a shear failure was inevitable, since shear strengths were only between 23% and 42% of flexural strength, depending on which column, and what assumption for concrete strength is adopted. As mentioned above, this is a consequence of the use of elastic design methodology, particularly as is related to design of columns for flexure. The susceptibility of these columns is typical for squat bridge columns designed according to elastic design.

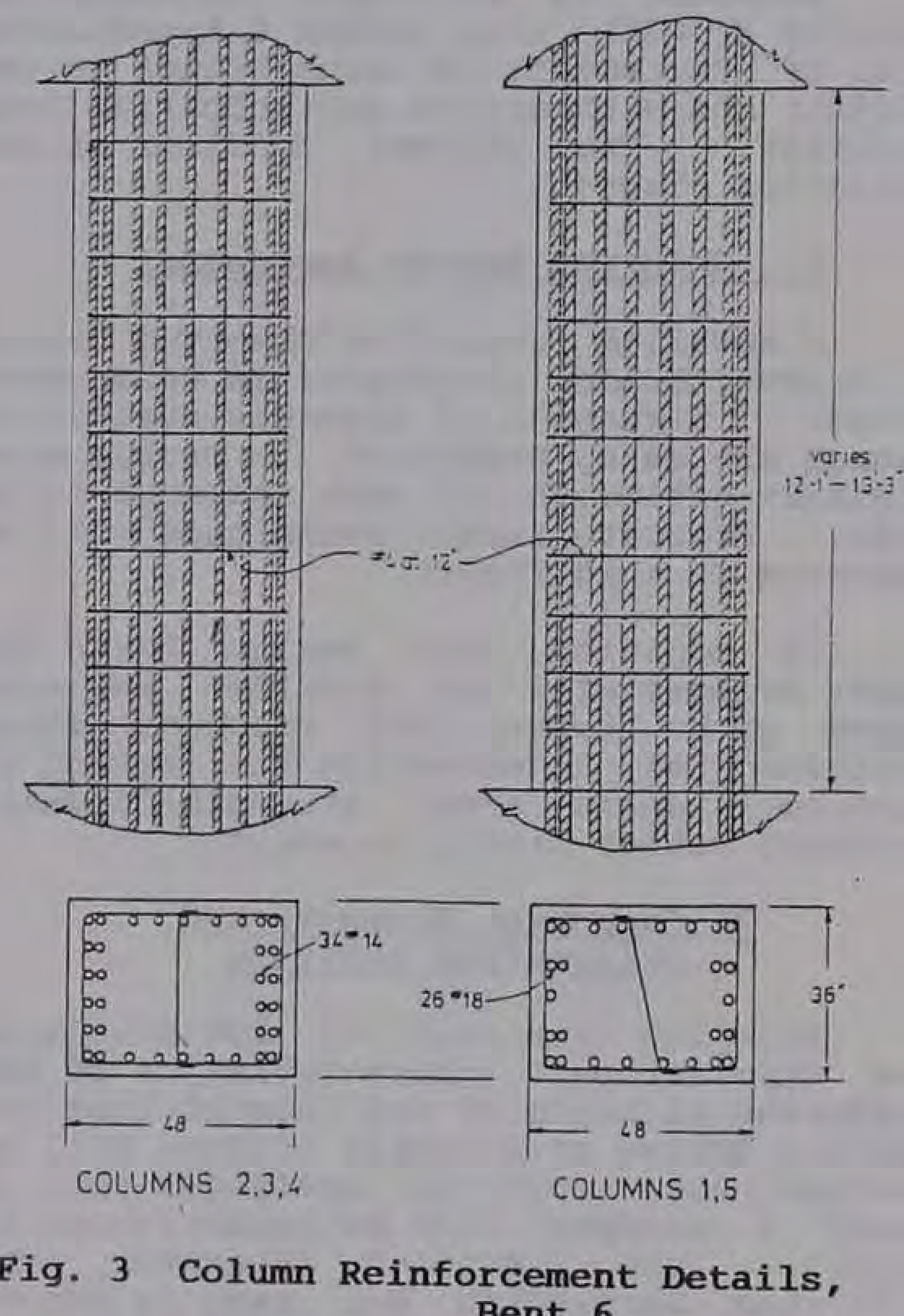

Fig. 3 Column Reinforcement Details, Bent 6

Table 1 Column Shear Forces (Kips)

Shear Force	Columns 1,5 Min	Columns 1,5 Max	Columns 2,3,4 Min	Columns 2,3,4 Max
At Flexural Cracking	144	202	169	223
At Flexural Strength	1210	1230	905	935
At Shear Strength	283	290	304	390

RESEARCH PROGRAM

It is clear that to obtain a full understanding of the behavior of this bridge, a full analysis of the interaction between the simply supported spans 5-6 and 6-7 and the continuous sections of the bridge and including the influence of connections and support at abutments and at the joint between continuous and simply supported sections should be made. The importance of this lies in the need to be able to assess the need for column retrofit to other complex bridges. In order to have confidence in assessment techniques and to avoid being unnecessarily conservative or unreasonably optimistic, the assessment techniques need calibration against real data. The I-5/I-605 bridge is an ideal candidate for such a calibration. The damage was very extensive, but collapse, which often results in evidence of the cause of failure being lost, did not occur.

Research is currently underway by faculty at UCSD, with Imbsen & Associates, Inc. of Sacramento as consultants to the project and with active participation from Caltrans. The project consists of the following stages.

1. Observed Damage Assessment

A detailed report on observed damage to freeway bridge structures is being prepared. Strengths of elements sustaining damage are being predicted to enable preliminary estimates of peak response to be made. Proposed repair techniques will be included in this report.

In addition, the design basic for older bridges will be examined and compared with current and proposed design methodologies, to establish the extent of disproportionate shear strength/flexural strength ratios likely to exist.

2. Component Strength and Deformation Analysis

Detailed analysis of the strengths and deformation characteristics of the different elements of the lateral load resisting system of selected bridges will be carried out. It is envisaged that at least 4 bridges will be investigated in detail: the I-605/I-5 separator, the I-710/San Bernardino bus ramp (a curved multispan bridge with observed damage to column top hinges) and two bridges, yet to be selected, located in the epicentral region that did not suffer significant damage, but which are estimated to have responded at close to ultimate strength and which represent typical generic structural bridge configuration.

This section of the study will involve moment-curvature analyses for all lateral-load resisting elements of the selected bridges to be prepared and integrated into realistic lateral-load deformation characteristics. Recent research data on flexural strength, ductility and shear strength of concrete bridge columns will be used to predict strength levels. In the case of the I-605/I-5 separator, it will be necessary to construct a new model for post shear-failure stiffness in order to represent the complete response of the bridge during the earthquake and the major October 4 aftershock. The strength deformation characteristics developed in Section 2 will be used as input for the analysis of Section 3.

3. Dynamic Analyses of Selected Bridges

The bridges will be subjected to a series of lateral load analyses of different levels of complexity, from equivalent static analyses using tributary mass (which will have been the design basis for the bridges), through elastic spectral modal analyses to 2-D and 3-D dynamic inelastic time-history analyses. In all cases, the input will be accelerograms recorded near the bridge sites during the earthquake and aftershock. Analyses will also be performed to investigate:

- The effectiveness of cable restrainers and other retrofitting measures installed in the bridges.
- The effect of premature column shear failure on overall bridge response (I-605/I-5).
- The response of the bridges in the repaired condition (if appropriate).

Because of the computer resource required to conduct transient nonlinear analyses, the computer processing will be conducted on the San Diego Cray XMP Supercomputer at UCSD.

4. Recommendations

The results of analyses carried out in Stages 1 to 3 of the project will be used to prepare detailed recommendations on the assessment of complex bridge structures for retrofitting. Particular emphasis will be placed on a comparison between analyses of different levels of sophistication, from simple equivalent lateral load analysis using tributary mass, to full three-dimensional dynamic analysis for predicting response and need for retrofitting. The special problems involved in assessing probable response when shear failure of columns is possible will receive particular emphasis. Guidelines establishing "high risk" structural profiles will be developed to aid designers to identify bridges needing urgent consideration for retrofit.

Techniques for retrofitting bridge structures to improve flexural strength and ductility and shear strength of existing bridges will be discussed and evaluated. Where needs for further re- search information are identified, recommendations for further research programs will be made.

CONCLUSIONS

Approval to proceed on this project was obtain in June 1988. Currently, work is underway on a detailed analysis of the I-5/I-605 separator. Analyses of other bridges will proceed shortly.

This program should result in effective means for identifying 'at risk' bridges being developed, with recommendations as to viable and economical techniques for repair or retrofitting. In this context, the project may be seen as complimentary to another project described in a paper to this symposium [1].]

ACKNOWLEDGEMENTS

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SESSION IIB

COMPARISON OF THE ONTARIO AND AASHTO DECK DESIGNS

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SYNOPSIS

The principle objective of this study is to compare the service level stresses for the two reinforced concrete bridge deck designs. One deck is constructed per the AASHTO specifications and the other is constructed per the Ontario Highway Bridge Design Code (OHBDC). Both are subjected to static loads, dynamic loads of known static magnitude and interstate truck traffic. The prototype study should yield insight into the behavior of minimally reinforced non-composite concrete bridge decks. The study could provide additional impetus to implement the specifications of the OHBDC as related to bridge deck design into the AASHTO specifications.

INTRODUCTION

The recent modifications of the bridge design code by the Ontario Ministry of Transportation and Communications have resulted in substantially less reinforcing steel for concrete bridge decks than conventional design (8). This reinforcing scheme and associated research has not been well accepted by the United States bridge design community. The American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges requires transverse reinforcing be based on elastic distribution of flexural actions in slabs (9,10). Converse to the traditional design assumptions, recent studies have illustrated that punching shear, and not flexure, is the usual ultimate mode of failure (1,2,4). Much of the research supporting minimum reinforcing requirements has been conducted by the Ontario Ministry. This research consists of model (1,2,3) and prototype (6,7) tests on composite bridge decks. The Ontario Highway Bridge Design Code (OHBDC) incorporates these results and specifies an empirical 0.3% isotropic design of composite bridge decks (9). United States research has involved a few model studies (4,5) and one prototype bridge test (5). These tests, in addition to field tests on in-service bridges by the Ontario Ministry, have indicated that OHBCD is also adquate for non-composite bridge decks.

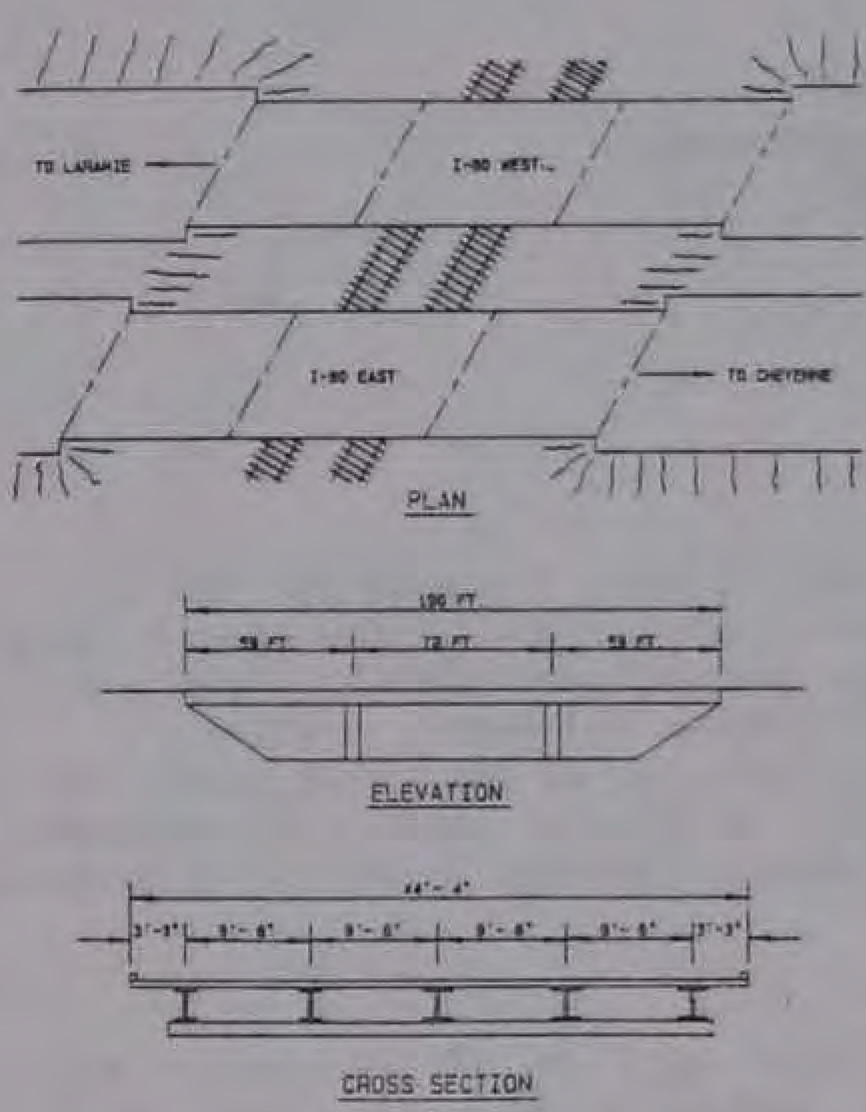
In conjunction with the Wyoming Highway Department, the University of Wyoming is conducting tests of two non-composite prototype bridge decks consisting of reinforcing schemes designed per the AASHTO bridge design code and OHBDC. The principle objective is to compare reinforcement stresses for the two designs subjected to static loads, dynamic loads of known static magnitude and interstate truck traffic. The prototype study should yield insight into the behavior of minimally reinforced non-composite concrete bridge decks. The study could also provide additional impetus to implement the recent specifications of the OHBDC as related to bridge deck design into the AASHTO specifications.

The objective of this manuscript is to describe the instrumented bridge and the expected test results. An overview of the bridge and strain gage layout is presented in conjunction with the load test descriptions. Test results will be presented elsewhere after the final report has been published.

DESCRIPTION OF THE INSTRUMENTED BRIDGES

Testing of two bridges built in 1958 was made possible by the replacement and widening of their existing decks. One new concrete deck was designed in accordance with AASHTO specifications while the other deck is designed by the OHBDC. Portions of each deck are instrumented to determine the reinforcement stresses.

The two bridges are located on Interstate I-80 approximately 13 miles west of Cheyenne, WY. Each bridge is composed of three spans 59, 72 and 59 feet in length. The superstructure consists of steel plate girders with a non-composite concrete deck and all girder supports employ rocker bearing type devices. Each new concrete deck has a clear roadway width of 42 feet with traffic barrier curbs of 1 ft. - 4 in. providing a total width of 44 ft. - 8 in. The five girders, one is newly constructed to provide for the deck widening, are spaced 9 ft. -6 in. apart. Each deck cantilevers 3 ft. - 6 in. from the exterior girders. The bridge has a skew of 30 degrees and is approximately level. The general layout of the bridge is shown in Figure 1.



General Bridge Layout
Figure 1

The eastbound deck was designed according to the AASHTO Bridge Specifications and the westbound deck was designed per the OHBDC. The latter require approximately 40 percent less reinforcement in critical sections and approximately 60 percent less steel tonnage. Further, because of less reinforcement in the top layer, frequently exposed to roadway salts, less maintenance is expected and hence lower life-cycle costs can be expected.

BRIDGE DECK INSTRUMENTATION AND LOADING

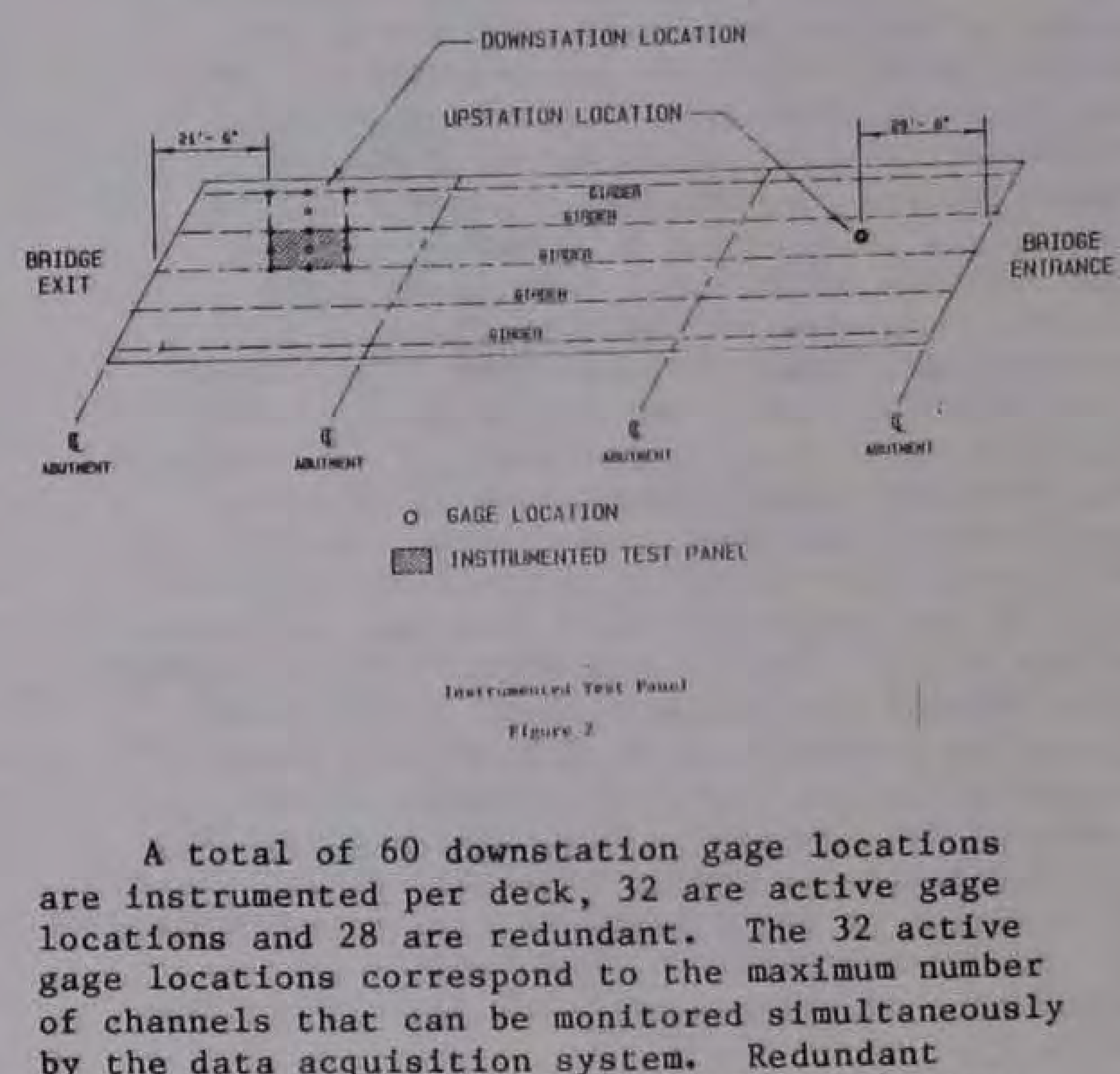
The major impetus of the study is the comparison of the service load level characteristics of the two designs. Deck reinforcement stresses are used as a basis for comparison of the designs. Load tests on the OHBDC deck provide information on the minimal reinforced non-composite decks that to date has not been addressed.

Three types of loading will be applied to the areas instrumented: static loads, dynamic loads of a known static magnitude and interstate truck traffic. Each prototype bridge is subjected to concentrated loads simulating a dual wheel at various locations on the deck. The load magnitude was selected to provide information at the service level.

Moving loads provide information when the test vehicle wheel path, speed and load are known. Reinforcement stress vs. time response is recorded and the most important characteristic, the peak stress is determined.

Traffic loading provides information for deck slabs subjected to continuous interstate truck traffic. This type of loading has not been investigated for reinforced deck slabs designed by the OHBDC. These dynamic loads are similar to the test except the load magnitudes, speeds and paths are unknown. Because of this, the only data that can be rationally analyzed and compared for both bridge designs is the peak stress. The time period each deck is monitored will be based on the consistency of the data. A three month period for each deck is expected to be adequate.

One hundred and twenty-eight locations were instrumented on the reinforcing bars. The majority of these gage locations are on a portion of the bridge referred to as a deck panel. A deck panel, defined as a rectangular area bounded by two adjacent girders and lateral bracing, was selected on each prototype bridge as shown in Figure 2. The panel is 18 feet long and 9 ft. - 6 inches wide. The instrumented panels are approximately 21 feet from the bridge exit to minimize the impact effects. The deck panel was selected directly under the traffic lane to allow monitoring of traffic as part of dynamic testing.



Instrumented Test Panel
Figure 2

A total of 60 downstation gage locations are instrumented per deck, 32 are active gage locations and 28 are redundant. The 32 active gage locations correspond to the maximum number of channels that can be monitored simultaneously by the data acquisition system. Redundant locations are used in the case of malfunction, construction damage, etc.

The gage layout for dynamic loading consists of 31 active downstation gage locations and one active upstation location. The active gages are chosen from the 60 downstation locations based on data acquired during static load testing. The upstation locations consist of one active and three redundant locations instrumented on bottom transverse bars. As the load truck passes over the bridge, it causes the strain in the upstation gage to exceed a specified value. This strain triggers the DAS to monitor the downstation gages.

RESULTS

The service level reinforcement strain/stress are to be reported. Specifically, the following data is expected on the short term:

1. Influence surfaces for each gage,
2. Strain vs. time/position for known moving loads,
3. Maximum strains for known moving loads,
4. Maximum strains for interstate truck traffic surveyed continuously over a period of several months.

Additional long term performance can be observed including deck deterioration (delamination, spalling, cracking, etc.) and change in the maximum strain response of interstate traffic.

PROJECT STATUS

The testing has been completed except for the interstate traffic monitoring. Data is being summarized and reported at the time of this writing.

ACKNOWLEDGEMENT

The technical and financial support of the Wyoming Highway Department is greatly appreciated.

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3

EXODERMIC BRIDGE DECKS–INSTRUMENTATION AND TESTING OF A BRIDGE REHABILITATION

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SYNOPSIS

Recently, an exodermic deck was used to replace a standard, full depth cast-in-place composite reinforced concrete deck on a bridge over the New York State Thruway. The exodermic deck, consisting of a layer of reinforced concrete poured over a steel grid in prefabricated panels, is much lighter and simpler to install than standard composite decking. Thus, it can be used for redecking existing bridges, or for decking new bridges, much more quickly, and at less expense, than are required for current standard procedures. This particular exodermic deck is the first to be installed in New York State.

In order to verify the load-carrying capability of the exodermic deck, a series of live load tests were conducted on this bridge before, and after, redecking. The results of these tests compare favorably to analytical predictions. While the live-load strain for the exodermic deck is somewhat higher than the original deck, as expected, the dead-load strain is considerably less and the overall strain is well within safe limits.

The philosophy of "less is more" can probably be applied to the determination of serviceability of a bridge deck. Specifically, a lower stress range in any material is known to positively affect fatigue life and can probably be assumed to extend the useful life of any material.

The history of innovation in structural materials, systems, and analysis has generally concerned means of achieving higher levels of stress in all materials. High strength steel, high strength concrete, high strength prestressing strand, and structural systems seeking to maximize material usage in the search for the lowest first cost have been the norm rather than the exception.

In the past 30 years three new types of bridge decks have evolved: orthotropic, prestressed concrete, and exodermic. Only one of these bridge decks can be said to fit the description "less is more" - exodermic.

Volume II of the Standard Plans for Highway Bridges entitled "Structural Steel Superstructures", which was originally published in 1962 by the Federal Highway Administration, served as the basis for the design of many highway bridges which are now 25 years old. In addition, it is safe to say that there are probably many similar bridges between 25 and 40 years old which have superstructure arrangements quite similar to that shown in the 1962 publication.

A typical stringer spacing for I Beam composite spans from 50 to 90 feet in the FHWA publication was 7'-8". The deck recommendation was a cast-in-place concrete deck 7 inches thick, with 1 inch of cover for the bottom reinforcement and 1 1/2 inches of cover over the top reinforcement. This deck, using standard weight concrete, weighed 85 lbs. per square foot. In a typical design, the main reinforcement bars, top and bottom of the slab, were #5 bars at 6 inches on center. Allowable steel stress was 20,000 psi and allowable concrete stress was 0.35f'c.

A lightweight exodermic deck providing 2 inches of cover over epoxy-coated reinforcing steel used as a replacement deck would weigh 60% of the weight of the typical deck shown in the 1962 publication. Maximum concrete and steel stresses in the exodermic deck would not exceed 50% of allowable. The implications of this, for serviceability in the future, are obvious. We believe that when there is no reason to maintain the existing dead load of a bridge deck, lightweight exodermic deck will be the preferred option.

Since the top surface of an exodermic deck is portland cement concrete, it is also obvious that in the case of a deck replacement or a new deck, for a bridge for which the dead load of the deck is desired to be heavier than that of exodermic deck, an appropriate thickness of asphaltic concrete can be used to achieve the desired dead load. In most cases, on both long span and short span bridges, the use of the lightweight, low service stress exodermic deck will be the preferred option.

The reason that live load stresses in an exodermic deck are dramatically lower than those in the standard cast-in-place concrete bridge decks is that the stiffness of a bridge deck is directly proportional to the cube of the depth. At midspan, the effective depth of an exodermic deck for the standard 7'-8" stringer spacing would be 8 1/4 in. The effective depth at midspan of the original 7 in. cast-in-place deck is 5.6875 in. This means that the exodermic deck stiffness is over 3 times the stiffness of the cast-in-place deck.

The stresses in the two decks vary as the square of the effective depth, and a similar comparison shows that the stresses in the exodermic deck will be less than half those of the original deck during the passage of live load.

The design of exodermic deck, when applied to the bridge rehabilitation needs of the United States, produces an astonishing result:

First, the deck lends itself to future replacement while maintaining traffic; either the reinforced concrete top half of the deck, or complete modules if necessary, can be removed and replaced.

Secondly, the concrete and steel live load unit stresses in an exodermic deck supported on existing floorsystem framing on most bridges between 25 and 40 years old will be approximately half that in the original deck, and the dead load stresses in the framing will also be reduced by half.

The development of exodermic deck required an easily accepted basis for design. The use of the current AASHTO rules for the design of a concrete filled grid was decided upon.

The result of full scale testing has confirmed the validity of that decision.

The complete report which is reviewed in this paper has been submitted for publication in the ASCE Structural Journal. ASCE rules do not permit publication of the paper by others. We can however send copies to those wishing to have one for his/her information.

The objective of the paper was to take productive advantage of the replacement of a 35 year old composite reinforced concrete deck with a composite exodermic deck on the original framing members. An analysis and the static live load tests to confirm computed results are presented for both the original and rehabilitated bridge. Good agreement confirms both the design specification adequacy and the analytic technique.

The Russell Road Bridge is a four span, two lane highway bridge over the New York State Thruway at Milepoint 145.38 which is in Albany between exits 23 and 24.

The structure was designed 35 years ago as a four stringer, four span, simply supported bridge. The 7 1/2 inch reinforced concrete deck, 31 ft. wide, was designed and constructed composite with four rolled beam stringers with bottom cover plates, spaced 8'-10" c.-c. The deck carried a 4 in. concrete overlay plus 2 in. of asphaltic concrete, for a total thickness of 13 1/2 inches.

The original deck and spiral shear connectors were removed, and an 8 3/4 in. exodermic deck was installed using welded headed studs as shear connectors.

On December 5, 1985, while the original deck was still in place, the stringers were instrumented and static live load measurements were made using a truck furnished by the New York State Thruway Authority. This process, using the same truck and virtually the same weight, was repeated on September 23, 1987 after the exodermic deck installation on the original steel stringers had been completed.

All four stringers were instrumented with SR-4 strain gages (Micromeasurements Inc.) with a gage factor of 2.00 (+/-1%). There were two gages mounted on the bottom face of each stringer at the midspan, the gages were mounted approximately one-third the distance from the centerline of the stringer to the edge of the flange, to either side of the centerline. The gages were oriented such that the major measurement axis was parallel to the principal longitudinal axis of the stringer. In this way, the gages would be most sensitive to the bending strains in the stringers. The strain gages, along with a common temperature compensation gage, were all wired to a BLH Model 1225 Switching and Balancing Unit. This unit was connected to a BLH Model 1200A Digital Strain Indicator which would read out calibrated values of strain (in microstrain).

Data was collected with a 67,000 pound (30,000 kg) truck, located in each of four positions on the bridge:

1. Against the west curb of the bridge
2. Centered over the west-center stringer
3. Against the east curb
4. Centered over the east-center stringer

In all cases the truck was located at the midspan of this set of stringers. Data was collected for the unloaded bridge at the beginning and end of the tests, and between each set of loaded data, for the calculation of true strain.

Substantial differences in data from two strain gages on the same beam would indicate the presence of twisting in the beam. Considering that the actual resolution in this data is about +/-5 microstrain, in no case is there significant evidence of beam twisting. Also, for all loading conditions, the relative values of strain in the four beams is both consistent and logical.

After the bridge was redecked, the test procedure was repeated. On September 23, 1987 all four stringers were instrumented with new strain gages (Micromeasurements Inc., model CEA-86-250UW-350) with a gage factor of 2.075 (+/-0.5%). This was necessary since none of the original strain gages was still functioning. The gage mounting, gages, switching and balancing equipment, etcetera used for the 1987 procedure were identical to the 1985 test procedure.

The actual resolution in the data from 2 strain gages on the same beam was again found to be about +/-5 microstrain. In no case is there significant evidence of beam twisting. Also, for all loading conditions, the relative values of strain in the four beams is both consistent and logical, as was the case during the first test.

Comparing these results to those from the 1985 set of tests, about a 90% increase in the live load strain is observed. However, the dead load strain (which was not measured) is much lower, due to the dramatically reduced weight of the new decking, as is shown below. In any case, the maximum live load stress, corresponding to the measured strains with the new decking, is only about four kpsi (28 MPa).

An analytical study of the bridge loading for each of the construction configurations was conducted in order to verify the test results and compare the live load to dead load stresses. The section moduli for the original construction and for the rehabilitated bridge are derived. In both cases, the section moduli are determined separately for the bridge sections associated with the outboard and inboard stringers. The bridge cross-section is symmetric, so that the two outboard stringers have identical section moduli, as do the two inboard stringers. The section moduli derived are approximations and, if anything, are slightly conservative (i.e. less than the actual values). The section moduli for the exodermic deck construction are somewhat less than the corresponding values for the original construction, which is in general agreement with the results of the tests which showed increased live load stresses for the rehabilitated construction, but result in substantially reduced calculated dead load stresses.

The calculated stresses, together with those determined from the measured strain values, are given. Considering the approximation associated with the calculated results, they are in reasonable agreement with the measured results. In particular, the assumption that the ends of the bridge stringers carry no moment (i.e. are simply supported) results in larger than realistic calculated stresses. The smaller discrepancy in the data for the rehabilitated bridge may indicate the calculation of more realistic section moduli for the exodermic deck than for the original construction, because of the lack of certainty concerning the extent of participation of the concrete and asphalt surface courses.

Using similar assumptions, the dead load stresses for the two constructions were calculated. For these calculations, the weight of the original deck was taken to be about 160 pounds per square foot (7.7 kPa) and the weight of the exodermic deck was taken to be about 65 pounds per square foot (3.1 kPa). The weight of the stringers was neglected. These results indicate that the dead load stresses are of the same order of magnitude as the "extreme" live load stresses. Furthermore, the dead load stresses for the exodermic deck construction are considerably less than those for the original construction. Thus, in most realistic situations, the total stresses (dead load plus live load) for the exodermic deck construction will be no larger, and smaller in many cases, than those for the more traditional, composite construction.

The following conclusions can be drawn concerning comparisons of traditional composite deck and exodermic deck construction:

1. Exodermic deck designed in accordance with the present AASHTO specification for composite concrete construction produces good agreement with actual strains measured in a full size bridge.

2. The detailing of exodermic deck appears to make composite action with framing members dependably achievable. Composite behavior was verified for an

effective deck width meeting the AASHTO design rules, with "t" equal to the full depth of the deck.

3. Stringers composite with exodermic decks are subject to slightly larger live load stresses, but smaller dead load stresses than would result for a full depth concrete deck on the same steel members.

It is the belief of the authors that if life cycle cost is the criterion for evaluation of alternatives, exodermic bridge deck will be at the top of the list. LESS is MORE - Less stress means more - longer fatigue life and more - greater serviceability.

Future improvements in fabrication technique are expected to bring exodermic to the top of the list of prefabricated bridge decks from the viewpoint of first cost as well as life cycle cost. The creative input of designers, manufacturers, and contractors cannot be predicted, but if owners ask the right questions, the construction industry will respond.

DESCRIPTION OF EXODERMIC DECK

Exodermic bridge deck is a design concept that combines steel grid and reinforced concrete in a unique way. It maximizes the use of the compressive strength of concrete and the tensile strength of steel. Based on service load (working stress) design in accordance with the AASHTO rules for filled grid decks, computation is straight forward and maximum stresses are conservative. The composite deck has an effective thickness (t) equal to the overall depth. The deck is strong and very stiff, its section modulus per foot is approximately 250% that of a grid filled with concrete of the same total weight. The deck provides extended fatigue life as a result of the location of the neutral axis in the vicinity of the welds and stress-raisers in the steel grid, as shown in Figure 1.

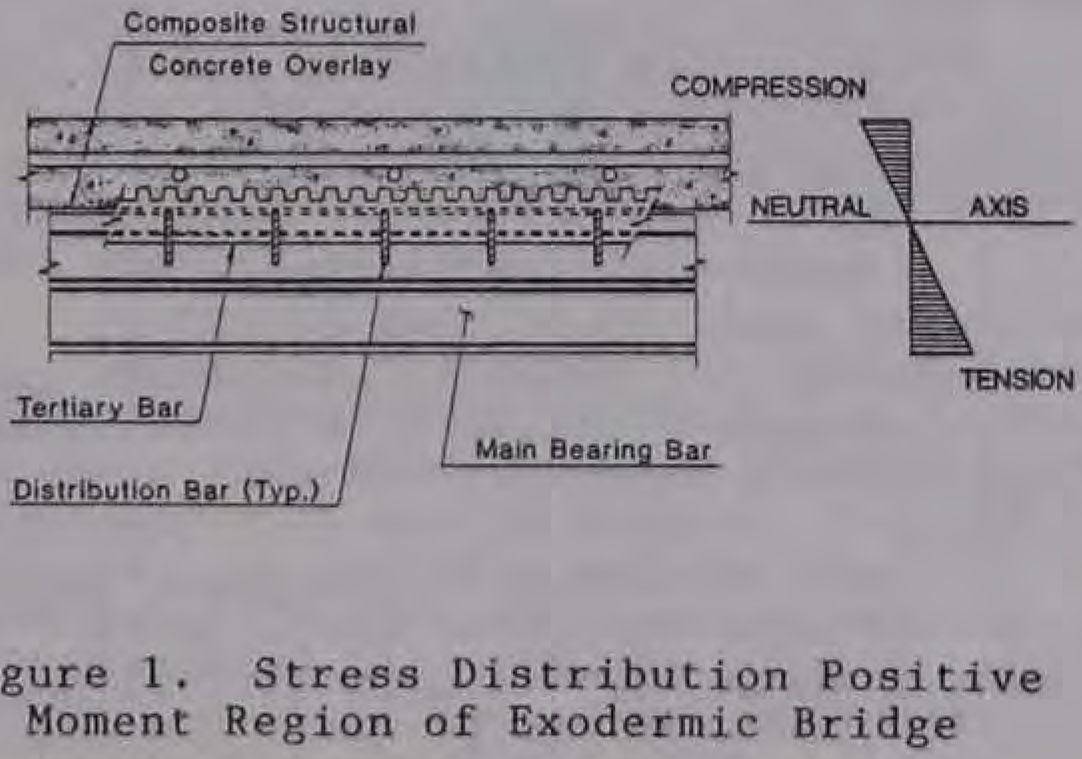

Figure 1. Stress Distribution Positive Moment Region of Exodermic Bridge

The deck comprises a reinforced concrete component on top of, and bonded to, a welded steel grid component, as shown in Figure 2. The dimensions and properties of each component of the deck are selected for the specific bridge by the design engineer.

The design is composite within itself and can be made composite with most types of existing or new bridge framing systems. The concrete component embeds a two-way web of epoxy-coated reinforcing bars and between 1/2 and 3/4 inches (13 and 19 mm) of the tertiary bars of the coated steel grid. Vertical studs welded to the tertiary bars of the steel grid are also embedded in the concrete component of the deck.

Horizontal shear transfer is developed through the partial embedment of the tertiary bars, in conjunction with the vertical studs, as shown in Figure 3.

All steel used in the design may be protected from corrosion to extend the useful life of the individual components to match the extended fatigue life afforded by the design. Embedded bar reinforcement must be fusion bonded epoxy coated to provide such protection.

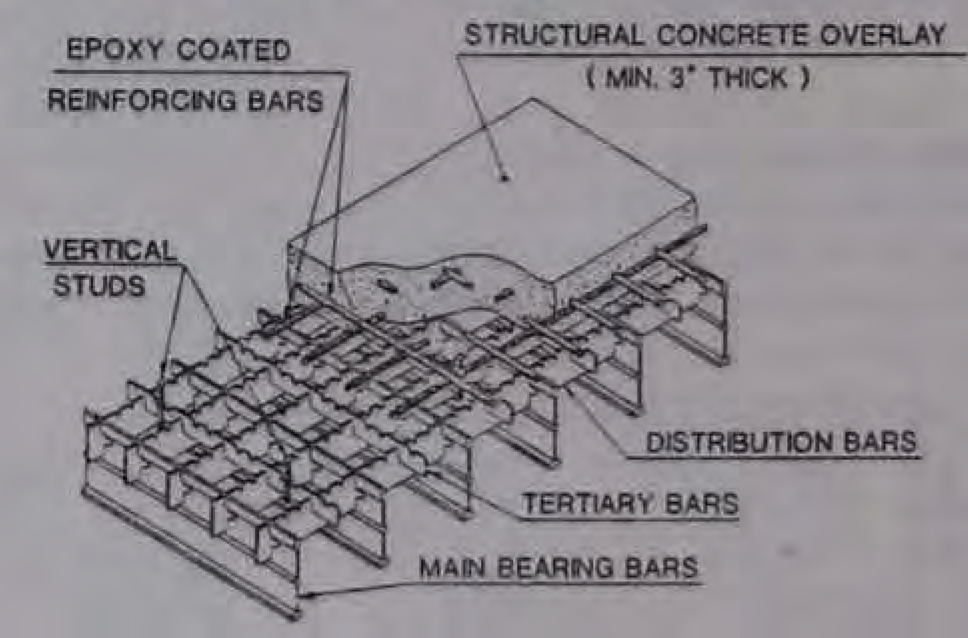

FIG. 2

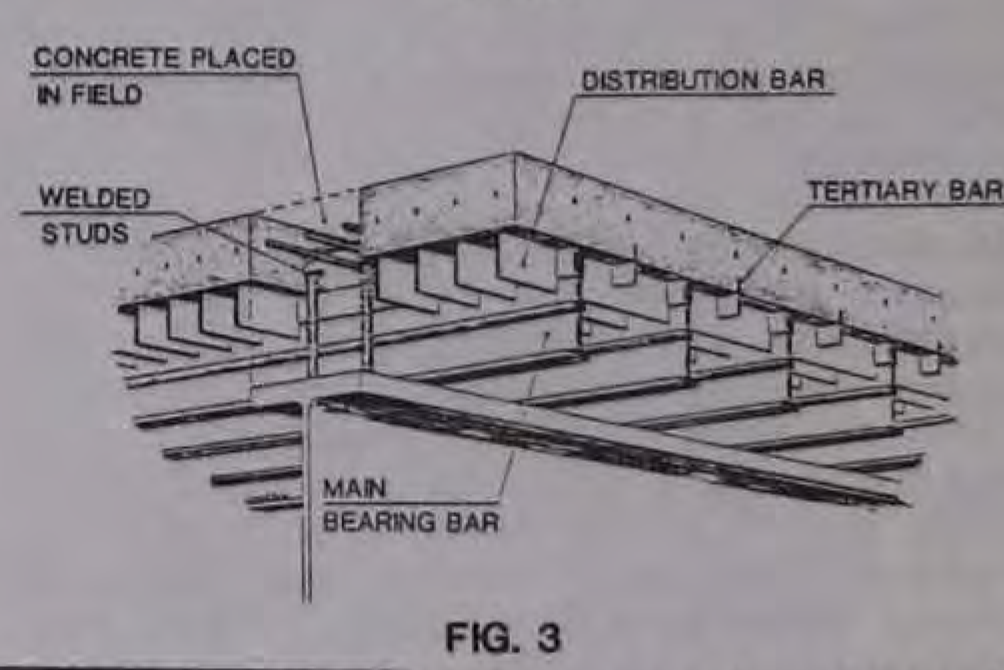

FIG. 3

There are at present several national companies, which meet any test of independent competing entities, licensed to manufacture this bridge deck anywhere in the US or Canada. This fact guarantees competitive pricing of exodermic bridge deck.

FIELD INSTRUMENTATION OF A DOUBLE CELL CONCRETE BOX CULVERT

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SYNOPSIS

Earth pressure cells, strain gages, piezometers and horizontal and vertical deflection points were used in instrumenting a full-sized box culvert. Data generated are to be used in reevaluating AASHTO design provisions and verifying FE models.

INTRODUCTION

Reinforced concrete box culverts (RCBC) are widely used in Nebraska and across the U.S. to provide safe and economical drainageways and as bridge replacements. In the State of Nebraska alone, the Nebraska Department of Roads (NDOR) constructs more than $2.5 million worth of box culverts annually. Enhancement of the design criteria for these structures can result in appreciable savings in taxpayers' money nationwide. Development of new mathematical models which utilize the computer's ability to perform numerical solutions in a rapid and efficient manner leads to a completely new look at the problem of culvert soil-structure interaction. In particular, the finite element (FE) method has proven to be extremely powerful in treating a problem of such complexity. More realistic and uniform safety factors can be realized by using these models in design of RCBC, while keeping the cost to a minimum. One of the most important aspects of analytical modeling, however, is that of obtaining the necessary accurate field data for use in verifying the mathematical model.

The load factors used in the design of RCBC are often based on the 1983 American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications and subsequent Interims (1). The relevant AASHTO provisions have recently become the subject of a reevaluation (5). The authors of this paper compared the AASHTO recommended soil pressures to field measurements and to theoretically predicted values obtained through the use of FE analysis employing the CANDE-1980 program. They concluded that the AASHTO values can very well be on the unconservative side, especially with respect to the lateral earth pressures acting on the side walls of the culvert. AASHTO's provisions concerning live load distribution through fill also came under scrutiny by the authors and they were found to lead to inconsistencies in evaluating the distribution of live loads on the top slab of the box, and therefore their effects on the final design and resulting cost of the structure.

The culvert research project, currently underway at the University of Nebraska-Lincoln (UNL), is sponsored by the NDOR and the Federal Highway Administration (FHWA). The research is expected to generate valuable data that will be useful both in verifying different computer models and in providing documentation for revising the current AASHTO recommended soil and traffic loads. The project consists of instrumenting and testing a functional double cell box culvert (see Fig. 1) under dead and live loads.

The experimental work will be accompanied by an analytical effort using CANDE. Material properties needed for the computer model are being determined by NDOR and UNL. In addition to laboratory testing, dilatometer (DMT) and cone (CPT) soundings are being conducted at the site.

INSTRUMENTATION

Durability, redundancy, and accuracy are the three main aspects of the instrumentation philosophy adopted for the project. It was felt that the success of the instrumentation, and therefore the entire project would be highly dependent on the extent to which these three features were satisfied. Actual instrument selection was a compromise between economy and practicality. The rationale used in selecting instrument type, number, and location will be discussed in the following paragraphs.

Earth Pressure Cells

The most important item of all the instrumentation is the earth pressure cells used to measure normal soil pressures on the faces of the structure. Previous research (2,4) has indicated that it is very difficult to obtain highly accurate data from earth pressure cell measurements (realistically, ± 20 to 40 percent). Thus, a great deal of time was devoted to finding a commercially available cell suitable for this project. Six desirable features identified for the pressure cell were as follows: 1) The cell reading should be environmentally stable; 2) The cell should be very robust and be able to withstand the punishment associated with typical culvert construction; 3) The hysteresis effects should be sufficiently low to allow repeated application of live loading; 4) The cell should have an active diameter of sufficient size so that soil grain size does not appreciably affect the readings, consequently the ratio of the active cell diameter to maximum soil grain size should be a minimum of 50; 5) In order to avoid the effect of soil arching near the face of the cell, the ratio of the maximum central deflection to the active diameter should be limited to 1/5000 (4); and 6) The cell should not be excessively costly so that a large number could be employed.

The cell which was felt to maximize the above desired characteristics is a vibrating-wire pressure cell, specially designed for this project and supplied by Geonor, Inc. Vibrating-wire cells in general are known for their inherent stability and robustness. Twenty-eight cells were placed in steel boxes. The boxes were mounted in their designated locations along the exterior faces of the culvert walls and slabs. The boxes provide a smooth transition surface from the culvert face to the cell diaphragm and allow for full recovery of the cells upon completion of the project. Recovery will permit a check on long-term zero drift and allow for reuse of the cells.

Fourteen of the twenty-eight cells were distributed in a grid pattern on the top slab to allow the determination of the distributions through soil of an AASHTO H-15 truck loading. The rest of the cells were placed around the culvert in sufficient quantity to allow for redundancy. The cell layout is shown in Fig. 1.

Reinforcing Steel Strain Gages

The data generated by strain gages will provide some degree of verification of the pressure cell data. In addition, the data will allow indirect determination of the dragdown forces on the culvert side walls by measuring the change in axial forces along the walls. Direct measurement of these shearing forces is difficult if not impossible with currently available instruments. Because of the high cost associated with concrete embedment strain gages, it was decided to use a large number of strain gages mounted on the reinforcing steel. The gage type selected is a surface mounted, spot welded, vibrating-wire strain gage, supplied by Slope Indicator Company (SINCO). These gages are also known for their robustness and long-term stability. Additionally, their electronic circuitry is fully compatible with that of the earth pressure cells, thus simplifying data acquisition. Forty strain gages were placed in pairs at specified locations around the culvert to enable moment and thrust determinations to be made. The layout of the strain gages is illustrated in Fig. 2.

Piezometers

Since it was necessary to obtain effective rather than total boundary soil pressures, six piezometers of the highly reliable vibrating-wire type were installed on the sides and bottom of the culvert to measure any hydrostatic pressures. Locations of the piezometers are shown in Fig. 1.

Settlement Observations

Measurement of the embankment settlement is intended to provide information on the soil arching effects in the soil prism directly above the culvert. In order to determine the relative settlements between the embankment and the soil prism above the culvert, three sets of settlement observation points were installed. These consist of 3/4-in steel pipe welded to steel plate and encased in free-fitting plastic pipe over full length of burial. Each of the two outermost sets consist of lower, intermediate, and upper points (Fig. 3.a). The lower and intermediate points will establish the settlement of the embankment adjacent to the culvert while the intermediate and upper points will establish the settlements from the top of the culvert to the ground surface. The point located over the top of the culvert will provide information on the compression of the embankment directly over the culvert. This point, as well as the upper points of the other two sets, is established 2.5 ft below the ground surface to minimize seasonal surface fluctuations. Second order optical leveling techniques (3) are being employed to monitor the vertical movements.

Longitudinal Deformation Measurements

The purpose of the longitudinal deformation measurements is to monitor longitudinal settlement and extension in the bottom slab. The haunch at the intersection of the middle wall and the bottom slab was chosen as the most stable reference line. A total of nine eye-bolts were permanently inserted in the concrete at an average spacing of 12 ft (Fig. 3.b). Elevation measurements are performed using second order optical leveling with distances between notches on the bolts measured on a regular basis using a surveyor's steel tape. The tape is fully supported during measurements and temperature corrections are applied to measured values. A single set of dedicated instruments and the same personnel are used for all survey work.

Transverse Deflection Measurements

Transverse deflection measurements are to be used primarily to supplement strain gage data. Relative deflection measurements using a tape-extensometer will provide the required information. Fig. 3.a shows the locations of these reference points.

Automated Data Acquisition

Due to the large number of gages used, the time required to take a complete set of readings by hand would have been considerable. In order to reduce this requirement and to meet the time constraints imposed by construction, an automated data acquisition system was devised. The signals from the various gages were transmitted to a single 64-channel processing unit (multiplexer) and were then relayed through a serial cable connection to a personal computer at the site. Data were printed out at the site for immediate review and also saved on a floppy disk for later analysis. Electricity is supplied by a portable gasoline powered motor generator. Multiplexer and computer equipment are transported to the site and operated from the back of a station wagon.

CONCLUSION

It is felt by all those involved in this project that the instrumentation is performing satisfactorily. All instruments, except two pressure cells in the bottom slab, appear to be functioning properly. One modification that is contemplated in any future field instrumentation of reinforcing steel is to use weldable resistance strain gages. It is felt that the problems of voltage induction, signal attenuation, water proofing and protection from damage during concreting operations might be more easily overcome than problems caused by properly mounting the relatively large but robust vibrating-wire gages on small circular bars. It is hoped that a small size, economical vibrating-wire gage will soon be developed for use on reinforcing steel.

The data reduction and analysis are currently underway at UNL. Final conclusions on the performance of the culvert and its impact on the relevant AASHTO provisions will be published at a later date. An example of some early findings are shown in Fig. 4.

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DISCLAIMER

FIELD INSTRUMENTATION OF THE I-295 CABLE STAYED BOX GIRDER BRIDGE

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SYNOPSIS

The I-295 Bridge over the James River near Richmond, Virginia has a twin box girder precast posttensioned design. The main span and the two adjacent approach spans are supported with a single plane of cable stays which are attached to the box girders of the bridge through a unique prestressed concrete delta frame. The Virginia Transportation Research Council and The University of Virginia have developed and implemented an instrumentation plan to measure static strains in the bridge components and the stay cables during construction and subsequently during service.

INTRODUCTION

Two innovations in bridge construction recently utilized in this country are cable stayed and segmentally prestressed concrete box girder bridges. Both types of bridges have in common the efficient use of high-strength materials. Moreover, the use of cantilever construction and cable stays in conjunction with segmentally post-tensioned box girder bridges has been found to be competitive for spans on the order of 1000 feet.

The analysis and design of cable stayed and segmentally erected posttensioned bridges are quite complex. Cable stayed bridges typically are continuous over two or more spans and the numerous connections between the stay cables and the superstucture girder lead to a highly indeterminate structure. Further complexity is introduced by the use of a prestressed segmentally constructed concrete bridge deck in conjunction with the cable stays. Time dependent factors such as creep and shrinkage of the conctrete and relaxation of the prestrsssing steel add further to the complexity. In the I-295 bridge delta frames have been introduced to transfer stay forces from the cables to the deck segments. These are new design features whose behavior has not been verified by field measurements or previous experience.

Considerable effort has been expended in developing suitable techniques for the analysis of segmentally prestressed and cable stayed bridges [1]. Field studies have been conducted on several segmentally erected prestressed bridges but none of the studies completed to date have involved cable stayed segmental bridges. Thus, additional evaluation of field behavior is desirable especially when new design features such as delta frames are being used.

DESCRIPTION OF BRIDGE

The I-295 bridge over the James River between Chesterfield County and Henrico County, Virginia will be a highly complex structure. The river crossing will be the 630 ft. middle span of a seven-span continuous structure consisting of twin precast, segmentally post tensioned concrete box girders.

The middle five spans of the bridge will be supported by multiple cable stays arranged in a single plane harp configuration from pylons located at either end of the main span. Cable stay forces will be transferred to the twin box girders through precast delta frame assemblies. The middle span will be constructed as cantilevers. Posttensioning will consist of two parts. Temporary bars will be placed for tensioning during construction. Permanent strand posttensioning, located inside the boxes, will be installed subsequently.

Because of the complexity of the design, monitoring of the bridge during construction and, later, during the in-service phase appears warranted. Such a study should provide valuable information regarding the behavior of this type of structure during and after construction and should significantly benefit subsequent designs of similar structures.

OBJECTIVES

The overall objective of this study is to evaluate the behavior and response of the cable stayed bridge which carries I-295 over the James River near Richmond, Virginia, during the construction phase and, subsequently, to monitor the in-service behavior over an extended period. This study seeks to address a number of questions regarding bridge response for which only limited experimental data are available. Specific objectives of the investigation include:

- to determine the live load induced stress range and spectrum in the cable stays;
- to evaluate stresses and stress resultants in typical deck segments as a means of predicting load carrying mechanisms;
- to evaluate the performance of the delta frame assemblies in distributing cable stay forces to the supported box segments;
- to obtain thermal gradient data for the box girder segments, the pylons and cable stays;
- to collect sufficient strain data during the construction phase to permit an evaluation of such factors as shear lag in the deck segments during and after stay tensioning, and to monitor overall force distributions in the critical elements of the bridge during construction.

INSTRUMENTATION

Instrumentation proposed for this study consists of an extensive array of electrical resistance strain gages and thermocouples to be installed in the main span (span 16) on both the north and south sides of the bridge in both north and south bound lanes. These strain gages are being installed in three deck segments of the main span on the south side of the centerline, in two segments of the pylon on the south side, and on three stay cables associated with each pylon. In addition to the electrical resistance strain gages, mechanical strain gage locations are being installed on twenty deck segments, four pier segments and both pylons on both the north and south sides of the bridge.

Stay Cable Instrumentation

A major concern in the long term performance of a cable stayed bridge is the satisfactory fatigue performance of the stay cables. As designed, the stay cable system in this bridge utilizes between 72 and 90 high-strength steel, seven-wire strands per stay. The tendons will be encased in polyethylene pipe and grouted with a portland cement mortar to provide corrosion protection. Several approaches for obtaining strain data from the stay cables are possible, but all have some limitations. Since the strains in the stay cables are the quantity of interest, direct instrumentation on the stays appeared to be the most desirable approach.

It was decided to mount foil resistance strain gages directly on the wires in the strand. Stays S2, S7, and S13 on each side of the bridge were selected for instrumentation. To provide for a reliable determination of average strains in the cable and to show distribution of strains throughout the cable cross section, eight gages were placed on each stay cable. All gages will be wired independently in the event some of the gages are lost during subsequent tensioning and grouting. Lead wires from all of the gages will exit through the expansion joint in the polyethylene pipe and from there either to a data acquisition system on the south side, or to switch and balance units and strain indicators to be read manually on the north side. Further details of this stay cable instrumentation are provided elsewhere [2].

Deck Segment Instrumentation

Primary deformation response of the main span and approach span girders of the bridge is expected to include bending within the vertical plane of the bridge and torsional twist of the sections. An additional response of interest will be the transfer of stay cable forces through the delta frames to the main deck segments. Electrical resistance strain gages will be installed in three deck segments of the main span on the south cantilever.

Based on a review of previous work, contact with a number of researchers, and laboratory experiments to verify and evaluate proposed methods, it was decided to mount the foil strain gages on four ft. lengths of reinforcing bar and to imbed these bars in the instrumented deck segments prior to casting. Up to nineteen strain gages are being placed throughout the cross section of each of the three sections to be instrumented. A sketch of these locations is shown in Fig. 2. Three bar "rosettes" are placed in the locations as indicated by "R".

Using a similar scheme, two of the pylon segments on the south side side were instrumented with electrical resistance strain gages. One location was the cast-in-place segment at deck level and the second was a precast segment just below stay 7 approximately 65 feet above deck level.

The strain-gaged reinforcing bars in both the deck segments and the pylon segments were placed in the reinforcing cages during the time between cage construction and placement in the precasting forms. The gaged bars were carefully wired into place against the reinforcing cage and lead wires run along the reinforcing to a block out in the wall of the box which is near the data acquisition system scanners.

Twenty deck sections were instrumented with mechanical gages, with two segments at each section. At each segment, five gage points were installed transversely across the deck and three across the lower flange inside the box. The gage points consisted of a pair of brass plugs, each with a small hole drilled to accept the points of the mechanical gage. The gage points were imbedded in the concrete after casting. The gage length on all of the gage locations was approximately 10 inches but a Whittemore gage was used on the north side segmnents while a Demec gage was utilized on the south side.

Similar mechanical gage installations were placed on eight pier segments, one each on four piers closest to the main span on either side of the river. These were located at the lowest accessible segment for ease in recording. Two segments in each of the two pylons have also been instrumented with the same mechanical gages, one segment just above the footing and one just above deck level.

Thermocouple Instrumentation

Thermal data is being obtained from thermocouples located in both boxes of a single deck section, from thermocouples in a pylon segment, from thermocouples on a stay cable and from surface temperature measurements taken when strain readings from the mechanical gages are recorded. The deck segments with the thermocouples are located on the south side and this data will also be automatically recorded using the data acquisition system.

DATA ACQUISITION AND ANALYSIS

The data acquisition system (DAS) was selected to be a unit that would be serviceable in the field, one that would fit the needs of the instrumentation layout and that had good software for data analysis. After considering a number of alternatives, a system manufactured by the John Fluke Company was selected. The system uses a Helios main controller to communicate with a number of individual remote scanning units each of which can be located near the instrumented sections. The data is shipped back to the main unit in digital form. Permanent data storage is accomplished by connecting the control unit to a personal computer. Communication between the personal computer and the DAS is accomplished using data acquisition software supplied by Fluke. For environmental control and security, the computer, a Compaq portable, will be located in the offices of the Virginia Department of Transportation in Richmond and will communicate with the DAS by telephone. The computer has been programmed to automatically dial up the DAS and download the data to a diskette which will be regularly changed by VDOT personnel.

All of the electrical resistance strain gages and the thermocouples on the south side will be automatically recorded using the data acquisition system. All of the mechanical strain data must be recorded manually. This requires an almost continuous presence of research staff on the bridge, especially when significant changes are taking place such as cable tensioning or segment erection. The manually recorded data is then placed on a computer spreadsheet file for reduction, analysis and plotting.

One of the major objectives during this investigation has been the appropriate analysis and interpretation of the large amount of data that is being compiled. All data will require some analysis, but proper interpretation of the measured strain responses in the box girder and pylon sections and the long term strain variations measured in the stay cables will require especially careful reduction and evaluation.

The longitudinal strains measured by the strain gages in the box sections will allow a reasonably complete estimate of the strain field at these sections. In particular, axial compression and bending moments acting in the double box may be estimated from the strain data. The strains recorded from both the electrical resistance and mechanical strain gages should allow the shear lag present in the box sections to be identified. The rosettes located in the webs and flanges of the box sections will provide information regarding the magnitude of shear stresses in the walls of the segments which will allow extimates of torsion in the box to be obtained. Design calculations for the bridge have been made available to the investigators and this information will be used for comparison with the measured field response to verify design assumptions and to provide additional insight and understanding into the behavior of cable stayed bridges.

PRESENT STATUS OF INVESTIGATION

As of August 1988, all of the approach spans on the north side are complete, all piers have been erected, the pylon on the north side has been completed and thirteen cantilever spans on the north side have been erected together with five of the cable stays which have been placed and tensioned. On the south side, all approach spans on one lane are complete but considerable work remains before the cantilever spans can be started.

On the north side, mechanical gage points have been installed on all segments that have been cast and on all piers and pylon segments. Only one box segment scheduled for instrumentation with electrical resistance strain gages has been cast and it has been successfully instrumented. Strain gages have been installed on three of the cable stays on the north side. Thermocouples have been placed in a pylon segment.

Some preliminary strain data has been recorded from the gages on the stay cables, and temperature data is beginning to be assembled from the thermocouples. Most of the data thus far has consisted of strains obtained from the mechanical gages on the deck segments. Efforts have been concentrated on the north side cantilever span. Zero readings on these gages were taken in the casting yard a few days after casting and as of this date, with only a portion of the north cantilever constructed, maximum strains recorded on the deck segments are on the order of 500 microinches per inch. Trends in the data seem to be consistent with the stages of construction and no unusual changes have been observed.

Although gage installation and data acquisition are on schedule and proceeding smoothly now, a variety of unforseen problems were encountered in the early phase of the study. These difficulties have been resolved and it is expected that valuable and accurate data will be obtained.

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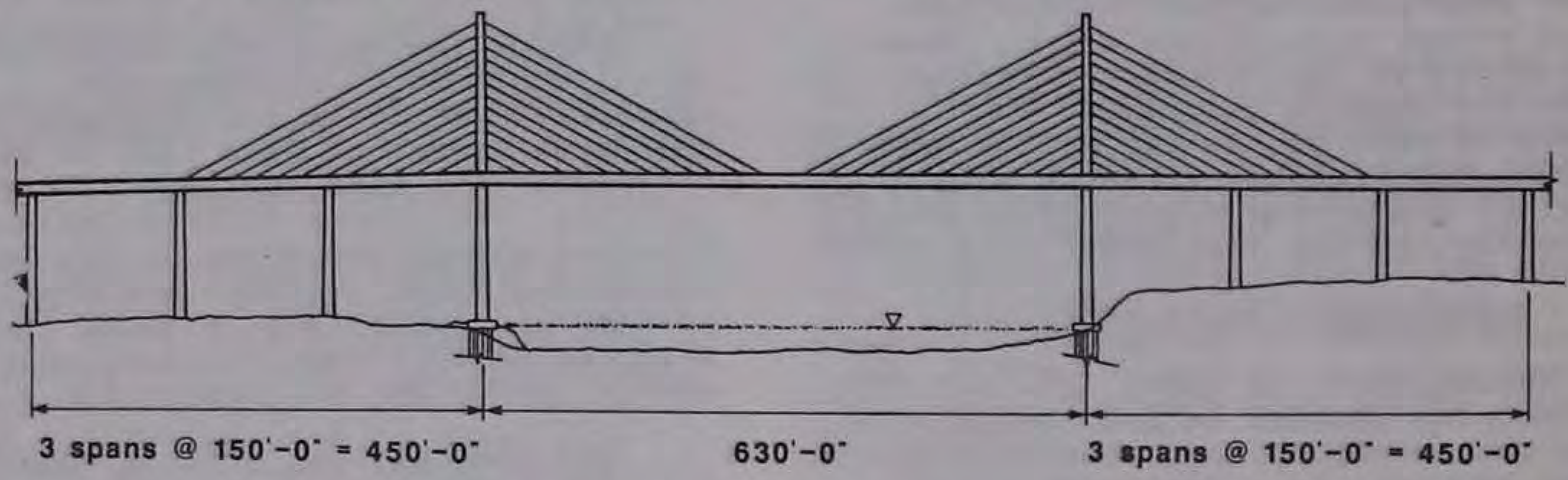

Figure 1. Partial Elevation of the I-295 James River Bridge

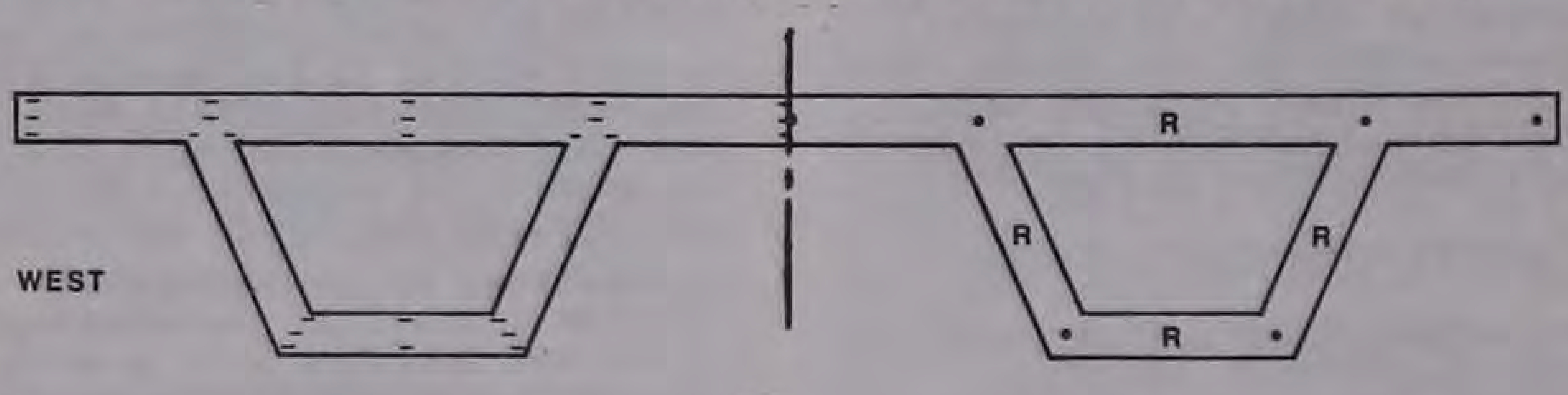

Figure 2. Strain Gage and Thermocouple Instrumentation of Box Girders

LATERAL LOAD REDUCTION ON BRIDGE ABUTMENT WALLS

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SYNOPSIS

A technique to lower or eliminate lateral earth pressures on abutment walls is presented. The technique uses geotextile reinforced soil and a small gap between the geotextile face and the abutment wall. The goal of the research is to lower the long term maintenance cost of the bridge approach and the expansion joint. It has been shown that corrugated cardboard can produce a controlled collapse to form the gap. Deformations recorded were similar to those produced in a similar embankment without a gap. Study of the long term behavior is ongoing.

INTRODUCTION

A laboratory and field research program is being conducted by the Wyoming Highway Department (WHD) and the Department of Civil Engineering at the University of Wyoming to reduce the lateral loads imposed by the soil on bridge abutment walls. The goal of this research is to lower the long-term maintenance costs required to repair or replace embankment fills and abutment walls that have moved due to excessive deformation.

Embankment deformation can be caused by several mechanisms, including lateral creep and shear failures. Lateral creep causes the surface to displace vertically, creating a bump at the pile supported abutment. Small rotational shear slides can occur through the embankment fill, under the abutment and into the exposed slope under the bridge. Both failure modes can cause the abutment to move towards the bridge, closing of the expansion joints and inducing higher stresses on the bridge components [1]. The cost to replace the fill, the approach slab and expansion joint is estimated at $25,000 per approach and an additional $1,600 annually is required to level a settled approach slab to the bridge deck [2].

The WHD currently uses geotextiles in the fill to provide tensile strength to reduce the potential for large shears. A simple technique to prestress the fabric so that it will lower the lateral pressure on the abutment wall and reduce the likelihood of its movement is explored.

CURRENT CONSTRUCTION PRACTICE AND PROPOSAL

The WHD construction procedure for geotextile reinforced embankments is similar for both new and reconstruction. The bottom layer of geotextile is placed on a prepared base and an extra length of fabric about five feet long is temporarily draped over the abutment and wing walls. One-half of the first lift thickness is placed on the geotextile. Additional fill material is then placed along the face of the wall to the final lift height and extending one foot out from it. The 5 foot lap of fabric is wrapped over this additional fill and the end (located at the half-lift level) is covered with the rest of the lift. This anchors the end with the friction developed by the half lift thickness. This locking action increases as subsequent lifts are constructed.

Since the fabric is placed against the wall and covered, it does not have an opportunity to be tightly stretched, a condition required for it to effectively use its strength. The soil is under K_o (or at-rest) conditions, hence the force on the wall is high.

If the wall moves, two actions combine to reduce the lateral force. The soil progresses from an "at-rest" condition toward an "active" condition, resulting in a load reduction of 25 to 50%. Secondly, the fabric becomes stretched and thus is able to provide tensile strength to the fill. This provides additional support to the soil and may eventually reduce the load to zero if sufficient deformation occurs.

By allowing the soil to deform before the construction is completed, the lateral load can be reduced without causing the existing wall to move. This deformation can be achieved by creating a temporary gap between the abutment wall and the face of the fabric during construction. This concept was presented to the WHD and a field project was initiated.

The construction of the gap raises two questions: how much gap is needed to achieve a substantial load reduction and how can the gap be developed? These topics were investigated in a series of laboratory tests performed at the University of Wyoming. Field testing of four prototype systems is presently underway.

LABORATORY TESTS FOR LATERAL DEFORMATION

A box was constructed to test a two foot thick lift of soil, i.e., a full lift height. Three side walls are fixed while the fourth wall is mounted on roller bearings and is held in place by an MTS load actuator. A uniform vertical pressure or surcharge is developed using an air bladder.

The first set of tests were conducted to investigate the reduction in lateral load caused by lateral movement of the geotextile face. Each test used a wrapped layer of soil loaded under different vertical pressures. The wall was then moved away from the soil using the stroke control and the horizontal load was measured. The soil started in K_o conditions and then decreased rapidly with deformation as shown in Fig. 1. It was found that the lateral load was reduced by 90% with less than one inch of lateral deformation.

The second set of tests required the movement of the wall away from the geotextile reinforced face while the soil was still unloaded. The top surface was

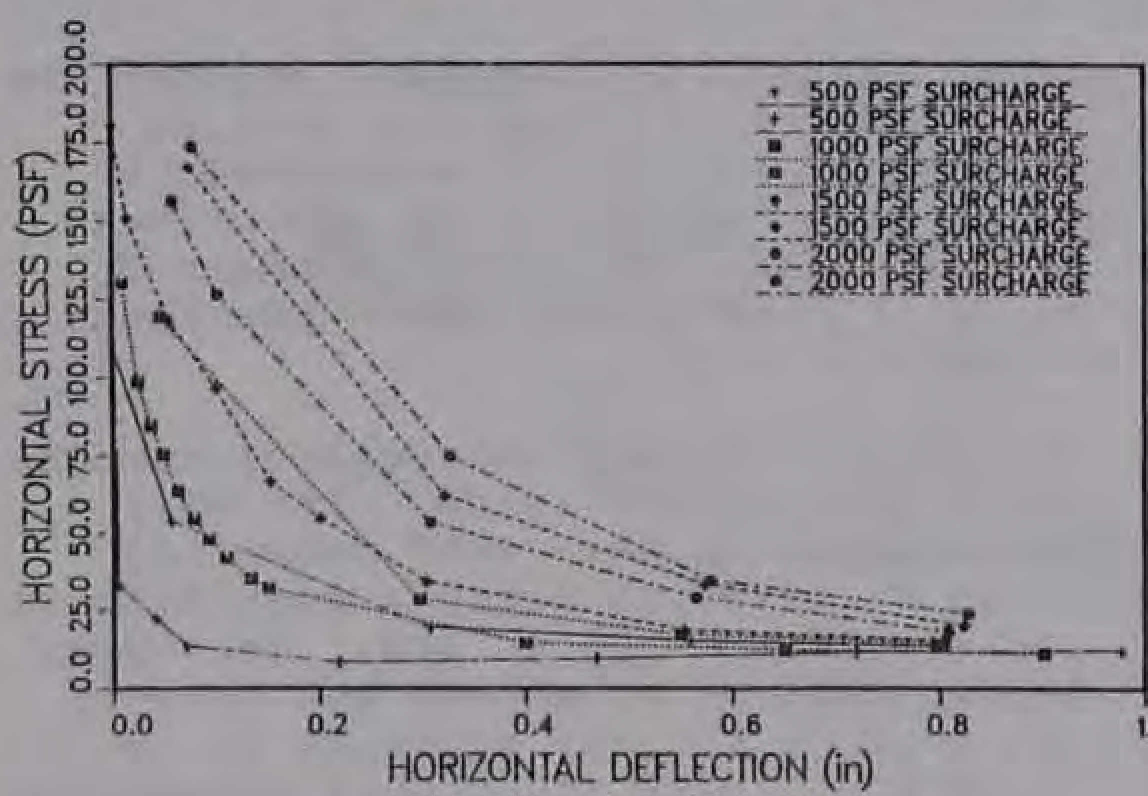

Fig. 1 - Stress Reduction due to Wall Deflection

then loaded and the additional lateral deformation was determined. It appears that once a stable face configuration is developed, additional loading causes only small lateral deformations (< 1/2 inch), at least under short term loading.

Several construction procedures were considered to develop the void between the reinforced soil and the wall. Styrofoam was initially considered, but its crushing strength was much higher than the pressure developed by the soil. Corrugated cardboard was finally selected since the dry strength is high relative to the expected constructed stresses (13 to 18 psi) while its wet strength is low (1.0 to 1.4 psi) during post construction wetting. The wet cardboard collapses completely when an even small shear load is applied. Secondly, being an organic material, it will totally decompose with time, insuring that the deformation will equal the full initial thickness of the cardboard.

PROTOTYPE SYSTEM

The project site for the prototype system is located on Interstate 80 approximately 30 miles east of Laramie, Wyoming. The divided interstate bridges cross over a double section of Union Pacific Railroad track. The reconstruction occurred from June to August, 1987 and consisted of replacing both bridge decks and the four approach slabs and embankments (Fig. 2).

Each embankment was prepared with a different reinforcement scheme. Three of the embankments were totally excavated and replaced with a 3/4 inch crushed rock "select" backfill. A different geotextile support technique was used at each of these embankments. One was constructed with the geotextile against the wall, another used a two inch corrugated cardboard material to form the gap (Fig. 3), and the third used a temporary plywood form designed by the contractor. The third embankment had an initial gap of 6 inches and the forms were removed after each lift had been placed. The fourth embankment was left in place except for the installation of a subdrain at the abutment footing.

INSTRUMENTATION

The embankments were extensively instrumented. Sixteen borings were completed, as shown in Fig. 2, and were instrumented with inclinometers to measure deviations from verticality with time and Sondex tubing to measure changes in fill height with time. The Sondex probe uses magnetic induction to determine relative movement between wire loops mounted on the flexible tubing. Six Sondex tubes were installed horizontally in the three reinforced embankments to measure lateral movement.

Twenty-two total pressure cells were installed in the two embankments in which the geotextile was installed against the wall (nonvoided) and with the cardboard (voided). Six of these were placed in the fills to measure vertical stress and the remaining sixteen were mounted vertically on the abutment walls to determine the lateral pressure developed by the soil. These cells are very sensitive to the placement bedding conditions. It was difficult to provide uniform bedding for the cells mounted in the abutments, therefore the trends shown by this data are more significant than the absolute values.

FIELD DATA AND RESULTS

The results from the inclinometer data indicate that less than 1.0 inch of horizontal movement has taken place in all but three of the tubes,of which two are located on the fill side slope. Similarly, the vertical Sondex readings in most of these holes show surface movements of less than 1.0 inch. All movements greater than 1.0 inch occurred in the unreinforced fill on either the face early after construction or on the side slopes away from the abutment wall.

Six Sondex tubes were placed horizontally in the middle of the second lifts in the three reinforced embankments. Approximately 0.5 inch of movement toward the abutment wall has been measured in both of the tubes in the cardboard voided fill and in one of the tubes in the six inch gap fill. This is about the accuracy of reading the Sondex horizontally, hence these should be considered only trends and not absolute movements. The other tubes have not indicated movement toward the walls.

Six total pressure cells have been installed to measure vertical pressure. Four of these indicate a measured stress that agrees with the theoretical vertical stress at each point. Two have been consistently measuring above the theoretical vertical stress. The recorded stresses have been varying considerably on these cells over time, indicating that the loads are still shifting in these embankments.

The remaining 16 cells were mounted vertically on the rear face of the abutment to measure the lateral pressure developed against the walls. Initial calibration of the cells with no applied stress indicated readings of 21.5 to 43.0 psf. Generally, the measured pressures in the cardboard voided embankment extended over this range at depths below the corbel. This indicates that there is a low pressure (approaching zero) developed against that wall. The corbel has a slightly greater pressure due to the nature of the slumping around it.

In the nonvoided abutment, several cells are showing pressures approximately equal to the theoretically computed lateral pressures under K_o (at-rest) conditions. The pressures have shown considerable variation over the past ten months. Cells that started out with pressures around the K_o value apparently decreased over the winter and are generally rising back to K_o conditions again. Several cells that had low initial readings remained low and are beginning to increase.

GENERAL OBSERVATIONS AND CONTINUED STUDY

The data presented above provides only a portion of the picture. Several additional observations are presented below.

1) The contractor modified his backfilling technique to compensate for the instrumentation. A more typical embankment would probably be stiffer than what is being indicated, producing smaller deformations and larger lateral loads.

2) The cardboard void technique appears very constructable. It is sufficiently stiff to compact against, yet soft enough when wetted to collapse. It is important that the cardboard remain dry prior to installation. Therefore, storage in a dry room prior to installation is essential.

3) The lateral loads in the nonvoided embankment decreased from their initial values in August, 1987 to February, 1988 and are increasing again. While thermal effects in the pressure cell may account for some of this, there does appear to be a change in load due to seasonal changes. It is possible that the thermal contraction of the bridge could allow the wall to tilt away from the soil, permitting active soil conditions. Warming of the bridge would cause the wall to move back into the soil, placing the soil in at-rest or even passive conditions. If this were an annual cycle, it is possible that the lateral loads could become large as the soil tended to fill in the space created by the tilting of the wall. This process can be monitored over several years.

SUMMARY

A technique has been presented to allow the soil to undergo large deformations behind a wall, which allows it to reach an active condition and to obtain support from the geotextile. Several different construction procedures were compared. A simple procedure using corrugated cardboard provides a controlled deformation after construction. Both laboratory and field evidence indicate that the lateral loading is reduced using the technique.

Equally important, soil deformations in the voided embankments appear to be comparable to those in which the soil at the unvoided abutment. This indicates that the support for the approach slab is the same even when the load on the abutment face is decreased. Hence the geotextile support is effective.

PROJECT STATUS

The instrumented bridge is being monitored approximately quarterly to observe the long term behavior.

ACKNOWLEDGEMENT

The technical and financial support of the Wyoming Highway Department is greatly appreciated.

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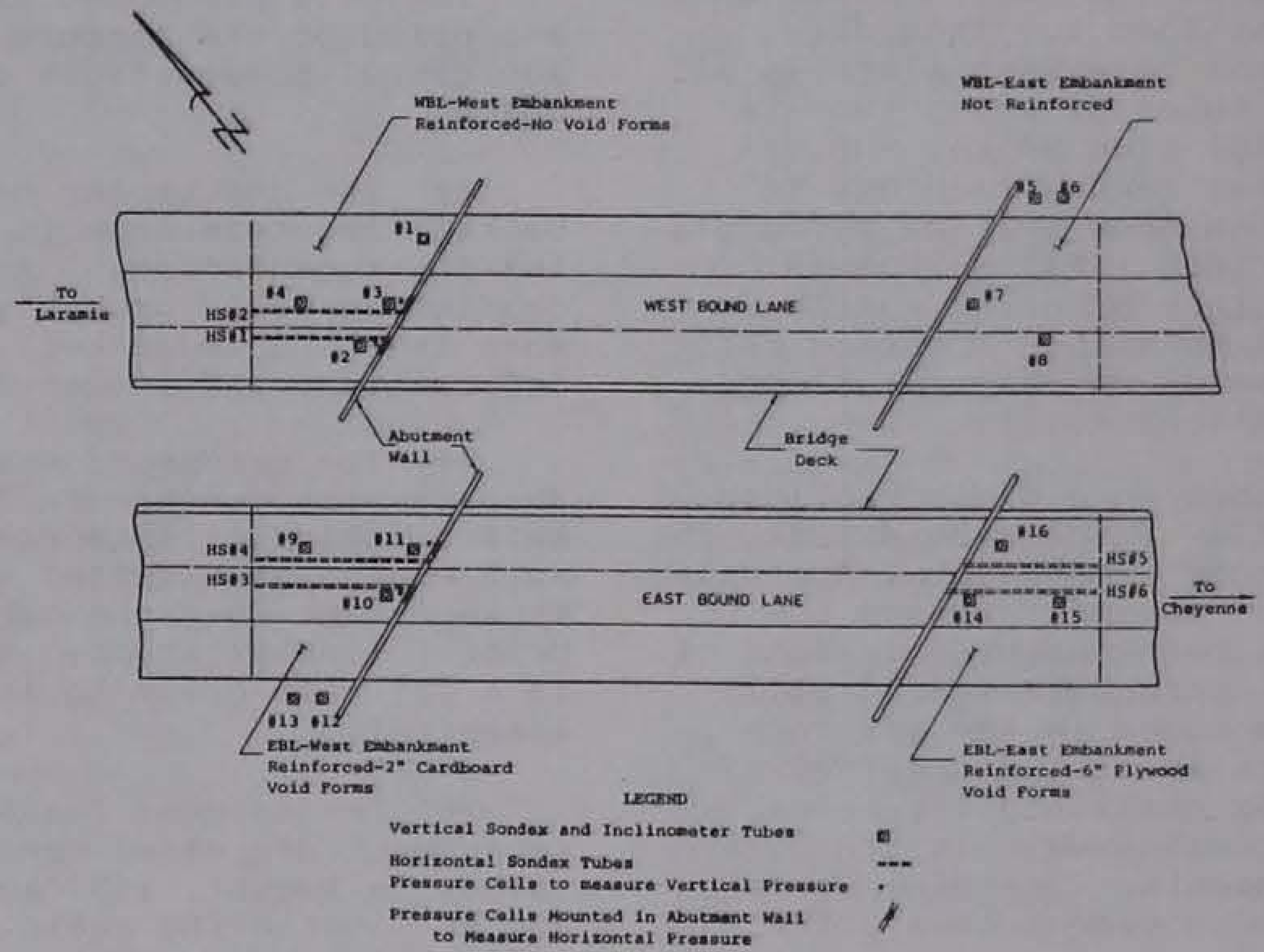

Fig. 2 - Plan View Showing Instrumentation

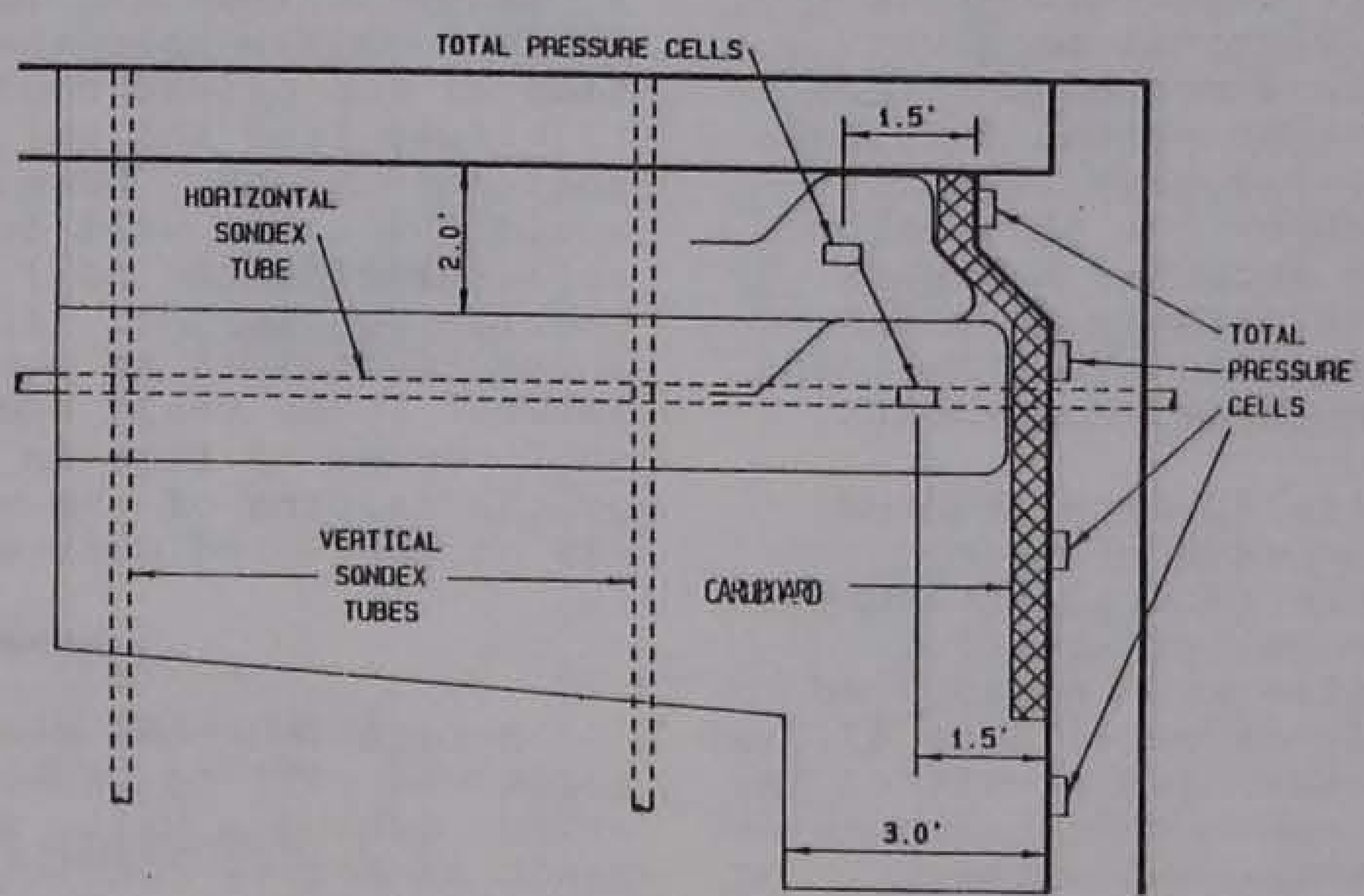

Fig. 3 - Cross Section of Cardboard Voided Fill

PROBABILISTIC BASIS FOR LRFD FOR BRIDGES

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SYNOPSIS

The objectives of the study are to establish the feasibility of basing highway bridge design criteria on probabilistic concepts and to identify the elements necessary for such a development. The probabilistic methods for structural analysis are reviewed. The available load and resistance models are evaluated. The procedure is developed for calculation of load and resistance factors. The structural reliability is used as the measure of adequacy in the selection of design criteria. It is proposed to determine load factors first, using bias factors and coefficients of variation. Then, resistance factors can be calculated to provide for a preselected target safety level.

INTRODUCTION

There are considerable new developments in bridge design and evaluation criteria in the United States. The AASHTO Specification is undergoing extensive changes, with possible introduction of load and resistance factor design (LRFD) format. This paper deals with the development of a probabilistic basis for calculation of load and resistance factors. The work involves identification of the parameters (load and resistance), definition of acceptance criteria (limit states), quantification of variation of the parameters (statistical data) and calibration (calculation of load and resistance factors).

All quantities (except physical and mathematical constants) that enter into engineering calculations involve uncertainties due to randomness of design parameters (eg. loads, material properties), imperfect models or human errors. Because of these uncertainties, absolute reliability is not an attainable goal. Selection of the optimum reliability level is an economical problem. Lower reliability results in frequent failures, while higher reliability requires more initial costs (materials and labor). Therefore, structural reliability is a convenient acceptability criterion in the development of a design code.

Bridge failure is defined as inability to perform the intended function (carry traffic). Bridges can fail in many ways (modes of failure), by cracking, corrosion, excessive deformations, exceeding carrying capacity for shear or bending moment, local or overall buckling, and so on. In the design code, safety is provided by specifying the design capacity larger than the expected loads. The optimum safety reserve can be selected in the probability-based calibration process.

The main elements of the development of load and resistance factors are: reliability analysis, bridge load models, bridge resistance models and calibration procedure.

RELIABILITY ANALYSIS

The safety analysis includes several steps:

1. Identification of limit states (modes of failure), acceptance criteria for performance in terms of deformations, deflections, cracking, collapse and so on.

2. Identification of parameters such as dimensions, material properties, geometrical configurations and formulation of the limit state functions (for each mode of failure).

3. Development of statistical models (distribution functions, means, standard deviations, coeffcients of correlation, time variation) for parameters identified in step 2, using tests data, measurements, surveys, observations, analysis and engineering judgement.

4. Calculation of the reliability or probability of failure for structural members and systems.

It is convenient to measure safety in terms of a reliability index, B, related to the probability of failure, P_F as follows [2],

$$B = - F_N^{-1}(P_F) \qquad (1)$$

where F_N^{-1} = inverse of the standard normal distribution function.

B can be calculated for each mode of failure. The available methods vary with regard to accuracy, computational effort and requirements for input data [4].

In a simple case, safety is determined by two variables, R and Q. R represents the structural resistance and Q load effect. For example, R can be the moment carrying capacity of a girder and Q can be the moment caused by the loads. Due to uncertainties in material properties, dimensions, workmanship, truck weights and occurrence rate, both R and Q are random variables. Therefore, the probability of failure is equal to the probability of R being less than Q. Let Z = R - Q. Z is also a random variable and it represents the safety margin. The reliability index can often be considered as the number of standard deviations from 0 to mean value of Z.

Usually R and Q are functions of other parameters (material properties, geometry, load components, load distribution factors). Limit state function (mathematical formulation for the boundary between safety and failure) may be nonlinear and very complicated. There are efficient numerical procedures developed for calculation of the reliability index in these cases, including Rackwitz and Fiessler procedure, evaluation of the up-crossing rates, Monte Carlo techniques or sampling methods [2].

LOAD MODELS FOR BRIDGES

The major load components of highway bridges are dead load, live load with impact, environmental loads, earth pressure, and abnormal loads. Each load group includes several subcomponents. For time varying loads, the model depends on the considered time interval. The load combination model can be based on Turkstra's rule [2].

Dead load, D, is the gravity load due to the self weight of the structural and nonstructural elements permanently connected to the bridge. Studies done for the Ontario Highway Bridge Design Code (OHBDC) indicate that the coefficient of variation varies for factory made elements (4%), cast-in-place concrete members (8%) and the wearing surface (25%).

Live load, L, covers a range of forces produced by vehicles moving on the bridge. Traditionally, the static and dynamic effects are considered separately. Therefore, in this study, L covers only the static components of forces such as truck weights, braking forces, and centrifugal forces. The dynamic component is denoted by I.

The effect of live load depends on wheel force, wheel geometry (configuration), position of the vehicle on the bridge (transverse and longitudinal), number of vehicles on the bridge (multiple presence), stiffness of the deck (slab), and stiffness of the girders. There is a considerable statistical data on truck weights (axle loads and configurations) gathered in Ontario. Recently, weigh-in-motion (WIM) measurements were performed in many states. Site-specific live load models are the subject of the on-going NCHRP Project 12-28(11).

A comparison shows that the actual live load moments are consistently more than two times larger than the nominal design moments. There are several reasons why the live load should be higher. The first is that many vehicles on the roads weigh more than the design vehicle weight of 72 kips. The second reason is that the live load model represents the mean of the maximum moment which can occur in 50 years. Over such a long period of time, it is quite possible to have heavy vehicles crossing over a bridge. The AASHTO HS20 gives the moment of a single design vehicle. However, the maximum loading is due to multiple trucks, which includes side-by-side and vehicles in the same lane. Thus, over 50 years, there is a strong probability of two extreme vehicles side-by-side in the lane, both of which exceed the AASHTO vehicle.

The major factors affecting the dynamic load on a bridge include surface condition (bumps, potholes), natural frequency of the bridge (span length, stiffness, mass), and dynamics of the vehicle (suspension, shock absorbers). It is practically impossible to predict the percentage of dynamic load contributed by each of these three factors. In addition, the dynamic load effect may be changed by superposition of several trucks (multiple presence) or even by several axles of the same truck.

Traditionally, the impact factor was calculated as a function of the loaded length of the bridge. However, the dynamic bridge tests point to the importance of other factors, such as surface condition and vehicle dynamics. Further study is required to determine the correlation between the dynamic and static portions of live load, in particular for very heavy trucks. Two factors should be noted in assessing dynamic impacts. Tests show impact decreases with truck weight. Also, the chance that all trucks simultaneously have high impacts will be small. Thus, it may be too conservative to use excessive impact values.

Load effect is a resultant of several components. It is unlikely, that all components take their maximum values simultaneously. There are sophisticated load combination techniques available to calculate the distribution of the total load, Q. They involve a considerable numerical effort. However, the parameters of Q can be also calculated using a simple procedure developed by Turkstra and described in [2]. The method is based on observation that governing combination is dominated by only one load component.

RESISTANCE MODELS FOR BRIDGES

It is convenient to consider resistance as a function of three variables, one representing material properties (including strength, modulus of elasticity, cracking stress, chemical composition), second representing fabrication effects (mostly geometry, dimensions, section moduli) and third one representing analysis factor (including uncertainties due to approximate analysis, idealized stress and strain models, support conditions).

Resistance of a bridge depends on resistance of members (girders) and connections, as well as the degree of redundancy. Redundancy is the ability of the structure to continue to function safely in an almost normal manner despite the failure of one of the main load carrying elements. To evaluate the redundancy of a structure, failure modes of the main load-carrying members must be examined to determine the possible secondary (redundant) load paths. These load paths must then be evaluated to determine that there are no weak links that would prevent the development of their full capacity.

Studies can be made of various common types of structures that would simplify the checks for redundancy, e.g. design criteria could be developed for bridge deck slabs which are a key element in the redundancy of stringer bridges. Design criteria can also be developed for checking the ability of the adjacent stringers to carry a "failed" stringer. Assurance of this ability of a structure could reasonably allow the safety reserve of an individual member to be reduced.

There are at least two load paths in a redundant structure, primary path and a secondary path. However, the secondary path should not be equally as stiff as the primary path. There must be some indication that the bridge is in distress. If not, the bridge may continue to be used without repair until the secondary path also fails.

Redundancy must be clearly separated from the natural interaction of bridge members that is often underevaluated in the design process. In the past, the use of the load distribution factor resulted in generally an underevaluation. Using newer methods of analysis a much more accurate evaluation can be made. In the result the underevaluation of the interaction of a bridge designed using an analysis method which properly considers all structural members is not significant for the ultimate strength of the bridge. In serviceability conditions the interaction of the non-structural elements such as wearing surface, curbs, parapets etc. may still have a significant role.

The analysis of redundancy in bridge structures can be done by using the system reliability methods [2]. The available reliability models allow for calculation of the reliability indices for the whole bridges. Bridge reliability depends on the reliability of its members (girders, slab, connections, details).

CALIBRATION PROCEDURE

The development of a design code includes five basic steps:

(1) Define scope of the code.
(2) Define code objective (select the measure of adequacy, e.g. structural reliability).
(3) Select the target reliability level.
(4) Select the code format.
(5) Calculate code parameters (load and resistance factors).

Scope AASHTO Specifications cover a broad range of bridge structures (steel, concret and timber). Even though it is not specifically mentioned in the code, the spans are covered up to about 300 ft (90m). There are some new materials, technologies and structural types which are not covered (e.g. segmental structures and cable stayed bridges). The design provision must be provided for structural members, connections and whole structures. The conditions for the structural integrity include flexural and shear capacity, compression and tension, local and overall stability, fatigue, cracking and corrosion. Load components and load combinations include dead load, live load (static and dynamic), environmental forces (temperature, wind, earthquake, snow, ice, water) and special forces (e.g., collisions).

Code Objective Code provisions must ensure that the safety level of designed structures is adequate. In the past, allowable stresses were specified not to be exceeded in any structural member or connection. However, the stresses induced by various load components were determined in an inconsistent manner (mostly by judgement). Allowable stresses generally do not reveal the actual safety reserve in the structure. Therefore, it is proposed to measure the structural adequacy in terms of safety (reliability index). For various conditions, structural materials and types, the acceptable values of the reliability index (target reliability index, B_T) can be established. Then the design criteria can be developed accordingly. The code objective is to minimize the discrepancy between the reliability of designed structures and preselected, target reliability level(s).

Target Reliability Level For various design situations covered by the code the acceptable safety levels must be established. These levels, conveniently expressed in terms of target reliability indices, serve as a basis in the development of design criteria (load and resistance factors). Selection of B_T is a multidisciplinary task. It involves calculation of the reliability indices for existing bridges and evaluation of their performance. Economic considerations also play an important role.

Code Format The major factors affecting the code format are simplicity of the provisions (volume, number of requirements, equations, differentiation between types of elements, materials, etc.) and accuracy in meeting the code objective (target safety level). An actual code is a compromise between simplicity and accuracy. In the extreme case, the design parameters (materials, sections, dimensions) are specified for each situation separately. Such a code would be extremely complicated and voluminous. On the other hand, a code could be reduced to a list of target reliability indexes. The designer would have to specify the load-carrying capacity parameters such that it is close to the target value. This would require an extensive reliability analysis for each design case considered.

The Load and Resistance Factors are calculated after nominal values of load components and resistances are established and after the target-reliability level is selected. Load factors can be determined first. Then, the resistance factors are calculated.

The factored load (product of nominal load and load factor) can be determined so that the probability of being exceeded is the same for all load components in the considered load combination. Several probabilities can be tried. Each will result is a different set of load factors. The final selection of the set can be done after the corresponding resistance factors are calculated.

Load factors are calucalated separately for each load combination. After the calculations are completed for all load combinations covered by the code, the load factors can be readjusted and rounded off.

After load factors are determined for all load combinations, the resistance factors can be calculated. It is recommended to consider a wide range of possible values (resistance factors are rounded off to the nearest 0.05). Reliability indices are calculated for each value of resistance factor. Closeness to the target reliability index, B_T, is used as acceptance criterion.

After the load and resistance factors are calculated, the practical design criteria are formulated by special adjustments. The purpose of these adjustments is to minimize the number of different factors, load factors, and load combinations to be considered by the user of the code.

Finally the reliability indexes are calculated for the adjusted load and resistance factors. The calculated B's are compared with B_T and if the comparison is satisfactory, the calibration process is completed.

CONCLUSIONS

The methodology is available for the development of rational bridge design criteria. In particular, this includes the modern structural analysis methods, probabilistic methods and theory of code optimization. A procedure is developed for calculation of load and resistance factors for a bridge design code. The proposed approach is based on strucutral reliability as the acceptance criterion. Various steps of code development are discussed. It is suggested to calculate load factors first, with the load factor value related to the degree of load variation. Then resistance factors can be calculated to provide for the target reliability level. An important part of the calibration procedure is the selection of the acceptable reliability index.

ACKNOWLEDGEMENTS

The study was performed as a part of the Project FHWA/RD-87/069 [1], sponsored by the Federal Highway Administration, which is gratefully acknowledged. The consultants were Fred Moses, Harold Sandberg and Niels Lind.

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SEISMIC DESIGN OF CABLE-STAYED BRIDGES: EVALUATION AND RESEARCH NEEDS

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SYNOPSIS

Earthquake-response characteristics of long-span, cable-stayed bridges are evaluated. The cases of multiple-support as well as uniform seismic excitations are considered; furthermore, effects of the nondispersive travelling seismic waves on the responses are studied. Different procedures to estimate earthquake loads are presented. Parameters affecting the seismic response are discussed; among other factors are non-linearity, ground motion inputs, structural connections and energy-absorption devices and special bearings between the bridge deck and the towers. In addition to the fact that the study sheds some light on the salient features of the seismic analysis and design of these long, contemporary bridges, the study provides suggestions regarding research needs which should be given further attention and consideration.

CABLE-STAYED BRIDGES AND EARTHQUAKES

Despite the fact that several cable-stayed bridges with long spans have been built or are planned for construction in Japan, in the United States and in other seismically active regions in the world, very few, if any, have yet experienced strong earthquake shaking. Accordingly, the information available concerning the performance of such bridges during earthquakes is meager and offers little assistance to bridge engineers planning a cable-stayed bridge in a seismically active region. Furthermore, the engineering experience with the earthquake-response analysis of this new structural system is quite limited at present. This lack of information about the true behavior of cable-stayed bridges during earthquakes increases the engineering awareness, not only about their seismic design, but also about the consequences of any future increase in their span lengths. Therefore, it is essential that pursuit of economical performance, design rationality and innovation, and confirmation of safety and durability proceed parallel to each other.

MULTIPLE-SUPPORT SEISMIC INPUT PROBLEM

Earthquake ground motions in the three orthogonal directions of a long-span bridge may be transmitted to the superstructure through the two tower piers and the two abutments or anchor piers (Fig. 1). If the bridge is long with respect to the wavelengths of the input motion in the frequency range of importance to its earthquake response, then different portions of the bridge may be subjected to significantly different excitations, a problem normally not important for buildings.

In general, earthquake excitation is the result of a combination of more than one wave pattern, each one characterized by its particle motion and its own apparent velocity of propagation. To represent this propagation character of the ground motion for the seismic analysis of long-span bridges, several approaches at different levels of approximation are available. In the first approach, a constant value of wave propagation velocity is to the entire seismic disturbance. Despite such simplification, the researches based on this approach have thrown some light on the effect of seismic wave propagation on the response of long-span suspension bridges.

A second way to describe the spatial variation of the seismic ground motion is to make use of the recovered earthquake records of the synchronized closely-spaced arrays.

EARTHQUAKE LOADS FOR CABLE-STAYED BRIDGES

Traditionally, short-span and typical, non-cable supported bridges are conceptually designed to withstand mainly service loads such as gravitational deal loads and live (vehicular of train) loads. Then they are checked, as a secondary or refining step, to ensure their resistance to environmental loads such as earthquakes, wind and thermal loads. In the case of cable-stayed bridges, however, the environmental loads are as important as the service loads both in the preliminary (or conceptual) and in the refining design processes. For example, the response of the bridge superstructure, towers and substructures to vertical, lateral and longitudinal ground motions during seismic attack should be investigated in the design process, particularly if the bridge is located in an active seismic zone.

Input Ground Motions

The input ground motions should satisfy the following criteria:

1. The three records, used as input motions, should contain at least 20 seconds of strong ground shaking or have a strong shaking duration of 6 times the fundamental period of the bridge, whichever is greater.

2. The ordinates of the input ground spectra should not be less than 90 percent of the design spectrum over the range of the first five periods of vibration of the bridge in the direction being considered. It is important to point out that multimodal contribution to the total response of the bridge is essential.

EARTHQUAKE RESPONSE CHARACTERISTICS

Based on comprehensive investigations of the earthquake-response characteristics of different cable-stayed bridge models under uniform and multiple-support seismic excitations , the following conclusions and remarks can be made:

1. Three-Dimensionality:

Figure 2 shows a few of the lowest mode shapes of the free vibration of a three-dimensional model. It can be observed in this figure that there is a strong coupling in the three orthogonal directions within almost every mode of vibration. The close spacing of the natural frequencies is another feature of the vibrational characteristics of these bridges. This three-dimensionality in the dynamic behavior of long-span and flexible cable-stayed bridges, and modal coupling, cannot be captured in any two-dimensional dynamic analysis. Therefore, the accuracy of any two-dimensional dynamic analysis of cable-stayed bridges is questionable, and it is recommended to always perform three-dimensional dynamic analysis of this unique class of structures in order to obtain a representative response to any dynamic loading.

2. Concrete- and Steel-Design Alternates:

For each consideration of a new cable-stayed bridge in North America, there are two design alternates; one is a mainly steel design and the other is a mainly concrete design. A comparative study is made between the concrete-design alternate, which is heavier and more rigid, and the steel-design alternate, which is lighter and more ductile or more susciptible to large displacement. It should be emphasized that the concrete design alternate attracts higher seismic-induced forces than those attracted by the steel design alternate.

3. Multiple-support Seismic Excitation:

Multiple-support seismic excitations can have a significant effect and should be considered in the earthquake-response analysis of such long and complex three-dimensional structures. This effect is important especially for more rigid cable-stayed bridges (for example, when the deck is of reinforced or prestressed concrete or when the closely spaced multi-cable supporting system is adopted).

The high member forces induced by the multi-support excitation (Figs. 4 and 5) are due to the differential motion of the quasi-static displacements. For the uniform input motion, the quasi-static displacements represent a rigid-body motion of the whole bridge, and no quasi-static forces will be induced in the bridge elements. This explains why the vibrational and total response curves are identical for the uniform input case (see Fig. 5). Obviously, this differential quasi-static motion is expected to be less pronounced when the bridge becomes more flexible.

4. Seismic-Wave Travelling Effects:

Depending on the dynamic properties of the local soils at the supporting points, as well as the soils at the surrounding bridge site, the travelling seismic wave effect, in terms of time delay and phase difference, should be considered in the seismic analysis of these bridges. Propagation of the earthquake wave train is simulated by taking the three orthogonal components as the input at the left anchorage of the bridge, and adding the appropriate time delays to the other inputs based on the travelling distance and the propagation speed (i.e., an unchanging wave pattern or nondispersive time-lag propagation).

Figure 3 shows a graphical representation of the variation in different response quantities of a bridge model and a function of wave speed. It can be seen from these figures that the various response quantities decrease, approaching a stable value as the wave speed becomes higher; this simply means that a uniform input case is approached.

5. Structural Connections:

Due to the large displacements and member forces induced by strong ground shaking in this type of structure, energy absorption devices and special bearings (such as vertical, horizontal, and longitudinal elastic links between the bridge deck and the towers) should be provided at the supporting points to dissipate seismic energy, thus assuring the serviceability of these bridges.

The response of a cable-stayed bridge to applied loads is highly dependent on the manner in which the bridge deck is connected to the towers. If the deck is swinging freely at the towers, the induced seismic forces will be kept to minimum values, but the bridge may be very flexible under service loading conditions (i.e., dead loads and live loads). On the other hand, a rigid connection between the deck and the towers will result in reduced movements under service loading conditions but will attract much higher seismic forces during an earthquake. Therefore, it is extremely important to provide special bearings or devices at the deck-tower connections to absorb the large seismic energy and reduce the response amplitudes. Good examples of these devices, which make it possible to control the natural period of vibration, are rubber-lead block bearings, elastic links, spring shoes and elasto-meric bearings,etc. These devices should be dimensioned so that they provide an adequate stiffness high enough to produce acceptable performance under day-to-day service conditions, yet soft enough to prevent high seismic inertial forces from being transmitted to the towers from the deck. These devices should also constitute a multi-defense line; that is, they should be composed of different, tough structural subsystems which are interconnected by very tough structural elements (structual fuses) whose inelastic behavior would permit the whole bridge to find its way out of the critical range of dynamic response.

7. Material and Geometric Nonlinearities:

Although for the present range of center or effective spans (up to 1400 ft. or 430 m) linear dynamic analysis is adequate , nonlinear static analysis under dead loads is still essential to start the linear dynamic analysis from the dead-load deformed state of the bridge. For the recent and future trends of longer center or effective span ($\geq$ 2000 ft. or 600 m), geometrically nonlinear dynamic analysis is necessary for computing the response of the bridge when rejected to strong ground shaking. Such trends make the need for a nonlinear analysis inevitable; this is essential not only for evaluating the stresses and deformation induced by environmental loadings, such as vehicular traffic, wind, and earthquakes, but also for assuring safety during construction.

Geometrical as well as material, nonlinear dynamic analysis is necessary to compute the response of long-span cable-stayed bridges subjected to very strong ground shaking.

RESEARCH RECOMMENDATIONS

Although several investigations have contributed recently and significantly to the understanding of the earthquake-response analysis and design of long-span cable-stayed bridges, it is still essential that more research be conducted on the subject to investigate other aspects of the problem. The following points are suggested for further attention and consideration:

1. The effect of dynamic soil-structure interaction, and local soil conditions at the site of the bridge, on the dynamic characteristics and the earthquake-response of these bridges,

2. The hysteretic capacity of the bridge towers due to dynamic axial force-bending moment interaction;

3. Laboratory and full-scale dynamic testing and measurements, conducted in order to verify current and future analysis procedures and to measure the true mechanical or structural damping values and dynamic characteristics of these bridges,

4. The use of stochastic methods of dynamic analysis (a random vibration or frequency-domain approach) to predict the earthquake-response of such bridges. These methods can allow for studying the effects of support-excitation cross-correlation and modal cross-correlation on the bridge response,

5. The development of more realistic analytical models for ground motions that can predict more representative and appropriately correlated support excitation time histories to account for the spatial and temporal variation of earthquake motion,

6. Analysis, experimental studies and design of more efficient and innovative bearings and special devices (e.g., oil-dampers, elasto-meric bearings, motion restrainers, etc.) to dissipate and absorb vibrational energy due to earthquake and wind, and consequently reduce the high deformations that can occur in such flexible structures at periods of strong ground shaking,

7. Examination of new design concepts regarding anchoring the cables to the towers and the deck, and the use of multi-cable systems for long-span cable-stayed bridges, and

8. Investigation of the dynamic behavior of this type of bridge during different levels of construction, and examination of the influence of the method of erection on the earthquake-response of the bridge in the various construction stages.

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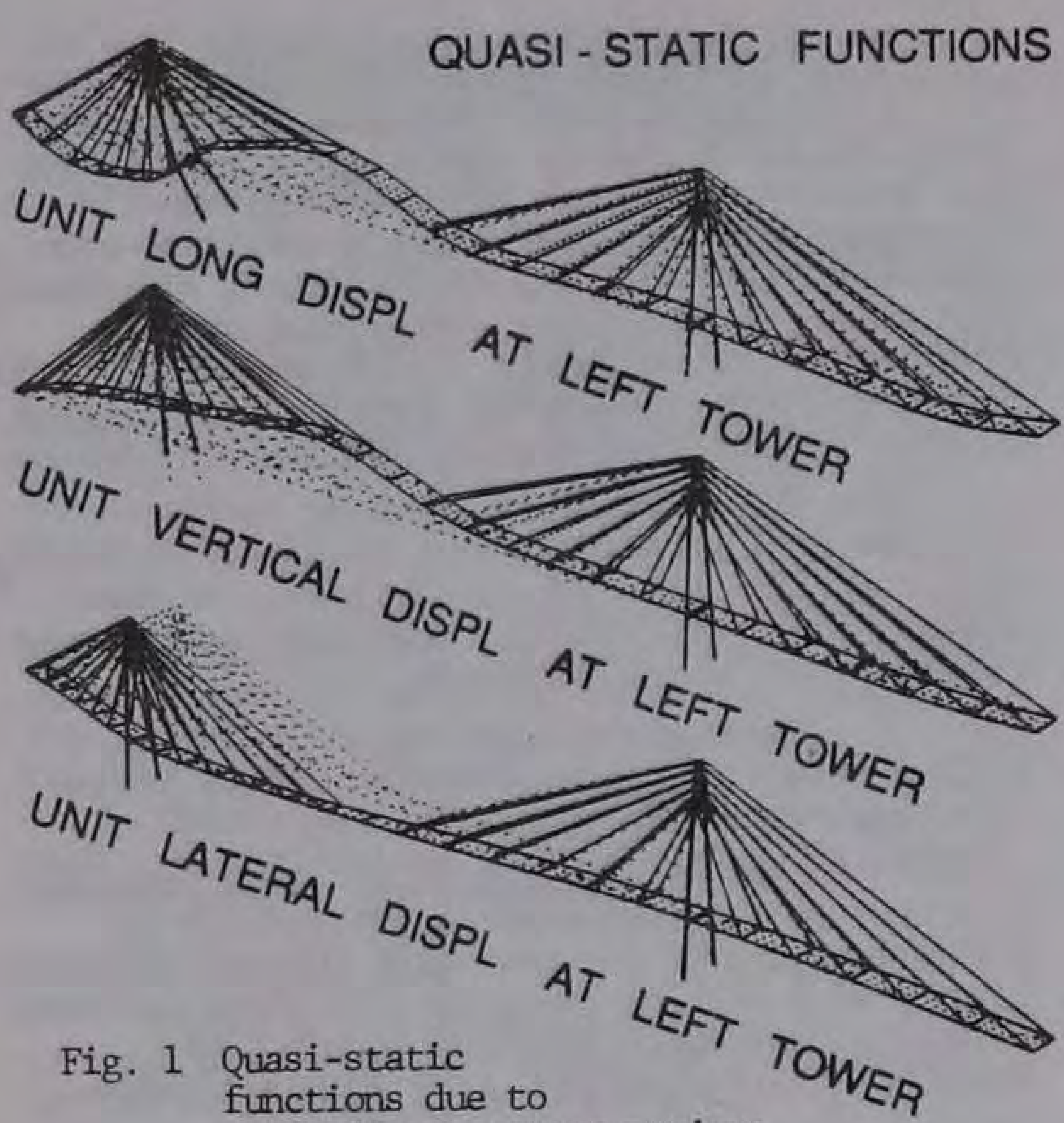

Fig. 1 Quasi-static functions due to non-uniform support motions.

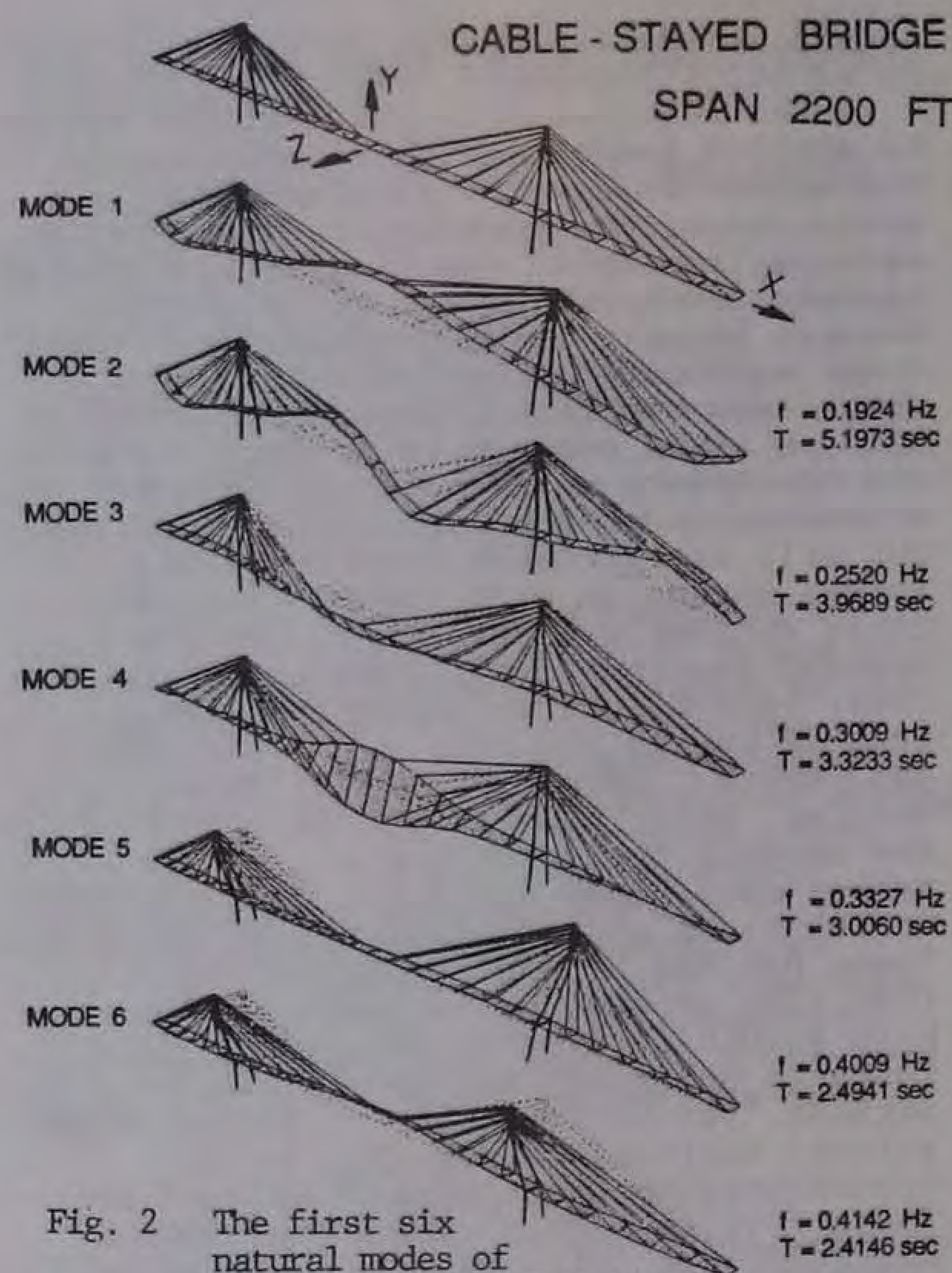

Fig. 2 The first six natural modes of vibration of a 3-D Model of center span equals to 2200 feet.

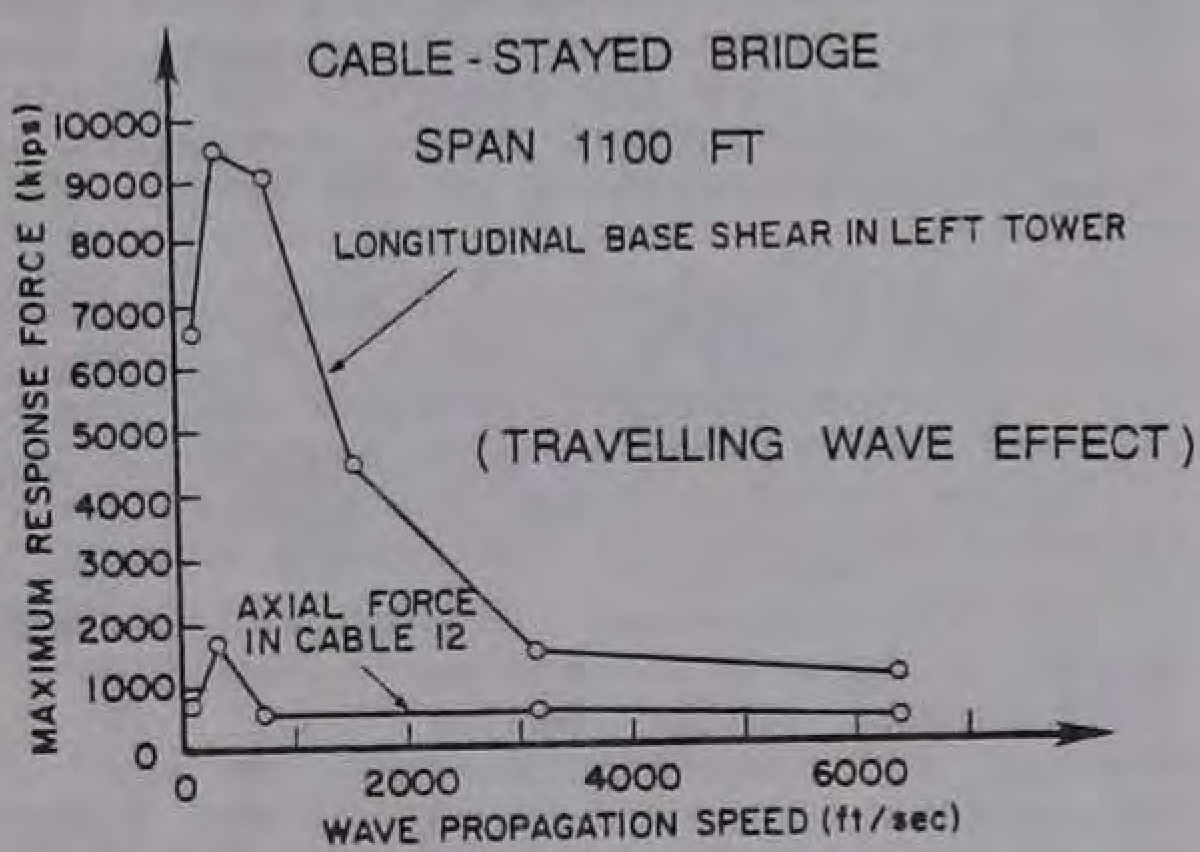

Fig. 3 Effect of seismic wave propagation speed on the maximum response forces of a 3-D model of 1100 ft. center span.

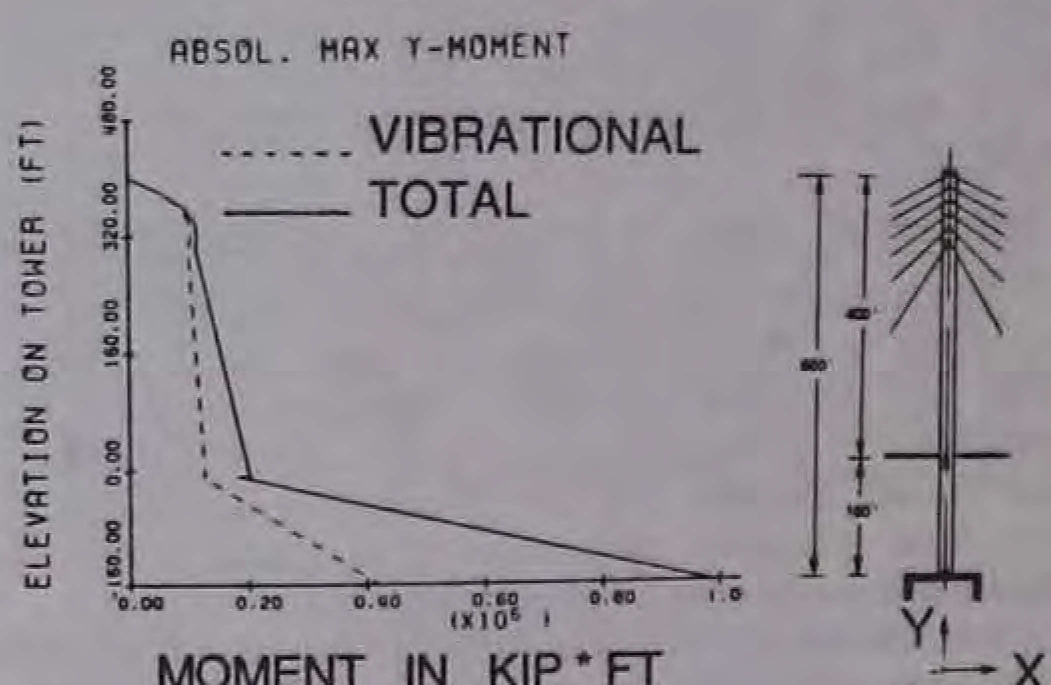

Fig. 4 Tower spatial distribution of the maximum response bending moment.

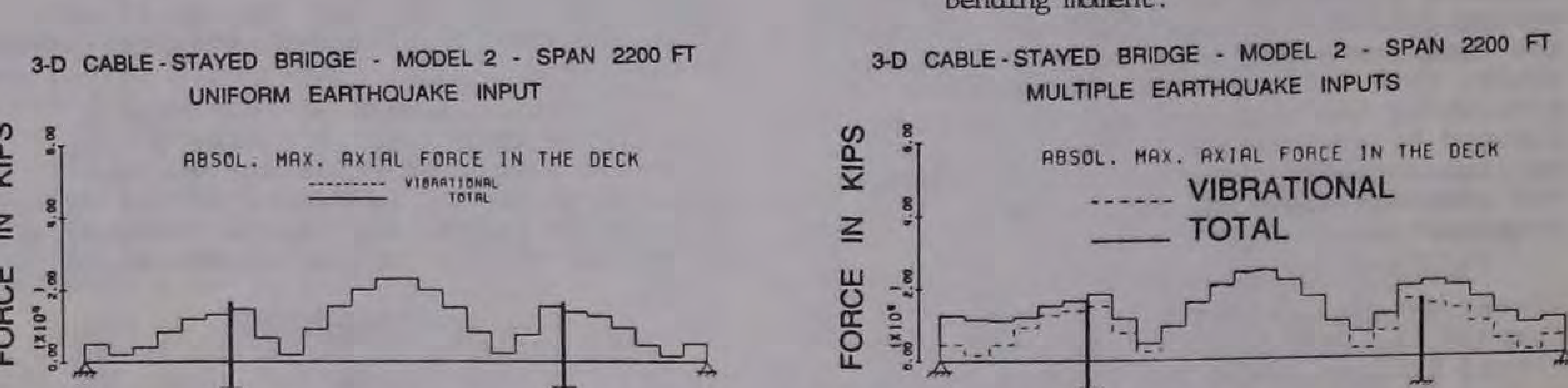

Fig. 5 Distribution of maximum axial force along the bridge deck due to uniform and multiple-support seismic excitations.

SEISMIC RETROFITTING OF BRIDGE COLUMNS

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SYNOPSIS

A current research program investigating methods for improving the seismic performance of existing bridge columns is described. Particular emphasis is placed on increasing flexural ductility capacity, and shear strength of columns of both circular and rectangular cross sections. Methods developed must be immediately viable, technically and economically, to enable implementation in a major retrofit program recently initiated by the California Department of Transportation (Caltrans), which is funding the research.

INTRODUCTION

The 1971 San Fernando earthquake caused substantial damage to recent bridge construction, and forced a reassessment of design philosophy for bridges. Research was undertaken both in the U.S. and overseas to improve analytical techniques, and to provide basic data on the strengths and deformation characteristics of lateral load resisting systems for bridges. In the U.S. research emphasis was primarily directed towards development of sophisticated time-history analysis techniques for bridges. Experimental research was primarily pursued as a means for verifying the analytical techniques.

Parallel to the analytical developments in the U.S., a comprehensive research program into the strength and ductility of bridge columns was carried out at the University of Canterbury, New Zealand, under the sponsorship of the New Zealand National Roads Board. This program has produced detailed information on the flexural strength and ductility, and on the shear strength, of concrete bridge columns, and of concrete piles. Particular emphasis was placed on the influence and effectiveness of lateral confinement reinforcement in the plastic hinge region of concrete columns, in improving flexural ductility.

At the same time as basic research was being carried out, Caltrans was making an initial impact on the considerable problem of improving the safety of existing bridges. Although column failure due to inadequate strength or ductility was recognized as a major problem, the greatest risk was assessed to be the inadequate connection across movement joints in bridges. As a consequence, a major retrofit program was undertaken by Caltrans to install restrainers across movement joints to reduce the risk of collapse of spans due to excessive relative movement. This retrofit program has recently been completed.

Shear failure of columns of the I-5/I-605 separator in the moderate Whittier Earthquake of October 1, 1987 reemphasized the inadequacies of pre-1973 column designs, and has resulted in plans for basic research into methods of retrofitting bridge columns. Results of this research will be used by Caltrans in formulating procedures for column retrofit as part of a major retrofit program approved recently.

The research program was approved in March 1988, and at the time of writing this paper was in the initial stages of implementation. The paper describes the research plan for the program, and some of the background research on which it is based.

ENHANCEMENT OF FLEXURAL STRENGTH AND DUCTILITY

Bridge columns designed before the 1971 San Fernando earthquake typically contain very little transverse reinforcement. A common detail for both circular and rectangular columns consisted of #4 (1/2" dia) transverse peripheral hoops at 12" centers regardless of column diameter. These hoops were typically closed by lapping in the cover concrete, rather than by lap-welding, or anchoring by bending back into the core concrete.

Longitudinal reinforcement was typically lapped with starter bars extending only 20 longitudinal bar diameters from the foundation.

As a consequence of these details, the ultimate curvature capable of being developed within the potential plastic hinge region is limited by the strain at which the cover concrete starts to spall. This is typically at about 0.004 strain. At higher strains, the hoop steel will be ineffective, and the small degree of confinement provided to the core concrete will be lost.

Initiation of spalling is also likely to result in bond failure at the lapped longitudinal reinforcement. Hence flexural strength and ductility will be limited to values corresponding to the spalling strain. Theoretical studies have shown these to be generally inadequate to survive earthquakes corresponding to currently accepted return periods.

Recent research [1] has established that close-spaced lateral confinement reinforcement in potential plastic hinge regions increases the crushing strength of the core concrete, and the effective ultimate compression strain of the concrete. The increase in ultimate compression strain from the unconfined value of about 0.003 to confined values of 0.03 or higher greatly increase the ductility capacity of confined sections. Provided the transverse confinement is spaced no more than 6 longitudinal bar diameters, and is properly anchored by welding laps or by hoop extensions bent back into the core concrete, the effective ultimate compression strain is the longitudinal strain at fracture of the transverse reinforcement. This may be found by equating the energy required to fracture the transverse reinforcement to the additional energy stored by the confined concrete, compared with unconfined concrete [2]. This enhancement in compression strength and compression strain is illustrated in Fig. 1.

Research results have shown [3] that new columns designed with reasonable volumetric ratios of confinement reinforcement ($0.005 \le \rho_s \le 0.03$) develop stable hysteresis loops during inelastic cycling to displacement ductilities exceeding $\mu = 6$. Where axial load levels are high, significant enhancement in flexural strength also results. Although it might be technically feasible to place external hoops on existing circular columns, weld laps and then spray gunite on the surface to provide a rigid connection with the existing concrete, it would be an expensive procedure and unsuitable for rectangular columns

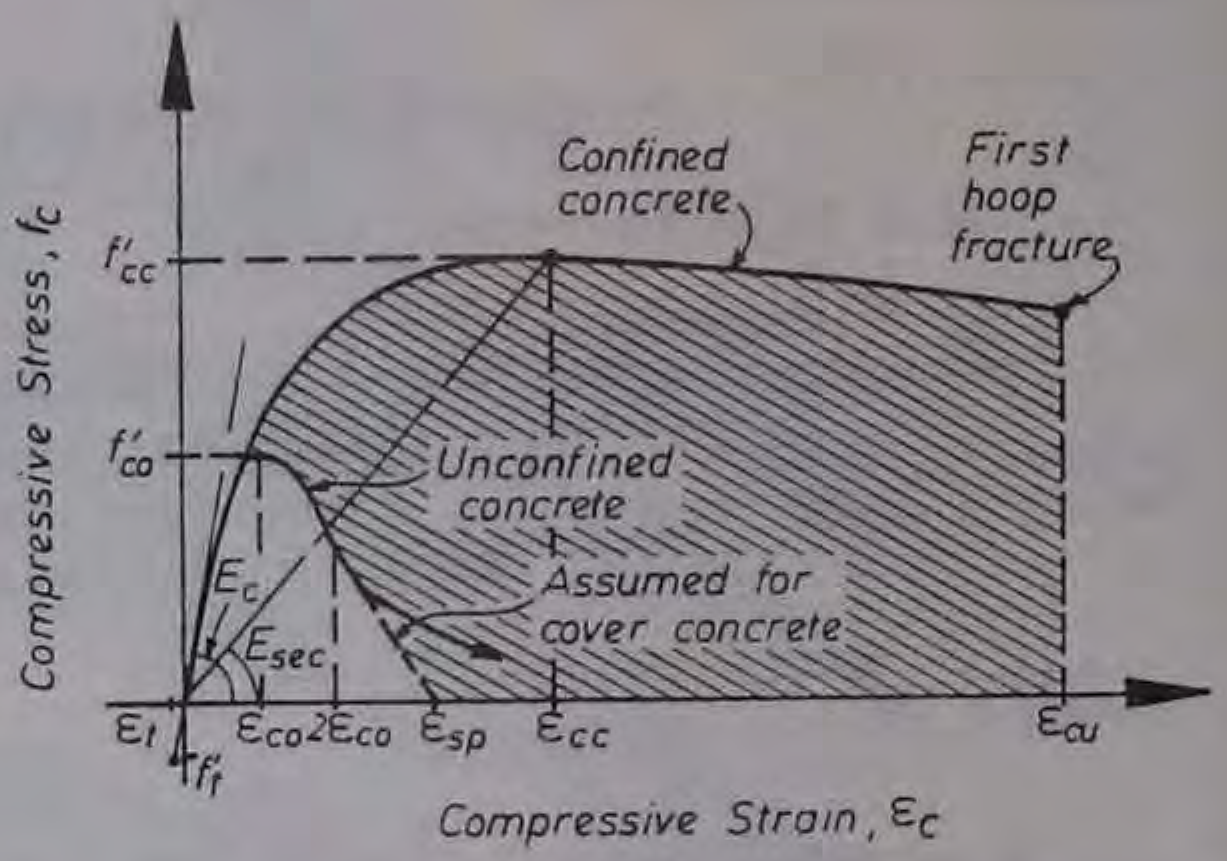

Fig. 1 Stress-Strain Model Proposed for Concrete

because of problems at the corners. Two alternative retrofit strategies appear more suitable. The first is suitable for both circular and rectangular columns, the second can only be used for circular or oval columns.

Confinement by jacketing the column in the plastic hinge regions by means of a cylindrical steel sleeve site-welded up two longitudinal seams promises to be an effective method for both circular and rectangular columns. The jacket would be constructed slightly oversize for ease of construction and the gap between the column and jacket would be grouted with high strength grout. This action would effectively make the column behave as a steel-encased concrete pile. Since the jacket would terminate at the critical section, the jacket would principally act as confinement reinforcement. However, experimental research on steel-encased concrete piles with the jacket terminating at the critical section indicates significant increase ins flexural strength as well as ductility increases. Fig. 2 shows typical hysteresis hoops obtained from such a system [4]. Very large displacement ductility factors were possible due to the high confinement capabilities of even thin steel shells.

It is expected that the lateral pressures from confinement will greatly improve bond conditions on the lapped longitudinal reinforcement.

Japanese research has indicated significant improvement to the response of steel-jacketed square concrete columns as well as to circular columns.

The second potential means of confinement of existing columns consists of wrapping a prestressing wire or tendon,

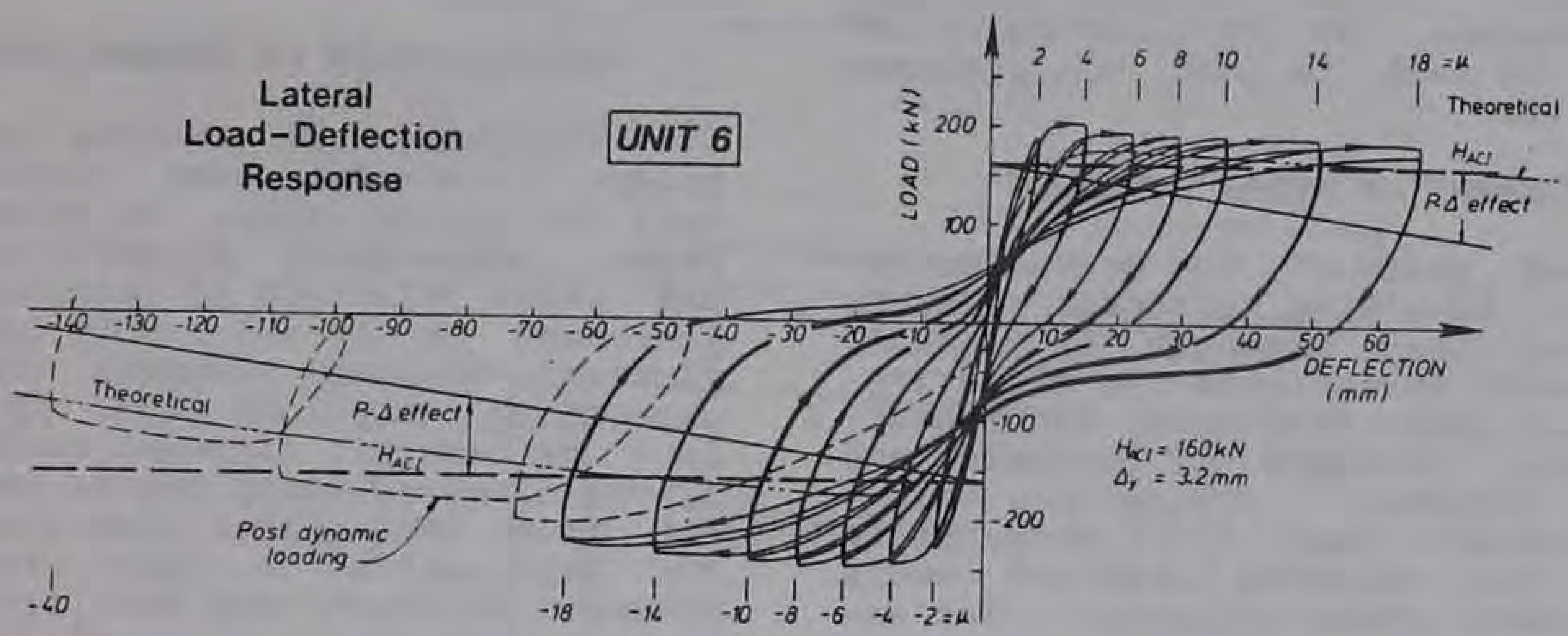

Fig. 2 Load Displacement Hysteresis Loops for Steel-Encased Pile

either initially stressed or unstressed, around the potential plastic hinge region. The wire wraps would be protected against corrosion by a pneumatically placed mortar or concrete cover, or by other methods such as epoxy coating.

In the current research project, recent theoretical research into the strength and ductility of concrete bridge columns and piles will be extended to provide an assessment methodology for ductility capacity of existing designs, particularly those which were common in the pre-1971 era.

Design methods will be developed to establish the improvement in strength and ductility of bridge piers resulting from different retrofit measures. As mentioned above, the most promising retrofit measures currently appear to be external confinement of potential plastic hinge regions by either welding an external steel plate jacket over the hinge region and bonding to the existing concrete by grouting or wrapping unstressed prestressing strand around the hinge region. Both techniques are expected to provide a small but significant increase in flexural strength, a large increase in flexural ductility capacity, an improvement in performance of lapped splices at the base of columns, and should also greatly improve shear strength.

Initial experimental work is directed towards improving flexural strength and ductility of circular columns. Six large-scale models (24" dia) of bridge column/base details will be tested using the test configuration of Fig. 3. Care is being taken to accurately model typical pile cap details as well as the columns, as it is possible that strengthening the columns may result in unacceptable high forces in the pile caps.

The first column models a typical pre-1971 design with high axial load (about 0.3 $f'_c A_g$) to provide experimental verification of analytical techniques for assessing existing strength and ductility. The remaining five circular test units will investigate the effectiveness of different retrofit measures. It is expected that this stage of the experimental program will be extending to include non-circular sections.

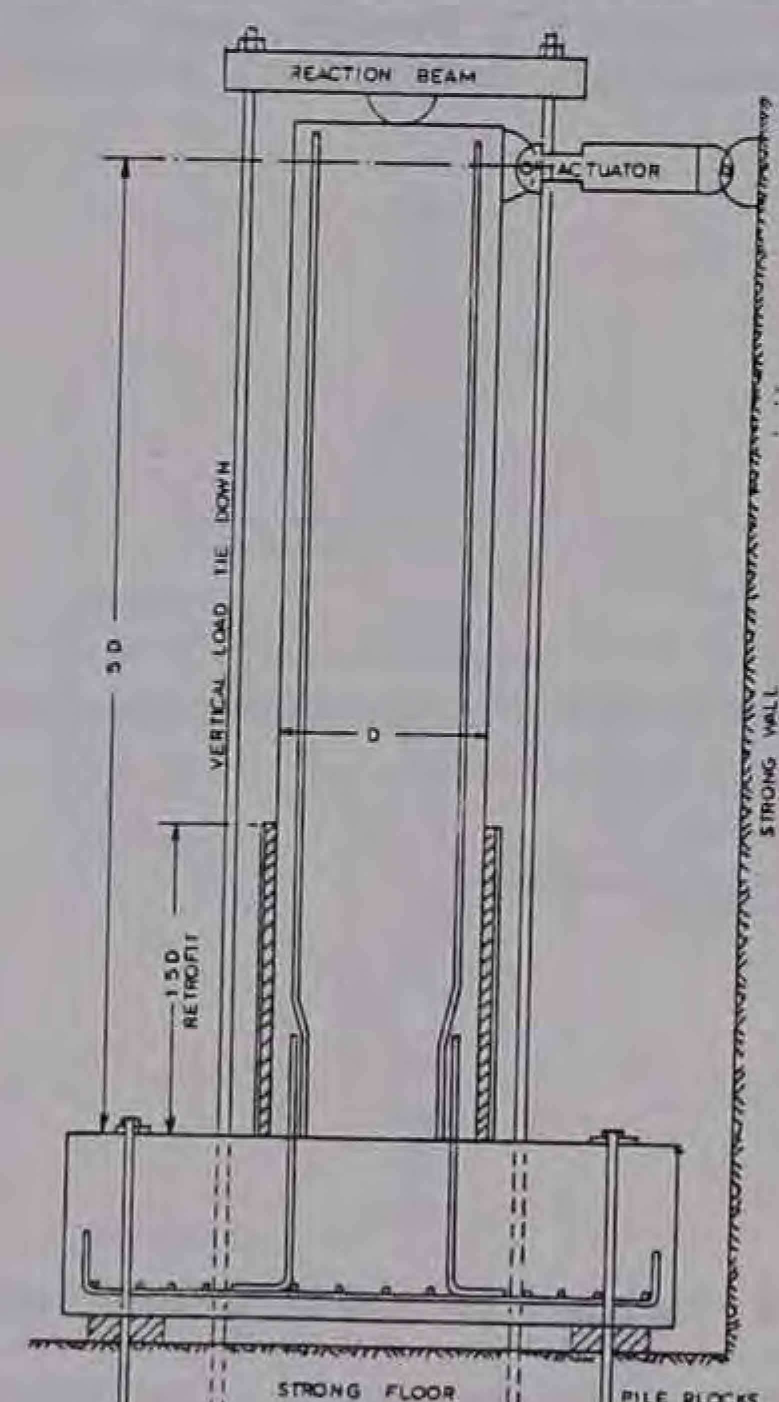

Fig. 3 Column Retrofit Flexural Tests

The columns will be subjected to constant axial load, and cycles of gradually increasing lateral displacement, applied to the column top. The tests are intended to model the behavior of single-column bridge piers under transverse seismic response, as this category of bridge pier is felt to pose the greatest threat to life.

ENHANCEMENT OF SHEAR STRENGTH

A second phase of the column retrofit program involving retrofit measures for improving shear strength is planned. This phase will investigate the effectiveness of steel-shell jacketing for enhancing the shear strength of circular and rectangular columns. Preliminary calculations indicate that 1/4" steel plate bonded to the existing concrete would greatly enhance shear strength. For example, the calculated increase in shear strength to the I-5/I-605 separator columns which failed in shear in the 1987 Whittier earthquake, provided by 1/4" thick plate, is 864Kips per column. This would have been sufficient to ensure suppression of the brittle shear failure mode that occurred. However, testing is needed to determine the effectiveness of the casing in simultaneously resisting shear and providing confining action. Six units would be tested in double bending using the test set up illustrated in Fig. 4. Three circular and three rectangular columns will be tested, with the first of each series of three representing typical "as-built" pre-1971 design, and the other two representing different retrofit measures.

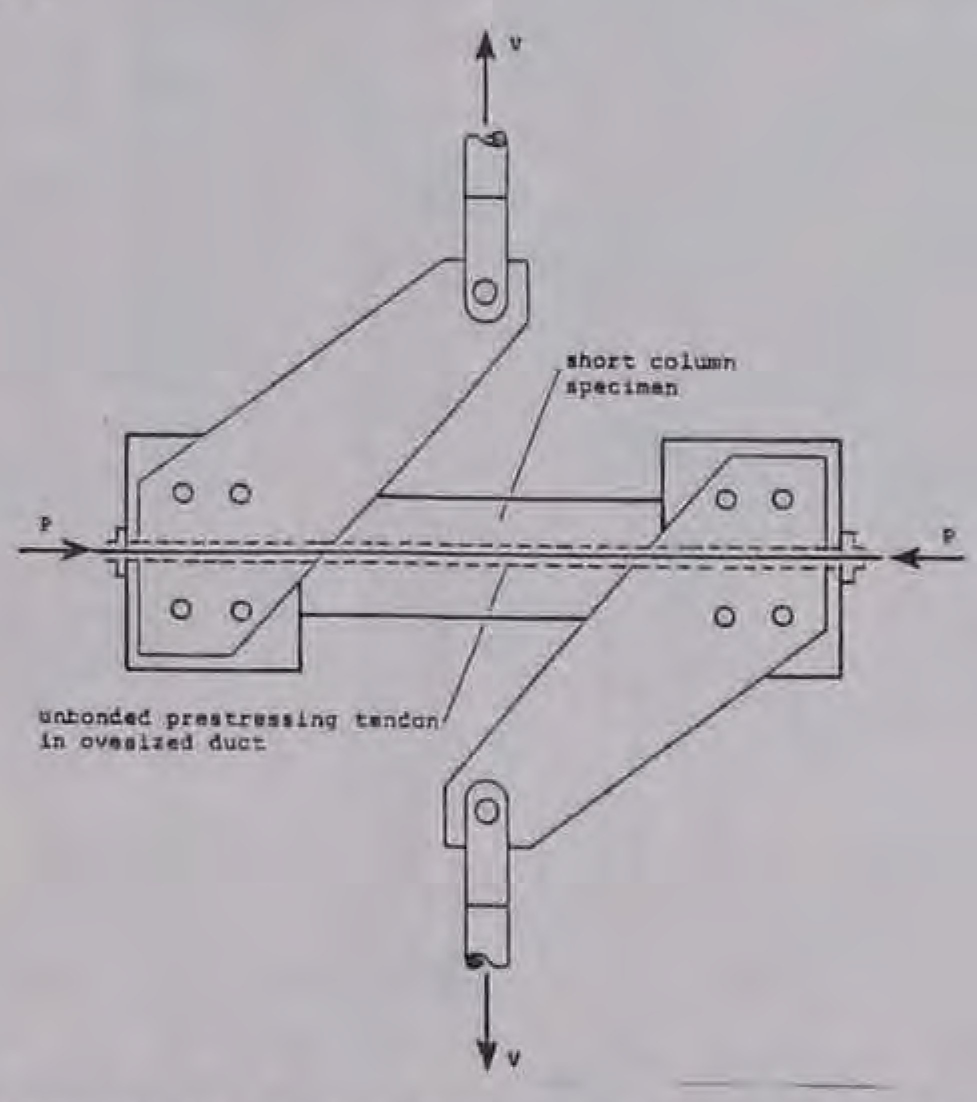

Fig. 4 Column Retrofit Shear Tests

In these column tests, the axial load will be simulated by an unbonded tendon anchored to the two ends of the test unit and passing through an oversize central duct. The intention of these tests is to develop retrofit measures which increase shear strength to the extent that ductile flexural modes of inelastic deformation can develop.

INVESTIGATION OF DYNAMIC RESPONSE

Virtually all testing to date of bridge columns has been carried out at very low strain rates. At seismic strain rates, compression strength of concrete and yield strength of reinforcement are increased by 10-30%. More importantly, flexural tension strength can be increased by more than 100%. This results in a pattern of flexural cracking under dynamic loading which can be very different from that under slow strain rates. The implication of this phenomenon to flexural stiffness and ductility capacity needs investigation by carefully designed shake-table test programs. The model size should be large enough so that scale effects do not mask the influence of the prime variable. A new shake table currently under construction at UCSD will enable comparatively large units (approximately 1/4 full size) to be taken to failure using simulated earthquake ground motion. Two test units are planned, each consisting of twin column bents as shown in Fig. 5.

Two-column bents have been identified as one of the most critical configurations of bridge support for seismic resistance. Compared with single-column piers, they have the additional complexity of a variable axial-load history when subjected to transverse excitation. Initial testing of twin-column piers suffering shear failure has indicated that behavior could not satisfactorily be predicted based on results of single-column tests.

The test units will be based on the Route 5 (truck lane)/I-405 separator, a two-span bridge supported on a two-column central bent which suffered catastrophic failure during the 1971 San Fernando earthquake. The first model will be based on the Route 5/I-405 central bent "as built", with a second unit, including retrofit measures, tested to prove effectiveness under dynamic conditions of the retrofit measure developed in the single-column test program.

CONCLUSIONS

The research program is not sufficiently advanced to be able to draw conclusions. However, it is expected that economically viable column retrofit measures will be developed. Theoretical studies indicate that the use of steel-jacketing of existing columns should be economical and should provide significant improvement in both flexural and shear behavior.

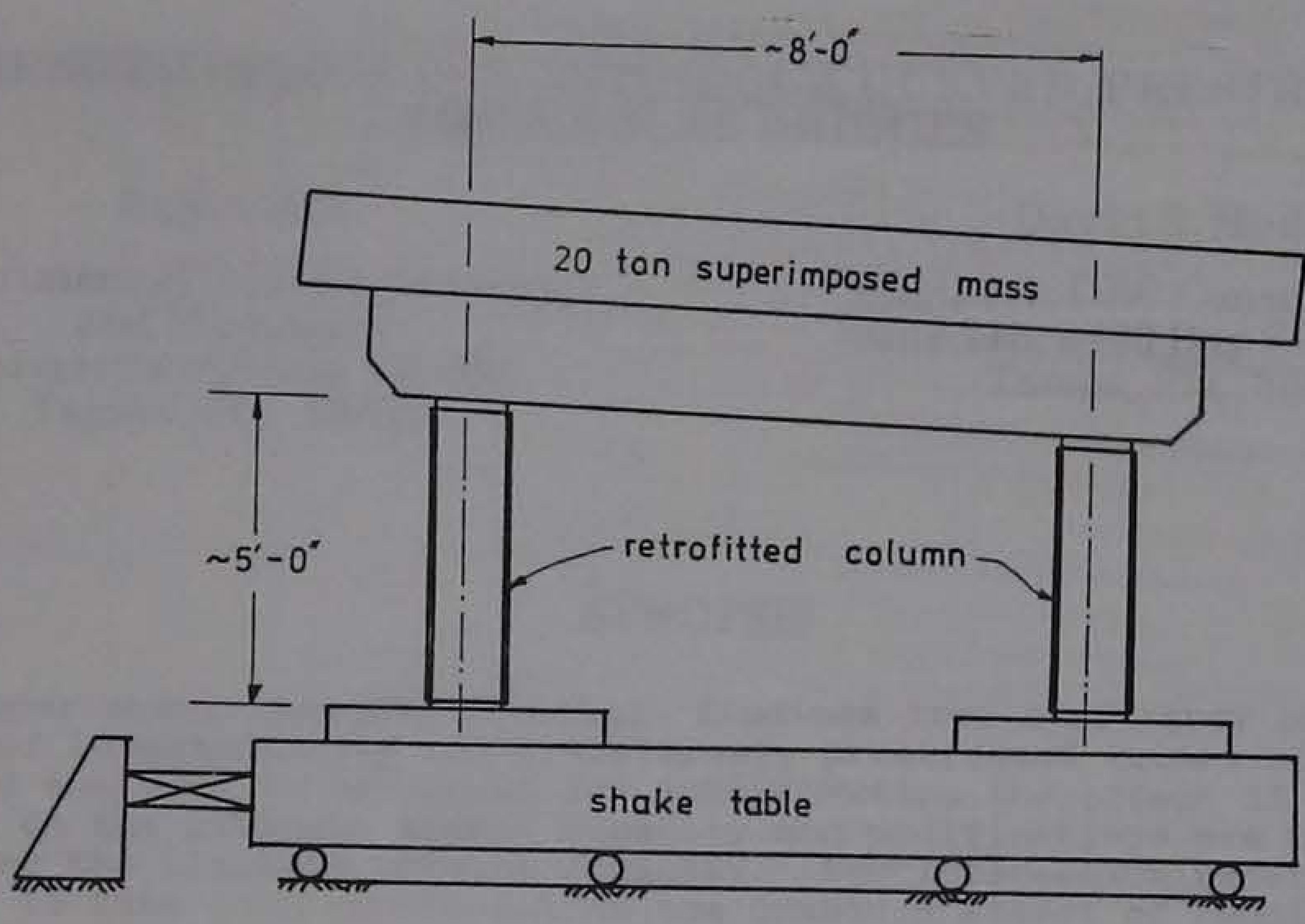

Fig. 5 Shake Table Testing of Route 5/I-405 Model

ACKNOWLEDGEMENTS

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STRENGTH DESIGN OF CONTINUOUS, CURVED, PRESTRESSED VOIDED SLAB BRIDGES

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SYNOPSIS

This paper summarizes the principal findings from a of study on the ultimate strength of longitudinally and transversely prestressed voided slab bridges. A simplified analysis is developed for incorporating the effect of transverse prestress on the ultimate moment capacity and modifications are proposed for calculating the ultimate torsion capacity. New interaction relations have been developed to take into consideration the combined effect of bending, shear and torsion.

INTRODUCTION

Curved bridges are primarily used in interchanges as access ramps or elevated highways. Voided slab bridges are especially attractive for this purpose from both strength and aesthetic considerations but perhaps more so for its adaptability to meet the complex requirements of geometry and space in urban areas.

The vast majority of voided slab bridges built in North America are in Ontario, Canada where almost one thousand such structures have been constructed during the last twenty five years.

All bridges must resist the effect of bending moments and shear. Whereas straight bridges are only subjected to torsion because of live load eccentricity, curved bridges must resist additional torsional moments due to curvature. Consequently, torsional moments are much larger and constitute a significant part of the load effects in curved bridges.

Since torsion causes both bending and shear stresses in non-circular cross-sections, the strength of curved bridges cannot be determined without considering the interaction between bending, shear and torsion at failure.

Current AASHTO specifications [1] provide rules for calculating the ultimate bending and shear capacities. There are however, no rules for determining torsion capacity nor is any consideration given to the interaction between bending, shear and torsion at failure. Moreover, the AASHTO rules for calculating ultimate bending capacity are largely empirical and are based on test results of beams prestressed in the longitudinal direction. Since the voided slab bridges are also prestressed in the transverse direction, the AASHTO rules are strictly not applicable for their design.

This study attempts to answer the following questions :

1. How significant is the effect of transverse prestress on the ultimate flexure capacity ?
2. How is the torsion capacity to be determined ?
3. How is the interaction between bending, shear and torsion to be considered ?

Of course satisfactory answers to the questions posed will only allow the determination of the capacity of a section. The ultimate load capacity of a continuous bridge may be much greater because of the possibility of re-distribution of forces.

In the sequel, a brief description is provided of the analysis carried out to answer the above questions. A detailed account is given elsewhere [2].

EFFECT OF TRANSVERSE PRESTRESS

The AASHTO procedure for calculating the ultimate bending capacity of a longitudinally prestressed member assumes the section to be in pure bending, i.e. a uniaxial stress state. If transverse prestressing is additionally introduced, transverse normal stresses develop leading to a biaxial stress state. Biaxial stress strain relations for concrete [3] have been obtained experimentally but these are only valid for constant ratios, α, of the transverse and longitudinal stress. Since longitudinal stresses due to bending and transverse stresses due to prestress vary with depth (unless prestressing force is applied at the centroid), their ratio, α, is also not constant in bridges. Therefore it is necessary to establish representative values of longitudinal and transverse stresses that may be used to define α.

Inspection of biaxial and uniaxial stress-strain curves in Figure 1 show that for strains below 0.002 there is practically no difference in the uniaxial and biaxial response. Consequently, the α values in this region are of no consequence. The only region of interest is the more highly stressed zone where strains exceed 0.002. Since the depth of the highly stressed part is limited, α can be approximated by basing its value on the average effective longitudinal and transverse stress in this region. Depending on the average transverse compressive stress due to prestress, α values range from between 0.05-0.15.

A computer program was developed that incorporated the biaxial stress strain relations for concrete in a strain compatibility analysis to determine the ultimate moment capacity. A comprehensive parametric study was undertaken to determine the effect of the stress ratio, α, on the ultimate moment capacity. For $\alpha = 0.15$, (valid for designs to the OHBDC specifications [4]) the increases in the ultimate moment capacity range from 2.5% for a reinforcing index of 0.1 to 7.8% for a reinforcing index of 0.3 over that obtained for the uniaxial case. However these increases are more substantial (between 6.1% to 18.8%) over that obtained using the AASHTO empirical equations.

These modest increases suggest that the effect of transverse prestressing on the ultimate moment capacity is not very significant and can be ignored in design.

TORSION CAPACITY

Rules for determining ultimate torsion capacity are included in the ACI Code [5]. However, since these rules are intended for solid sections and box sections it was necessary to conduct investigations to extend their application to voided slab bridges. Based on limited comparison with test data a correction factor proposed by Elliot & Clark [6] was recommended for use in voided slab bridges.

The ACI rules for designing for torsion were intended for rectangular beams and is based on skew bending theory. In applying them to wider sections such as voided slabs, certain limitations affecting detailing of reinforcement should be recognized.

Typical reinforcement for torsion consists of closed stirrups and longitudinal steel. The longitudinal steel can be easily provided in the wider sections and requires no discussion. The vertical stirrup steel provides resistance against bending in the skew direction. Clearly, for this steel to be effective, it should be located far from the skew neutral axis so as to maximize the lever arm. Thus, it should be concentrated at the ends of the voided section. Any stirrup steel provided in the middle of the section, i.e. near the skew neutral axis does not contribute towards the torsional resistance. It does however, contribute towards shear resistance.

It is evident therefore that torsion steel in wide sections such as bridge decks cannot be combined with shear steel as is possible for narrow rectangular beams. The section should be designed and detailed separately.

COMBINED BENDING, SHEAR & TORSION

The effect of bending on the ultimate shear capacity is incorporated in existing design rules. However, the effect of torsion on the bending and shear capacities is only addressed in the published literature.

Proposed interaction relations for bending, shear and torsion were reviewed. Means and standard deviations were calculated to determine their effectiveness in predicting the collapse load of beams tested to failure.

Based on this study, two new interaction relations for bending, torsion and shear shown in Figures 2 and 3 were proposed. Inspection of Figures 2 and 3 shows that the form of the proposed interaction relations are identical. In these figures Mu, Tu, Vu are the load effects in bending, torsion and shear. The corresponding independent capacities are Muo, Tuo and Vuo.

The means and standard deviations for the interaction relations shown in Figures 2 and 3 show better correlation with test data compared to those by previous researchers. More importantly, they lead to significant reduction in the number of checks needed during design.

The validity of the interaction relations proposed was checked by applying them to predict the collapse

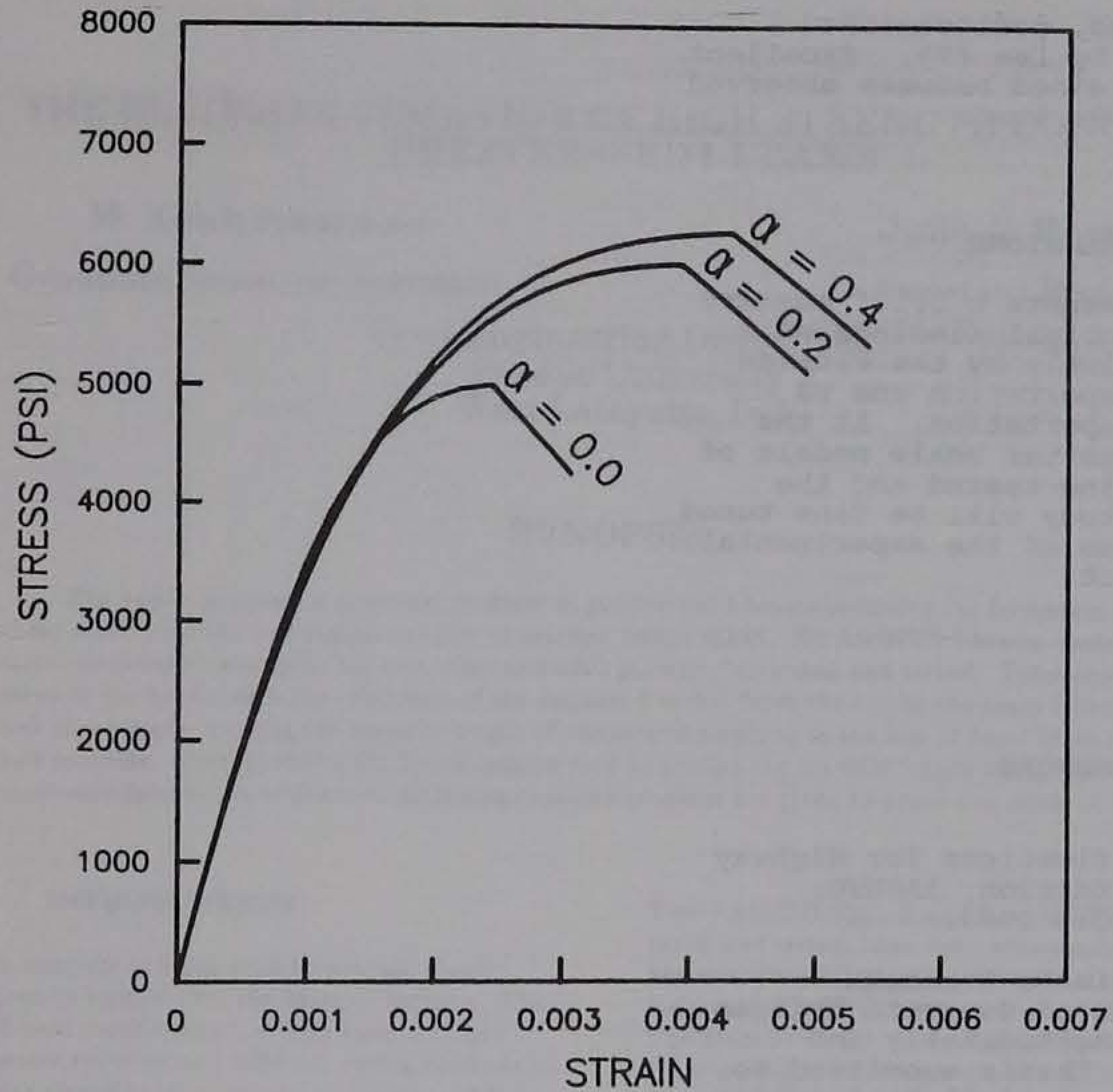

Figure 1 Idealized Stress-Strain Curves in Biaxial Compression [2].

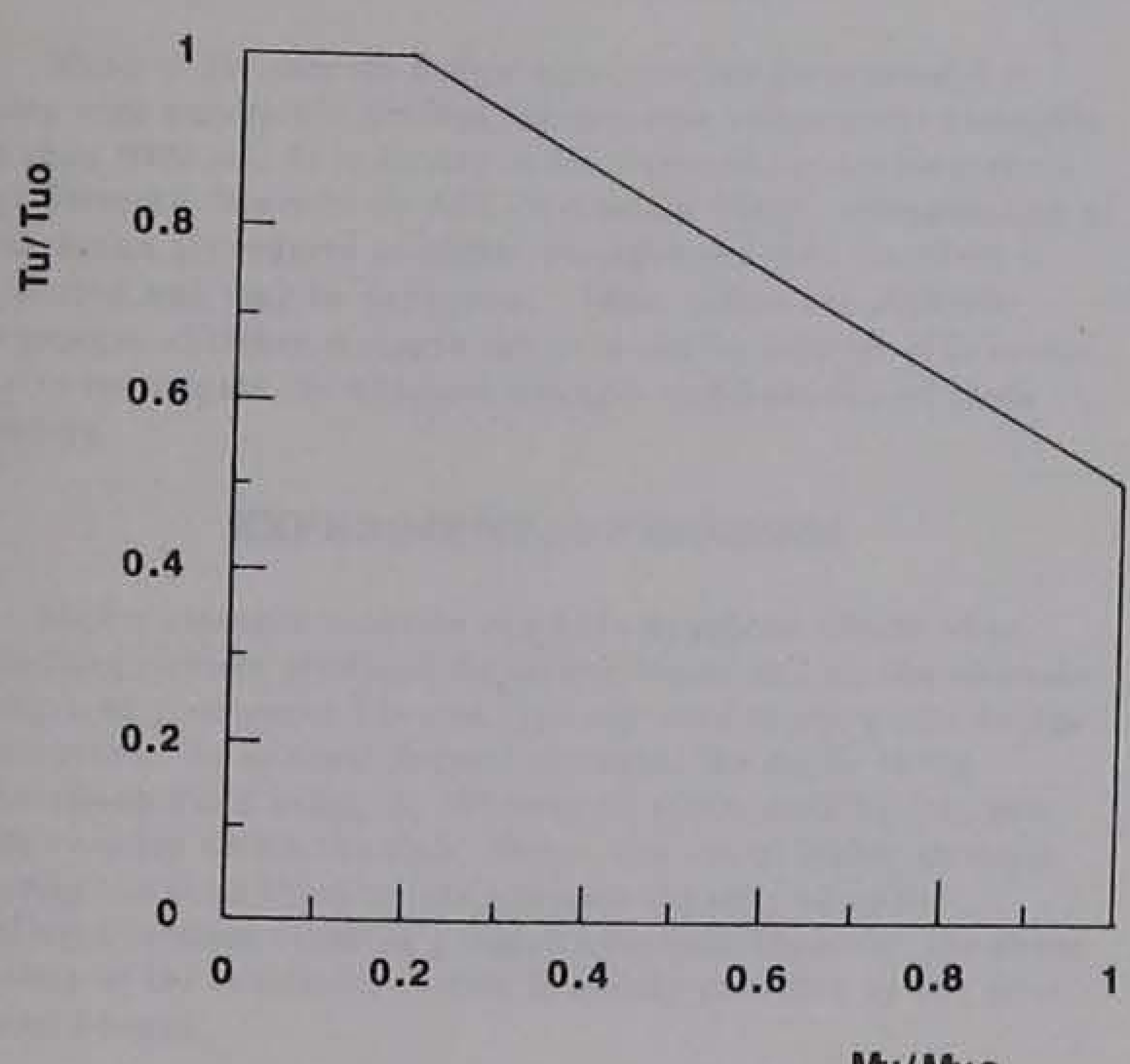

Figure 2 Interaction Relation for Bending and Torsion.

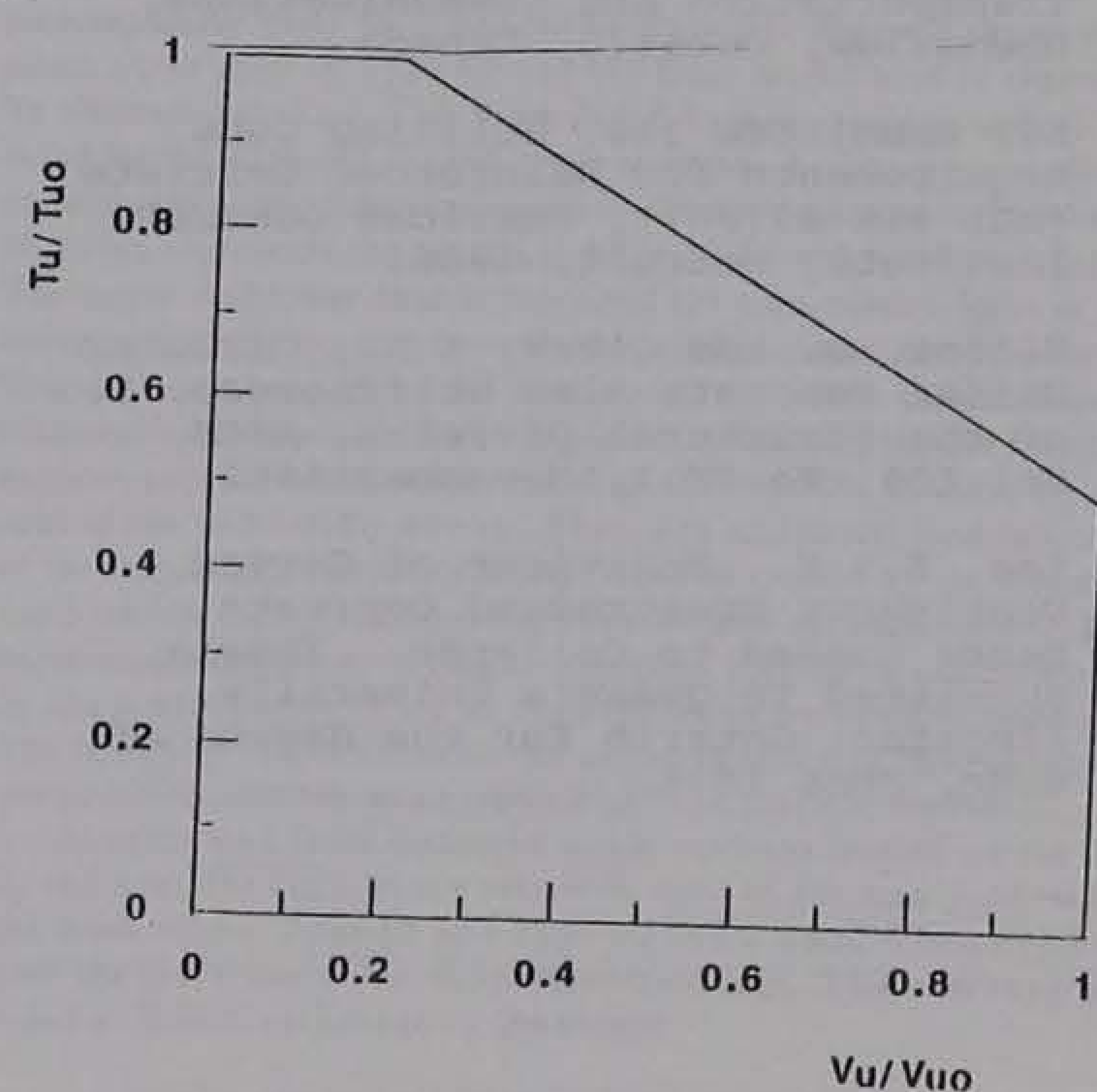

Figure 3 Interaction Relation for Shear and Torsion.

load of a continuous, posttensioned curved beam tested by Lee [7]. Excellent correlation was obtained between observed and predicted failure loads.

CONCLUSIONS

This paper presents a brief summary of some of the principal findings of a study sponsored jointly by the Florida Department of Transportation and US Department of Transportation. At the present time two quarter scale models of the bridges are being tested and the findings of this study will be fine tuned following completion of the experimental phase of the project.

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THE ULTIMATE BEHAVIOR OF HIGH STRENGTH CONCRETE PRESTRESSED I-BEAMS

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SYNOPSIS

The paper discusses a potential problem in prestressed I-beams involving the formation of a web shear crack near the end support region of exterior bridge spans. Six AASHTO I-beams containing concrete compressive strengths between 8000 and 9000 psi were fabricated and tested. Tests conducted on three of the beams with the centerline of the support 6 inches from the end of the beam failed due to a web shear crack crossing the transfer length of the strand resulting in the loss of bond between strand and concrete. Current AASHTO Specifications used to predict the transfer length along with other recommendations are evaluated. Detailing recommendations are given to avoid this mode of failure.

INTRODUCTION

Higher strength concrete is being used by precast plants manufacturing prestressed I-girders for the State of Indiana. The precaster will use different combinations of high early strength cement, high-range water reducers and different curing methods to reach an early transfer strength; thus improving the time of form turn-around. Typically, a transfer strength of 4000 psi can be reached in 12 to 18 hours after casting. Often this results in members with 28 day concrete strengths in excess of 7000 psi. Use of this higher strength concrete in design could lead to savings in dead load allowing thinner members, longer bridge spans, and increased lateral spacing of I-beams.

RESEARCH SIGNIFICANCE

Many of the current design equations for pretensioned I-beams were empirically derived for concrete compressive strengths less than 6000 psi. As indicated in the State-of-the-art Report on High Strength Concrete by ACI Committee 363 [1], extrapolation of these design procedures to higher strength concrete members is unjustified and may be dangerous. Thus, before the multiple advantages of higher strength concrete can be utilized, it is necessary to investigate the ultimate strength and behavior of these members.

EXPERIMENTAL PROGRAM

Higher strength concrete can have beneficial effects when evaluating stresses produced by service loads, and on the ultimate strength of prestressed I-beams typically used in composite bridge construction. At nominal flexural strength, the depth of the compression block using the Whitney [2] stress distribution, generally remains within the slab. Hence, the use of higher strength concrete has little effect on the ultimate capacity of under-reinforced sections resisting positive moments. However, the shear capacity of the composite section is mainly provided by the prestressed I-beam.

Hence, this experimental study focused on the behavior of the prestressed I-beams as non-composite sections. Each beam was designed to fail in shear using the 1983 AASHTO [3] provisions. Two AASHTO Type II and four AASHTO Type I-beams were fabricated and tested. Nine tests were conducted on the six beams. Material properties and geometry of the six beams are given in Reference [4]. The two beam cross sections are shown in Figure 1. The amount of web reinforcement was controlled by the horizontal shear strength criteria specified in AASHTO (9.20.4). A symmetric two point loading system was used in the first six tests (see Figure 2a); a single point load system was used to retest three of the six beams (see Figure 2b).

The first two beams tested were AASHTO Type I's. The test span of the Type I-1 was 18 feet. Two foot overhangs past the centerline of the support removed the transfer length from the shear span. Type I-1 failed in flexure. The shear span of the second beam, Type I-2, was reduced to 5 feet resulting in a reduced moment-shear ratio as compared to Type I-1. Figure 3 shows the south shear span of Type I-2 and the truss model used to represent its ultimate behavior. This beam failed by web crushing as indicated by the enclosed dashed circle in Figure 3. The truss model is constructed in the following manner; the vertical tension member of 39.3 kips represents the resultant force of the web reinforcement. The upper and lower chords represent the compression force in the concrete and the tension force in the reinforcement, respectively. Diagonal members are added to complete the truss. An additional diagonal compression member is placed connecting the load and support points, without this member, the capacity of the truss is that of the web reinforcement. Thus, any additional load in excess of the web reinforcement capacity, must be carried by the diagonal compression member connecting the load and support points. The horizontal component of this compression member is equilibrated by the lower tension chord at the support to maintain equilibrium. The failure of this beam occurred as the capacity of this diagonal compression member was exceeded and the member exploded. Loads calculated from measured strain readings located on the strand near the support are relatively close to the load predicted by the truss model. Type I-1 and Type I-2 had a 2 and 3 foot overhang past the centerline of the support, respectively. This overhang provided sufficient anchorage to the strand.

A typical support detail of an exterior bridge span is shown in Figure 4. Depending on the type of bearing system used, the centerline of the support is typically between 6 and 12 inches from

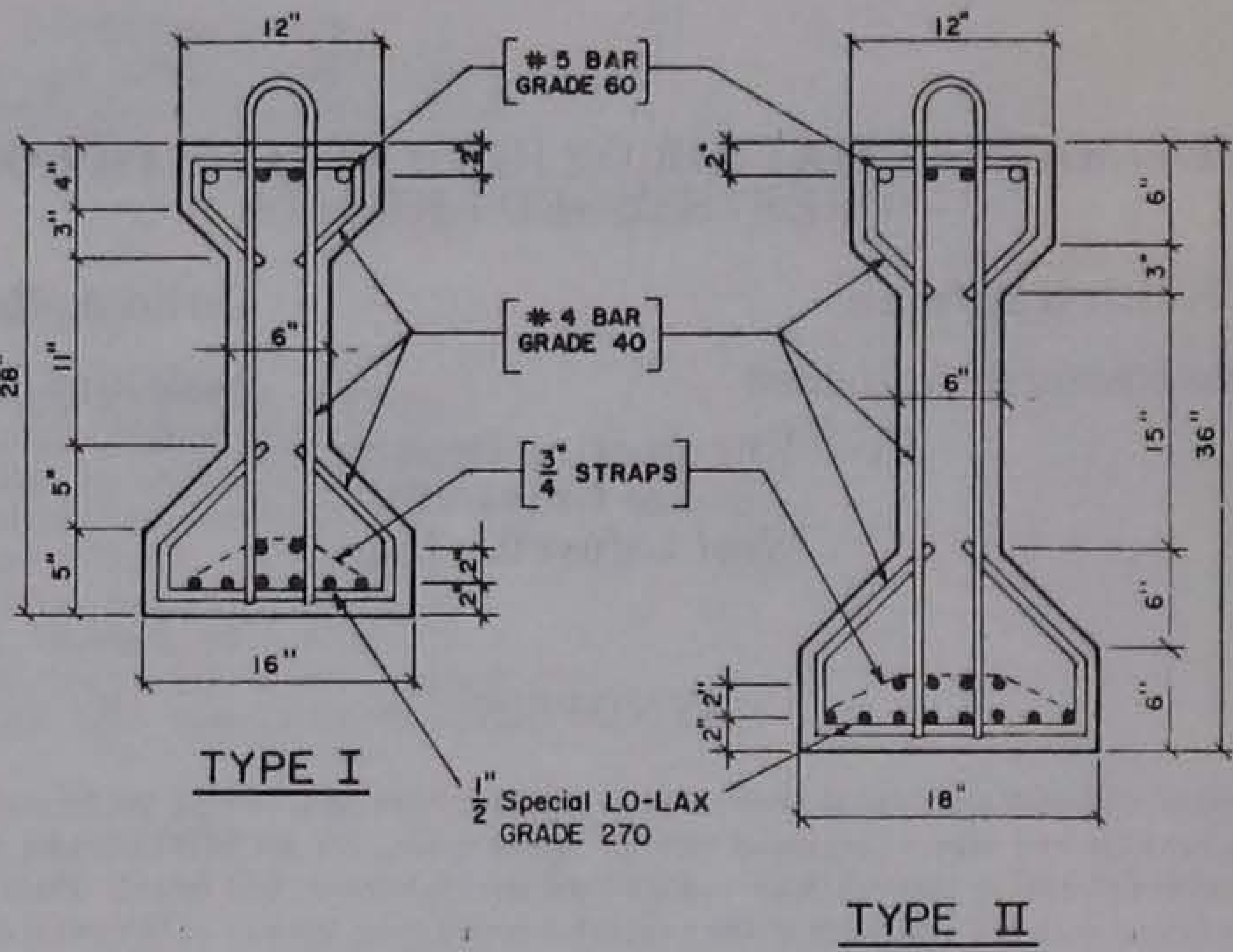

Figure 1 - Type I and II cross section

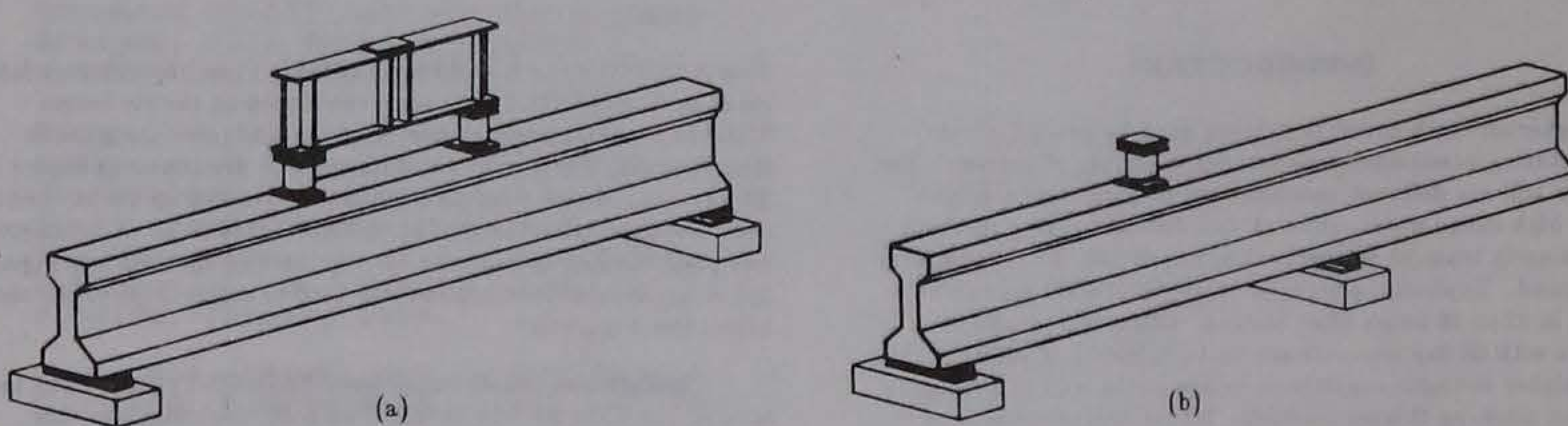

Figure 2 - Test setup

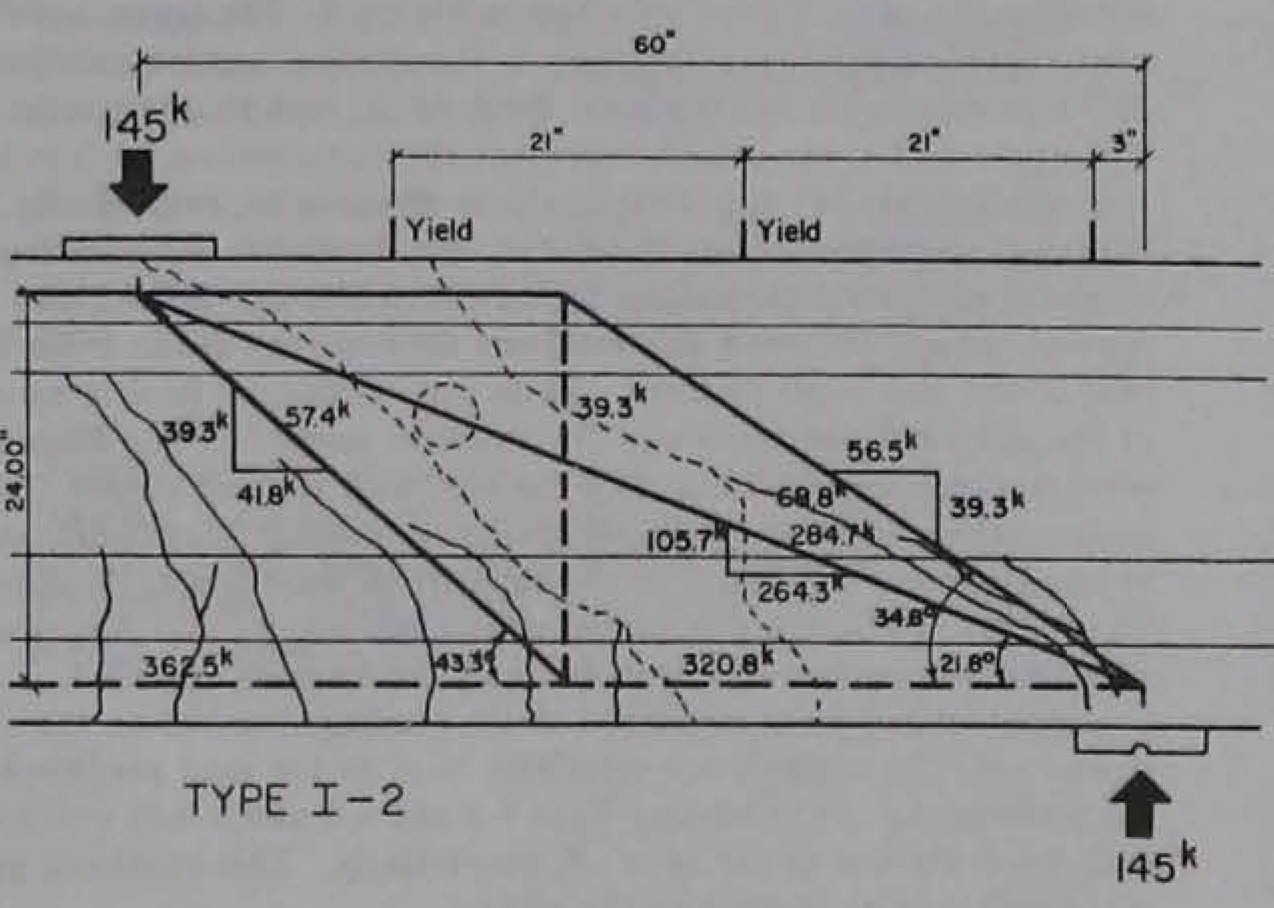

Figure 3 - Type I-2, web crushing failure

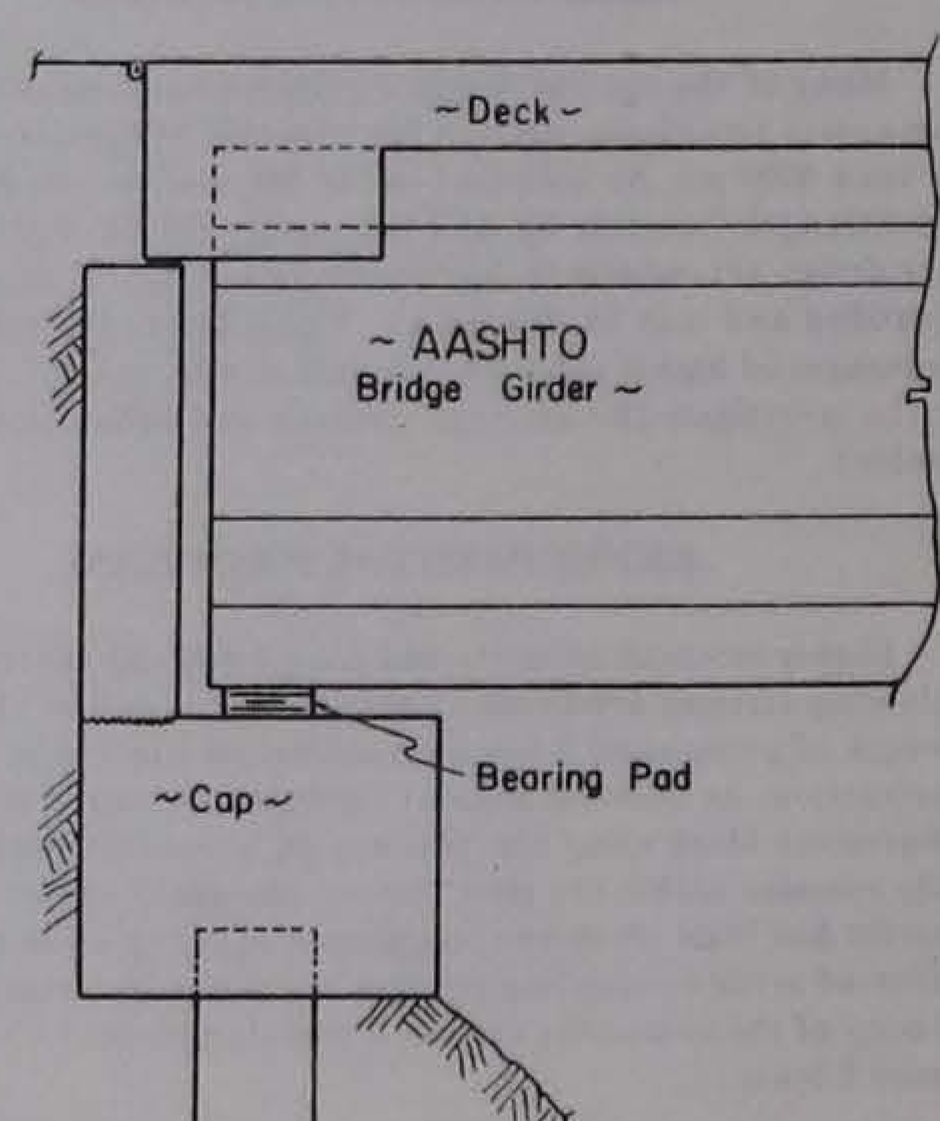

Figure 4 - Typical end support detail

the end of the beam. Three beams were fabricated and tested to determine the behavior of this typical support detail. Two tests were conducted on three beams with the second test designated by the letter "A." The following discussion will focus on the results of these six tests.

Type I-3, I-4 and II-1 had the centerline of the support 6 inches from the end of the beam. Hence, the transfer length of the strand, taken as 50 diameters, was within the shear span. For each test of the three beams, the formation and propagation of the first web shear crack produced loss of bond between the strand and concrete resulting in a premature failure and a substantial reduction in load carrying capacity. Any attempt to increase the load resulted in further strand slip and failure of the beam resulting from excessive width of the web shear crack leading to crushing of the concrete near the load point of the beam. The loss of bond stopped further redistribution of internal forces, and the web reinforcement could not be mobilized. Hence, the ultimate capacity of these three beams was the load required to produce a web shear crack. The shear force versus centerline deflection graph for the three beams are shown in Figure 5. A second test was performed on these three beams to determine the length of overhang that would provide sufficient anchorage to the prestressing strand. An 18 inch overhang was used on Type I-3A. The formation of the first web shear crack produced slip in approximately 50% of the strands; however, the load which produced the web shear crack was maintained. An increase of the load eventually produced loss of bond in all of the strands leading to a shear tension failure. A 24 inch overhang was used in the second test of Type I-4A and II-1A. This length of overhang provided sufficient anchorage to the strands in these two beams. The AASHTO prediction of the nominal shear strength and the failure load of the six beams is listed in Table I.

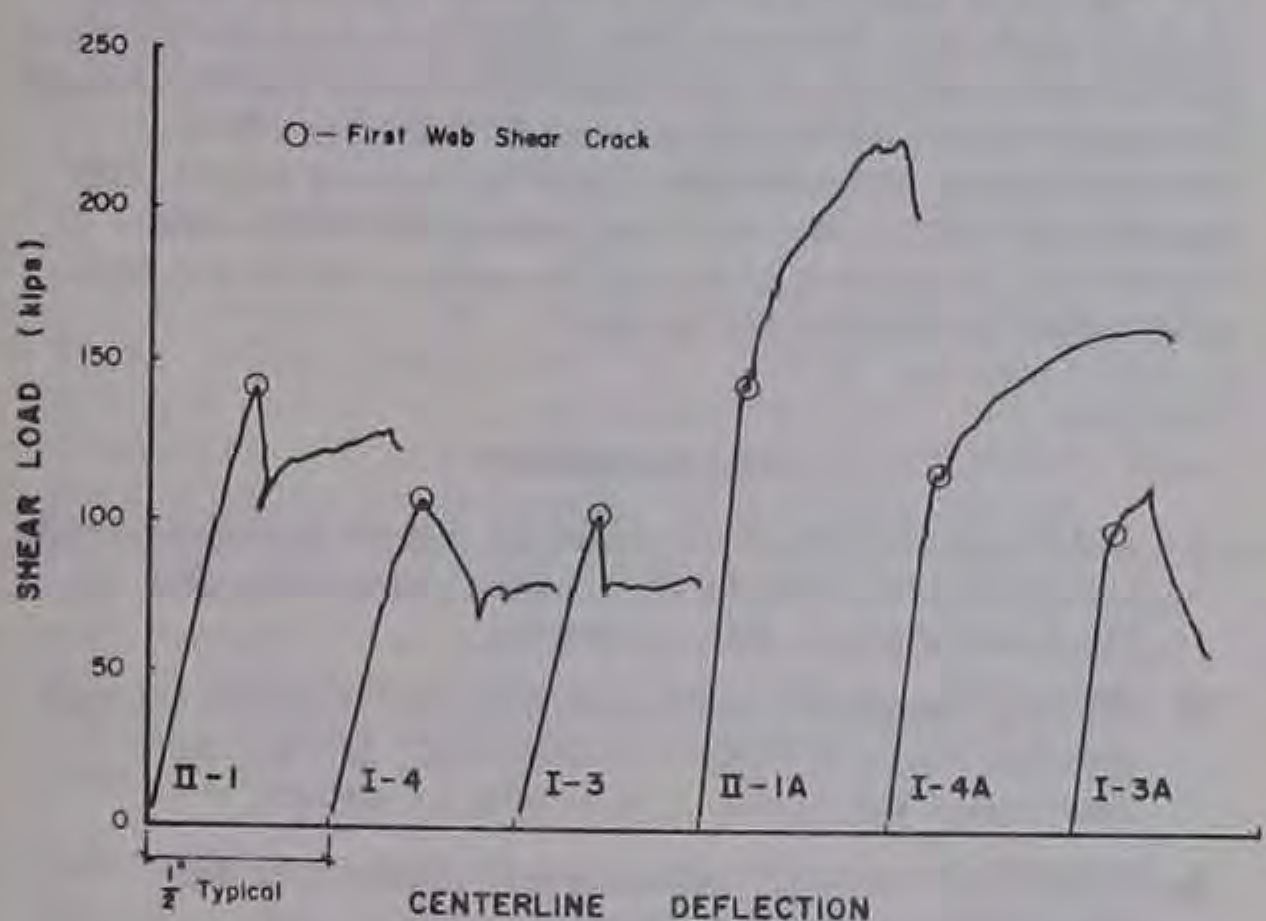

Figure 5 - Load versus centerline deflection

ANALYSIS OF TEST RESULTS

In the tests where the transfer length was within the shear span, the formation and propagation of a web shear crack across the prestressing strand produced a premature failure resulting from the loss of bond between concrete and prestressing strand. This phenomena would be critical near the end supports of exterior bridge spans with the transfer length extending past the support into the main span.

The shear strength provided by the concrete as specified in AASHTO (9.20) is the lesser of V_{cw} or V_{ci}. Two types of shear cracks are recognized by this particular provision, web shear cracks and inclined flexural shear cracks. The term V_{cw} predicts the load required to produce a web shear crack: this type of crack typically appears near a support;

$$V_{cw} = \left(3.5\sqrt{f'_c} + 0.3\, f_{pc} \right) b_w\, d + V_p \qquad \text{(i)}$$

Equation (i) is an empirical approximation to the exact elastic solution to this problem. The shear required to produce a web shear crack is determined by calculating the principal tension stress at the centroid of the cross section and setting it equal to the tensile strength of the concrete. Table I shows that Equation (i) predicted the formation of a web shear crack relatively well for the beams tested.

Table I - Web Shear Cracking Load and Nominal Strength

Type	f'_c	f_{pc}	Test Results V_{cw}	V_n	AASHTO V_{cw}	$V_n^{\dagger}$
	(psi)	(psi)	(kips)	(kips)	(kips)	(kips)
I-3	8370	1112	101	101	100.0	120.3
I-3A	8810	"	98	113	101.3	113.5
I-4	8370	1126	110	110	100.7	117.5
I-4A	8810	"	118	161	101.9	127.1
II-1	9090	1152	140	140	135.8	167.2
II-1A	8950	"	150	222	135.3	162.7

† $V_n = V_{cw} + V_s$ at $\frac{h}{2}$ from face of support
$b_w = 6$ inches for all beams
$d = 25.5$ and 33.33 inches for Type I and Type II beams, respectively

In pretensioned members, the prestressing force is transferred to the concrete by bond. The length required to develop this force is called the transfer length. The additional length required to develop the stress in the strand at nominal strength is referred to as the flexural bond length. The development length of the strand is the summation of the transfer and flexure length. AASHTO Specification (9.27) requires that the development length from a critical section satisfy the following equation:

$$L_d \geq \frac{f_{se}}{3}\, d_b + \left(f_{ps} - f_{se} \right) d_b \qquad \text{(ii)}$$

This equation is limited to three and seven wire strand. Equation (ii) indicates that the development length is a function of the nominal diameter of the strand, effective stress and the nominal stress in the strand. In Equation (ii) the first term is the transfer length, and the second term is the additional length required to develop the stress in the strand at the nominal trength. The transfer length is commonly taken as 50 d_b, which assumes an effective stress in the strand of 150 ksi. In other words, Grade 270 stress relieved strand, with a 70% jacking load, and approximately 20% prestress losses.

Zia and Mostafa[5] proposed as an alternative the following equation to determine the development length:

$$L_d = L_t + L_b = 1.5\, \frac{f_{si}}{f'_{ci}}\, d_b - 4.6 + 1.25 \left(f_{ps} - f_{se} \right) d_b \qquad \text{(iii)}$$

With the transfer length given as:

$$L_t = 1.5\, \frac{f_{si}}{f'_{ci}}\, d_b - 4.6 \qquad \text{(iv)}$$

The transfer length in Equation (iv) is a function of, the nominal diameter of the strand, stress in the strand after initial losses and the concrete strength at transfer.

In all tests of this experimental investigation [4], the support plate had a width of eight inches. The formation of the web shear crack for all six tests crossed the strand just ahead of the face of the support; hence, the available transfer length was the distance measured from the centerline of the support to the end of the beam plus 4 inches. Table II contains the estimated transfer lengths predicted by the current AASHTO recommendations and the proposed Zia and Mostafa equation for the six tests evaluating the strand anchorage mode of failure.

Table II - Transfer Length

Beam	f'_{ci} (ksi)	f_{si} (ksi)	f_{se} (ksi)	AASHTO L_t (in)	Zia L_t (in)	Transfer Length Available (in)
☞ Type I-3	5.84	193.3	184.6	30.8	20.2	10
☞ Type I-3A	"	"	"	"	"	22
☞ Type I-4	5.84	193.3	187.8	31.3	20.2	10
Type I-4A	"	"	"	"	"	28
☞ Type II-1	5.98	193.5	183.5	30.6	19.7	10
Type II-1A	"	"	"	"	"	28

☞ indicates shear tension failure

$$\text{AASHTO - } L_t = \frac{f_{se}}{3}\, d_b \qquad \text{Zia - } L_t = 1.5\,\frac{f_{si}}{f'_{ci}}\, d_b - 4.6$$

$$d_b = 0.5 \qquad\qquad d_b = 0.5$$

Based on the results of these tests, the presence of the transfer length extending into the shear span can be defined as the criteria leading to a failure resulting from the formation of a web shear crack across the prestressing strand producing loss of bond. As can be seen in Table II, the AASHTO recommendations would indicate a shear tension failure for all six tests. For these tests however; the proposed equation by Zia and Mostafa would seem to give a better estimate of a shear tension failure. As in the case of beam Type I-3A where the predicted transfer length, 20.2 inches, was less than the available transfer length of 22 inches, the presence of a web shear crack did not result in immediate failure. The formation of the web shear crack did not produce slip in all of the strands and the beam was capable of maintaining the load which produced the crack. An additional load was required to produce the full loss of bond. Hence, the test data from this study indicate that the proposed Zia and Mostafa equation seems to better predict the transfer length required to prevent shear tension failures. The current AASHTO recommendations are shown to be more conservative in this respect.

CONCLUSIONS AND RECOMMENDATIONS

This paper has presented an evaluation of the ultimate behavior of higher strength concrete prestressed I-beams loaded in shear and bending with the major focus towards the mode of failure defined as shear tension. The formation and propagation of a web shear crack across the transfer length of prestressing strand near the simple support of an exterior span has been shown to lead to premature failure due to the loss of bond between the strand and the concrete. This loss of bond results in the inability of the beam to maintain static equilibrium at the support; thus restricting mobilization of the web reinforcement. Hence, the load carrying capacity of this type of failure is only the load which produces a web shear crack regardless of the presence of web reinforcement. The use of passive reinforcement consisting of straps and deformed reinforcement encasing the prestressing strand near the support region did not show significant beneficial effects in preventing this mode of failure.

Possible detailing solutions to prevent this problem are:

1. Keep the transfer length of the prestressing strand behind the face of the support. This can be achieved by increasing the length of the beam which will result in some geometry changes in the abutment design.
2. Increase the design compressive strength at transfer to reduce the required transfer length. This assumes the use of the Zia and Mostafa equation in predicting the transfer length.
3. Use end blocks to increase the cracking capacity of the beam near the end region. This would prevent a crack from reaching the transfer length of the prestressing strand.
4. Extend the prestressing strand past the end of the beam and provide some sort of positive anchorage, ie. integral end bents, mechanical anchorage or passive reinforcement.

These recommendations will add to the cost of the structure; however, to develop the full shear capacity of the beam resisting factored loads for members with relatively thin webs, it is essential that the transfer length of the prestressing strand be protected from any additional increase in stress due to web shear cracks.

NOTATION

The notation used in this report is that of ACI 318-83 [6].

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UPDATE OF LRFD AASHTO DESIGN SPECIFICATIONS

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SYNOPSIS

In response to a high level of interest among State Bridge Engineers, the AASHTO Subcommittee on Bridges and Structures requested that the National Cooperative Highway Research Program (NCHRP) conduct a study to recommend an outline for an updated AASHTO Bridge Specification. The scope of the study required an identification of gaps and inconsistencies in the present specifications and an assessment of the feasibility of a revised probability-based specification. This study has been completed by Modjeski and Masters, and the results are reported in NCHRP 20-7/31 Final Report, "Development of Comprehensive Bridge Specifications and Commentary". A subsequent two-tier research effort was developed to rewrite the AASHTO Specifications into a probability-based LRFD format. NCHRP 12-33 has been developed in an effort to achieve the ultimate goal of the AASHTO Subcommittee, a more comprehensive, yet user-friendly bridge specification, based upon the latest available research, which will achieve more uniform margins of safety.

INTRODUCTION

The AASHTO Subcommittee on Bridges and Structures has recognized the need for clear, practical specifications based on the best current technology. In recent years, some State Bridge Engineers have called attention to the potential advantages of developing a completely new comprehensive specification, together with an accompanying commentary.

In response to a high level of interest among State Bridge Engineers, the AASHTO Subcommittee on Bridges and Structures requested that the National Cooperative Highway Research Program (NCHRP) conduct a study to recommend an outline for an updated AASHTO bridge specification. The scope of the study required an identification of the gaps and inconsistencies in the present specifications and an assessment of the feasibility of basing the revised specifications on a probability-based philosophy.

This study has been completed by Modjeski and Masters, the NCHRP's Contractor, and the results are reported in NCHRP 20-7/31 Final Report, "Development of Comprehensive Bridge Specifications and Commentary". This report identified many areas where current bridge design technology and design practice are not reflected in the existing AASHTO specifications. Additionally, it recommended that new specifications be developed based on probability-based Load and Resistance Factor Design (LRFD) concepts. The study also recommended that a comprehensive companion commentary be developed.

As a result of the recommendations of the feasibility study included in NCHRP 20-7/31, a new project, NCHRP 12-33, also entitled "Development of Comprehensive Bridge Specifications and Commentary" has been underwritten. The objective of this project is the development of the recommended probability-based LRFD bridge design specifications and commentary for consideration by the AASHTO Subcommittee on Bridges and Structures for adoption. The new specifications are expected to draw heavily from recent developments in bridge design practice throughout the world, as well as from recently completed and ongoing bridge research.

ORGANIZATION OF EFFORT

A two-tier research effort was developed to rewrite the AASHTO Specifications into a probability-based LRFD format (see Figure 1). The upper tier of the effort will have the responsibility of ensuring that the entire document is consistent in concept, content and applicability. This includes providing the general concepts, format and style of the document to the lower tier. The upper tier is directly responsible to the NCHRP Panel and ultimately the AASHTO Subcommittee. It is the responsibility of the individuals at this level to produce an easily useable document within the prescribed time frame for completion. The lower tier will consist of several small committees or Task Groups analogous to the Ontario Highway Bridge Design Code (OHBDC) Technical Committees, each responsible for the actual writing of a section of the new specification.

The upper tier consists of the Principal Investigator's (PI) staff and a Code Coordinating Committee (CCC) consisting of practitioners and governmental officials and researchers. Modjeski and Masters has been chosen as the research agency providing the PI. The Chairmen of the Task Groups will be included on the CCC.

The PI is responsible for proposing the research team members and a preliminary table of contents outline. The final draft table of contents will be developed jointly by the NCHRP Panel and the CCC. Some further evolution of the table of contents, by the Task Groups themselves, can be anticipated during the development.

The lower tier will, for the the most part, consist of researchers (or research agencies), specialized in the technologies represented by the various sections of the new specification. Each section will have three or four individuals assigned to it. It will be the responsibility of this Task Group to develop the Section and Commentary assigned to it. Each of these Task Groups will have an AASHTO Technical Committee assigned to it to provide review.

RESEARCH APPROACH

The effort required to develop a new bridge design specification and commentary has been divided into seven tasks.

Task 1
Development of Task Groups and
Establishment of Fundamental Philosophy

This work has already begun with the selection of members of the CCC by the PI with the final approval of the selection by the Panel during their meeting at the 1988 AASHTO Bridge Subcommittee Meeting in Atlanta. This group will consist of nine members, two from the PI group and seven additional individuals approved by the Panel. It is further envisioned that five of these individuals would chair most of the primary Task Groups, e.g., General Requirements; General Features; Loads; Analysis of Structures; Deck Systems; Concrete Structures; Metal Structures; Timber Structures; Code Calibration; Foundations and Retaining Walls; and Soil-Structure Interaction Systems. This implies that an individual CCC member could be chairperson of, perhaps, two Task Groups. This will help to centralize information and keep the CCC down to a size which would be workable and efficient. The remaining two CCC members will serve in an at-large capacity. Practical experience has been an important attribute sought in these key individuals.

With the CCC effectively in place, the actual Task Groups will also be developed. A list of Task Group candidates provided by the Panel, the rosters of various technical committees and lists of technical colleagues will be consulted in arriving at the group of engineers nominated to the Panel for approval. Regional balance will also be considered. As in the case of the CCC, relatively compact Task Groups, numbering three to four individuals, including the chairperson, are envisioned. Those members of the CCC who also have responsibilities as Task Group chairpersons will be asked to nominate the members of their respective Task Group(s) as part of their responsibility as members of the CCC. The PI and the Panel will review and approve the nominations.

Where clear industrial associations, such as AISC, ACI or the Timber Institute, can be identified, they will be given the opportunity to contribute working representation on the Task Groups. The State Departments of Transportations and the Federal Highway Administration will be invited to contribute engineers from their respective staffs, who have special expertise to contribute to this project.

The PI and the CCC will jointly develop the fundamental philosophical premise for the specification as part of this Task.

Task 2
Identify Detail Needs
and
Develop a Strawman

The PI and the CCC will prepare a strawman specification consisting of a very detailed listing of provisions, with an indication as to whether each given provision will be adapted from:

- o the current AASHTO Specification
- o the current OHBDC (or possibly the forthcoming OHBDC revision)
- o the AISC LRFD Specification
- o available research
- o some other source, which will be identified.

This document will serve as the skeleton which will be "fleshed-out" by the appropriate Task Groups. Therefore, the Task Group to which each provision, or group of provisions, will be assigned will also be noted in the strawman. The PI is currently developing a preliminary strawman for consideration by the CCC as a starting point.

Task 3
Development of Draft Chapters and
Commentaries and Review by the PI,
CCC, Panel and AASHTO Bridge Subcommittee

This task comprises the actual word-by-word, line-by-line development of the detailed specification provisions and commentary material, except for the load factors and resistance factors. It is anticipated that almost 40 percent of the total effort on the project will go into Task 3. The work will be performed basically at the Task Group level, with the CCC members acting not only as chairpersons of various Task Groups, but also contributing as working members of those Task Groups. The term "working members", as it relates to members of the CCC working and Task Groups, means exactly that - the members of the CCC are expected to participate in the actual wording and development of specifications, provisions and commentaries. In fact, the members of the CCC are expected to be the backbone of the actual writing effort, drawing on the resources and talents of the members of their Task Group, as best fits their individual chapter requirements.

The PI's effort will be devoted to coordinating the Task Groups, assuring complete coverage (this should be well-established through the use of the strawman) and providing general supervision and coordination.

It is entirely possible that coordination will be more effective if some of the Task Groups are started in overlapping fashion, so that some of the Task Groups could be nearly finished as others are starting. The decision on scheduling the start of various groups will be made with the PI and the CCC.

The Final Report for Project 20-7/31 indicated that there were several gaps and inconsistencies in the existing specification which should be addressed during the development of the new specification. Those gaps and inconsistencies, which are expected to receive the highest priority for inclusion, are shown below, along with the Task Group to which they will be assigned:

- o Development of a comprehensive commentary - all Task Groups
- o Development of guidelines for the use of more refined methods of design of girder bridges, e.g., grid and finite element methods - Analysis of Structures
- o The adoption of the isotropic reinforcement design procedure as an alternative to the current specification - Deck Systems or Concrete Structures
- o The improvement of effective flange width equations by adoption of design procedures similar to those in the OHBDC - Analysis of Structures
- o The inclusion of the segmental concrete specifications currently being developed under Project 20-7/32, with the concurrence of AASHTO (this is an area where initial treatment for serviceability and conventional strength design may be necessary) - Concrete Structures
- o Adoption of some more complete design provisions for multi-web concrete girders - Concrete Structures or Analysis of Structures
- o Provision of additional guidelines for fatigue of prestressed concrete girders and limitation on use of shielded or blanketed strands - Concrete Structures
- o Identification of either, or both, CEB/FIP and ACI as adequate procedures for evaluating creep and shrinkage - Concrete Structures
- o Recognition of the Space Truss Analogy procedure for shear, torsion and special detailing of reinforced and prestressed concrete - Concrete Structures
- o Inclusion of provisions for unsymmetric plate girders currently under consideration, and those which may be developed during the course of the subsequent specification development project - Metal Structures
- o Development of ultimate-strength capacity for friction joints and bolted construction - Metal Structures

One of the major issues facing the Task Group on Loads will be consideration of a revised live load model. This issue was discussed in some detail in the Final Report for Project 20-7/31 and will be an area where extensive input from AASHTO will also be useful.

During the review phase, it is anticipated that the PI and the CCC will be reviewing the entire document, not just those areas for which they were personally responsible. Members of each individual Task Group will deal almost exclusively with the review of the portions for which they are responsible. Note that this places a substantial burden for review on the PI and the CCC. The Panel and AASHTO will also be active during this review function, and it is entirely appropriate that members of the PI and CCC groups make presentations to the AASHTO Technical Committees during the 1990 and 1991 AASHTO Subcommittee Meetings, if not sooner. These presentations are an additional requirement for these two groups.

The Commentary submitted with the first draft specification will probably be significantly different than the final Commentary. This initial Commentary to the first draft will probably be oriented more toward providing information as to why one particular provision was selected rather than another possible provision, why a particular equation or chart was used rather than some other one. It will also be used as a vehicle to assist the AASHTO and Panel reviewers, indicating where additional input may be required or be useful.

During Task 3, information will also be developed to demonstrate the impact of varying the reliability index on construction cost for five or six common structures as an aid to the Panel and AASHTO in selecting the target reliability index. The structural forms selected for this study will have a geographical dispersion, and the assistance of various State Departments of Transportation will be enlisted. This effort will fall to the PI and to the Task Group on Calibration.

Task 4
Calibration

Calibration is the process of calculating load and resistance factors to achieve the desired level of reliability. This is clearly specialist work. The PI and the CCC will supervise the calibration effort and review the results in detail. However, they will not actually calculate the load and resistance factors.

The calibration effort is seen as extending over a considerable portion of the project, although the choice of the target reliability index can be postponed to near mid-calibration time frame. It was proposed that initial discussions on choosing the reliability index receive serious attention at the 1989 AASHTO Subcommittee Meeting, with a target of choosing the reliability index at the 1990 AASHTO Subcommittee Meeting. At the 1988 Subcommittee Meeting the Bridge Engineers were reminded of the significance of the reliability index, both to the physical world, and to the schedule for developing a new specification. The Final Report to NCHRP Project 20-7/31 suggested that AASHTO establish a Technical Committee on Bridge Code Calibration and this Technical Committee has been formed.

Task 5
Revision to First Draft
and
Review of Second Draft

Following receipt of comments to the first draft, that draft will be upgraded to second draft status by addressing the comments that have been received and by adding, at least draft-level results of the calibration process in the form of load and resistance factors. The second draft will then be reviewed by the PI, CCC, Panel and AASHTO. At this time, review periods are envisioned to become somewhat more compressed as major comments should have been received as a result of the review of the first draft.

As with the first-level review, review of the second and subsequent drafts will fall heavily to the PI and the CCC, with the Task Groups reviewing their specific areas of responsibility.

Task 6
Revisions to the Second Draft and Review

This process will be basically similar to Task 5, but, again, somewhat more compressed in time. At the end of this task, the Specification and Commentary should be in a form suitable for review by AASHTO for adoption.

Task 7
Coordination with Other
Ongoing Research and Projects

Many research projects, currently being funded by the NCHRP, AASHTO or individual states, may bear on the development of this specification. Among those that come immediately to mind are the projects dealing with LRFD design of foundations, the rewriting of Division 2 of the specification, NCHRP Project 12-23 relating to rewriting the Manual for Maintenance Inspection, and the FHWA Project on comprehensive experimental and analytical evaluation of bridges. It is anticipated that several of these projects will yield provisions, and possibly virtually entire chapters, of the proposed specification. If these deliverables are to be received in a form suitable for direct inclusion in the new specification, considerable coordination will be necessary to be sure that they fit technically and philosophically into the structure of the new code. This Task will fall almost entirely to the PI, with some specialized input from members of the CCC.

CONCLUSION

Project 12-33 has been organized and the seven tasks outlined above developed in an effort to achieve the ultimate goal sought by the AASHTO Subcommittee in initiating these studies. This goal is a more comprehensive, yet user-friendly bridge specification, based upon the latest available research, which will achieve more uniform margins of safety.

While probabilistic theories will be employed during the codification process, the user of the specification will require little or no knowledge of probability theory to accurately apply the specification. In fact, in application, the LRFD provisions will be very similar to the current Load Factor Design (LFD) provisions.

AASHTO's original adoption of LFD as an acceptable design methodology represented a revolutionary step in the development of the AASHTO Specifications, i.e., the adoption of the concept that factors of safety should be variable and be a function of the uncertainty of the components of total load. In comparison, the proposed updating and conversion of LFD to probability-based LFRD is merely evolutional. The original (and existing) LFD provisions were, in effect, calibrated at a single point based on past experience, whereas, the proposed conversion to probability-based provisions represents a broad attempt at calibration to a preselected level of reliability.

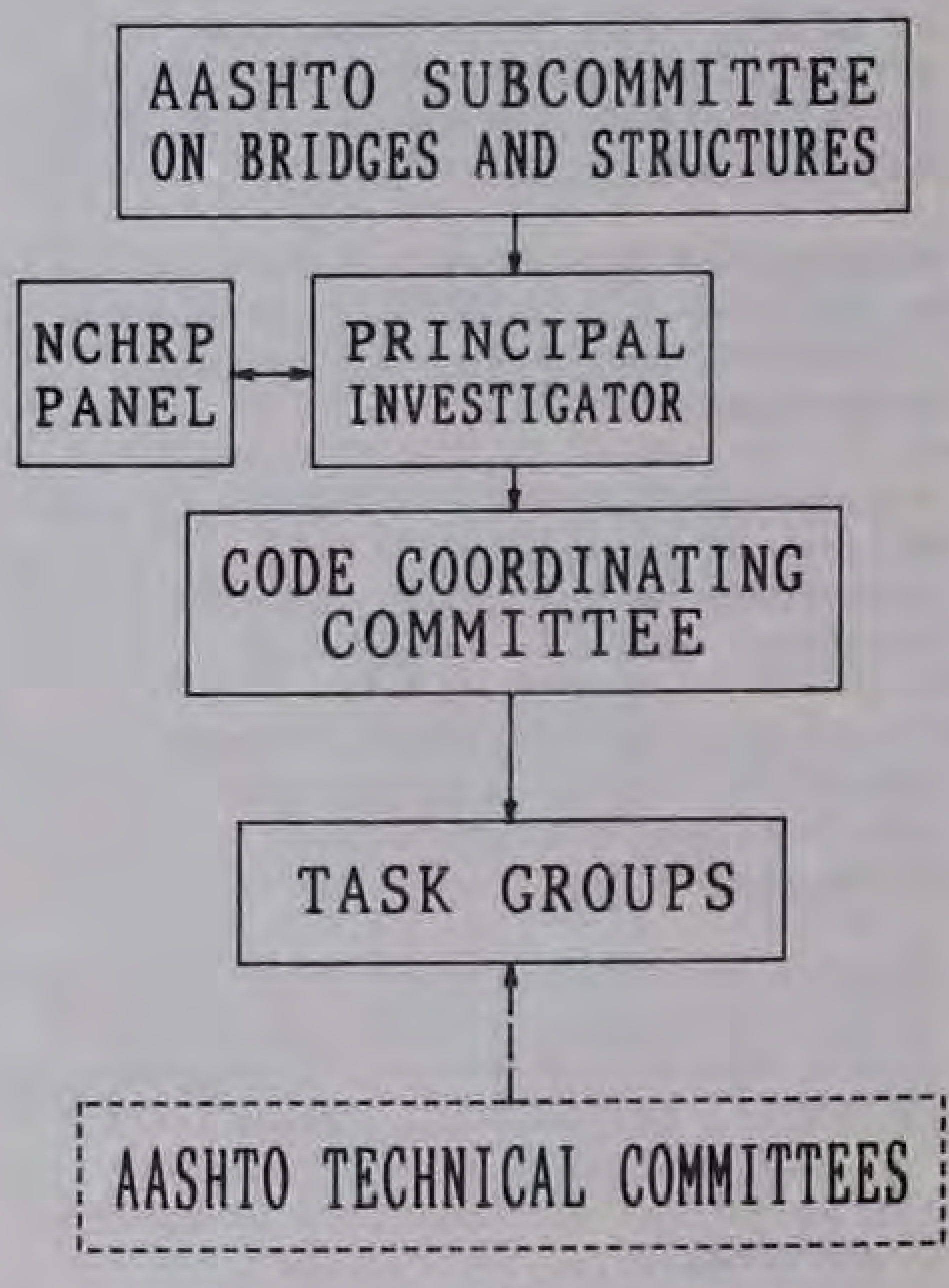

Figure 1 - Organization Chart

SESSION IIIA

CRACKING AND FAILURE OF DECK SLAB IN GIRDER-SLAB BRIDGES

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SYNOPSIS

In an experimental investigation, both static and fatigue tests were conducted on a series of girder-slab type, reinforced concrete bridge deck panels to determine the mode of failure under the action of a simulated wheel load. The influence of distribution steel on the crack pattern was studied, and the detrimental effect of crack growth on the punching resistance of the slab observed. Experimental results show that punching is the only expected mode of failure of a normally designed reinforced concrete deck slab, implying lesser requirements of tension steel and that critically nucleated crack growth may weaken the punching resistance.

INTRODUCTION

Several occurrences of localized pot hole type failures (Fig. 1) and an alarming level of cracking in concrete deck slabs of girder-slab bridges in Saudi Arabia prompted a detailed investigation [1] which included, among others, in-situ nondestructive testing for concrete quality and strength, gathering of axle load data and experimental study on simulated deck panels to seek an explanation and to suggest recommendations to avert such future occurrences.

In view of the fact that the decks were subjected to excessive loading from grossly overweight trucks, the experimental work was planned to examine the effect of heavy loads on the behavior of composite girder-slab panels and to record the modes of failure. Major findings are presented here to demonstrate the crack patterns under subcritical load levels, the mode of failure and the damaging effect of flexure cracks on the punching capacity.

EXPERIMENTAL PROGRAM

The experimental work focussed on three items: (a) static and fatigue tests on simulated deck panels, (b) crack pattern under a subcritical moving patch load in relation to the amount of distribution steel, and (c) punching capacity of a deck slab weakened by nucleation of cracks, oriented critically.

Figs. 2a-2c show the cross sections of the slab-on-beam panels used in this experimental work. The cross-section of Fig. 2a was used in static and fatigue tests by introducing three different amounts of both top and bottom main steel. To observe the influence of distribution steel on flexure cracks, panels of cross section shown in Fig. 2b were used. Distribution steel amounted to four values: 100% (isotropic), 67%, 37% and 18% of the main tension steel. Crack patterns were recorded for the case of a load moving from one station to another along the length of the panels. For the study of possible detrimental influence of flexure cracks on the punching capacity, panels whose cross-section is shown in Fig. 2c were cast with a central embedded thin metallic insert in the shape of a frustrum of a cone. Influence of the flaw resulting from the propagation and nucleation of cracks was modelled experimentally by the inclusion of this embedded insert. The geometry of the frustrum was varied by introducing different angles, bottom diameters and the height.

The panels selected for tests were not designed following a prototype. Instead, consideration was given to use specimens as large as possible to eliminate small size effects and to reasonably proportion the models keeping the span-thickness ratio of the slab within the practical range. All panels were cast using a water-cement ratio of 0.53 and cement content of 24 lb per cubic ft. The compressive strength of concrete determined from 3 x 6 in cylinders varied from 3700 psi to 4400 psi.

In all tests, the loading was confined to a single load applied to the top surface of the panel by an MTS actuator through a pad to simulate the action of a wheel load. The loaded area varied for different panels. In some panels, a few selected rebars were strain gaged to record strains. Tests measurements included record of the failure load and the mode of failure, crack mapping and record of deflection and strains.

From static tests, the loading was increased monotonically till failure, recording the pertinent test data. Fatigue tests being supplementary to a past study [2], were limited to the case of a maximum load intensity of 60% of the static failure load to note the endurance limit.

RESULTS AND DISCUSSION

For static tests, three different loaded areas, namely 3 in x 6 in, 4 in x 8 in and 8 in x 16 in, were used. Data presented in Table 1 for three different types of panels show that the load P_u increases as expected with the loaded area, as punching is the eventual mode of failure for all tests. For the size of bearing areas used, no noticeable change in P_u was recorded due to variability in the main transverse tension steel (Table 1). This is attributable to the fact that the strength of the panels was not governed by flexure. Although the rebars immediately under the load yielded prior to the failure, the rebars away from the critical ones showed rapid decline in the stress with distance from the loaded area. The localized yielding of rebars did not induce a flexure-controlled failure. Even for A3 panels which had the lowest amount of transverse steel, the ultimate load to cause flexural failure in accordance with the conventional theory would be much less than the recorded value of P_u (Table 1). Results confirm the previously held notion that the actual flexural capacity of such slabs is substantially higher than the computed value from a conventional theory which fails to account for the effect of the support restraint. Furthermore, the results also show that in view of the enhancement of the flexural capacity and the punching mode of failure, no significant advantage is attained by increasing the tension steel.

Limited fatigue tests using 3 in x 6 in bearing area and a load cycle ratio of $P_{min}/P_{max} = 0.1$ with $P_{max} = .6\ P_u$ showed that failure is unlikely to occur at cycles less than one million. Fatigue data presented in ref. 2 under a pulsating load with $P_{max} > .6\ P_u$ showed rapid reduction in fatigue life with higher value of P_{max}. All panels in fatigue tests, regardless of the maximum load level, failed in punching mode, showing failure similar to that recorded in static tests. For a deck slab in a girder-slab bridge, failure due to rebar fatigue is unlikely to occur. In view of the well accepted notion that a moving load is more critical than a pulsating load at a fixed location, a value of $P_{max} = .5\ P_u$ has been established as a safe threshold value to inhibit punching failure of a deck slab due to fatigue.

With regard to the formation of crack patterns in relation to the amount of distribution steel, it was noted that at early stage of cracking, the amount of transverse cracking was less but the longitudinal cracks were wider for slabs with smaller amount of distribution steel. As load increased, cracking in both transverse and longitudinal directions intensified, and the emerging pattern showed radial divergence from the load locations. The fan-shaped crack growth under a subcritical load level in all panels (Fig. 2b) showed a high degree of similarity. Although a closer examination would show some differences in the crack pattern at early stage of cracking, no definite distinction can be made at higher loading.

Test data from Panel A3 showed two interesting findings. Generally the punching failure zone can be idealized as shown in Fig. 3 in confirmation with the experimentally observed failure pattern. In order that the emerging cracks become detrimental to the punching resistance of the deck slab, the crack formation must depict two essential features:

Table 1 - Static Load Test Data

Panel	Main tension steel	Loaded Area (in x in)	Ultimate Load P_u (kips)
A1	5/16 in dia bars @ 4 in c/c (8 mm @ 100 mm)	3 x 6	21.6
		4 x 8	25.4
		8 x 16	37.9
A2	1/4 in dia bars @ 4 in c/c (6 mm @ 100 mm)	3 x 6	23.2
		4 x 8	24.7
		8 x 16	36.6
A3	5/32 in dia bars @ 4 in c/c (4 mm @ 100 mm)	3 x 6	23.1
		4 x 8	25.4
		8 x 16	36.3

(i) the randomly oriented cracks must nucleate to form an enclosed envelope, defacto inducing a surface of separation and (ii) concomittantly the depth of this cracked zone must propagate upwards to a close proximity to the failure surface (Fig. 3). Such an occurrence which may result from random cracking due to excessively heavy wheel loads would endanger the safety of a slab by weakening its punching resistance.

A number of tests on panels with different sizes of embedded conical inserts affirms the proposed hypothesis. For a flaw that was confined well within the failure zone, the punching capacity was not observed to decline appreciably. Varying the angle of the conical frustrums, the critical angle was observed to be in the vicinity of 20° for the reinforced concrete deck slab in consideration.

In view of the experimental findings, it has been concluded that a reinforced concrete deck slab in a girder-slab bridge should have a minimum thickness to inhibit punching and the main tension reinforcement in the slab need not be based on a flexure theory which disregards the prevailing arch action. The failure of the deck slab under a wheel load will be due to punching for a certain minimum amount of tension steel and no significant increase in load capacity will ensue by increasing the tension steel.

CONCLUSIONS

Based on this study, the following conclusions are drawn.

1. Support restraint enhances the flexural capacity of a deck slab in a girder-slab bridge to a large extent and in view of this, punching is the only expected mode of failure for a concrete deck slab of normal design in a girder-slab bridge.

2. Punching capacity can significantly diminish if the flexure cracks are of sufficient depth and their critical nucleation and orientation constitutes a weakened zone.

3. The emerging cracks under subcritical load level tend to diverge in all directions from the loaded area. Lesser amount of distribution steel tend to produce lesser transverse cracks in the early stage of cracking.

4. For a deck slab of girder-slab bridges, the fatigue failure under a patch load is unlikely to occur if the ratio of the applied load to the failure load is less than 0.5.

ACKNOWLEDGEMENT

The support of the King Abdulaziz City for Science and Technology (KACST) and the King Fahd University of Petroleum and Minerals is thankfully acknowledged.

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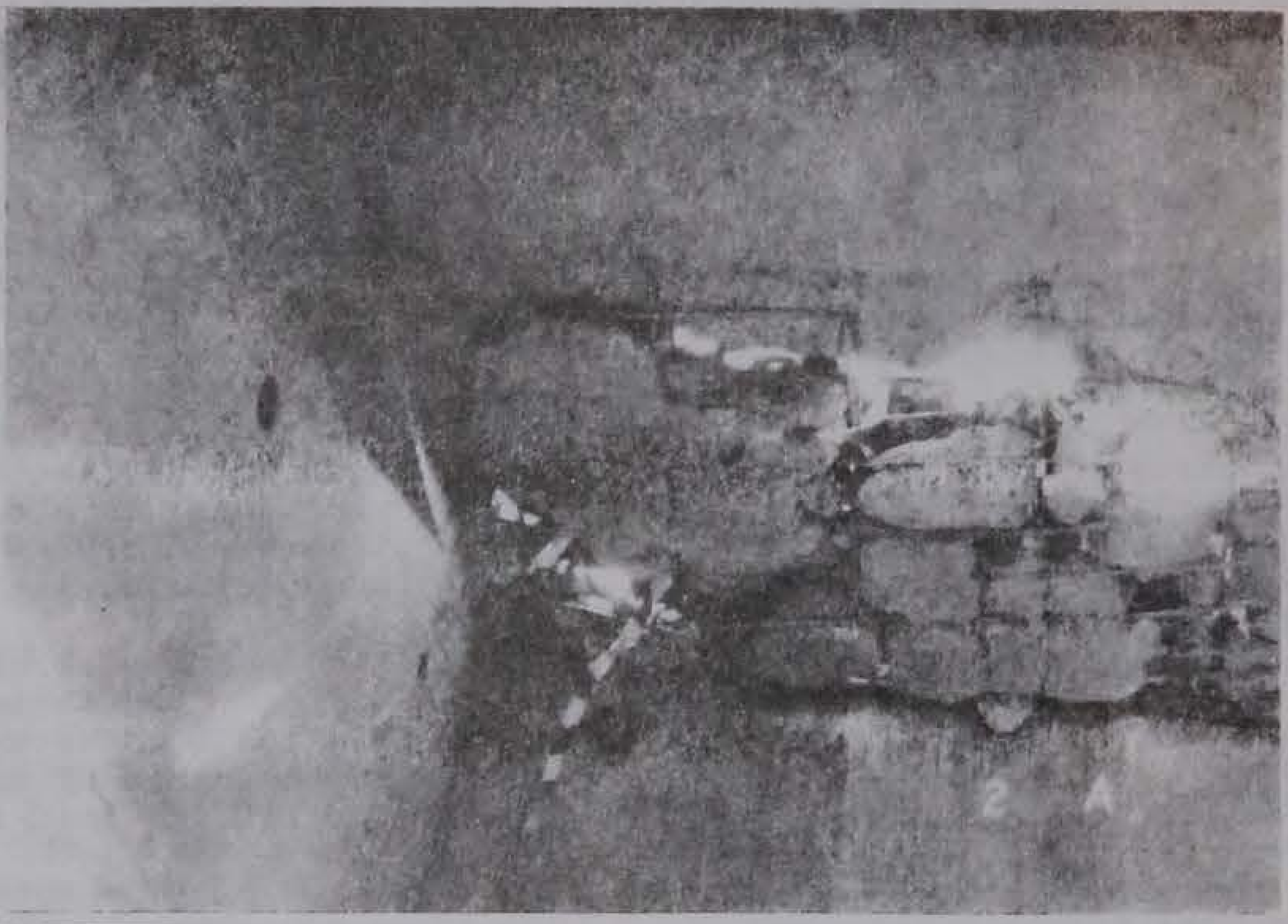
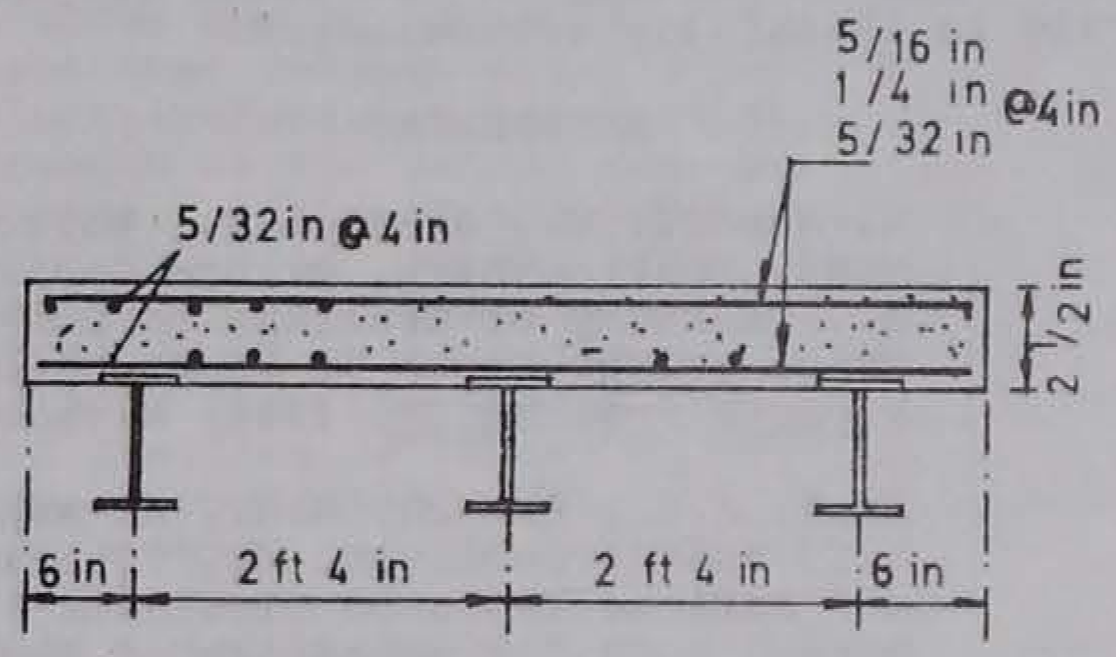
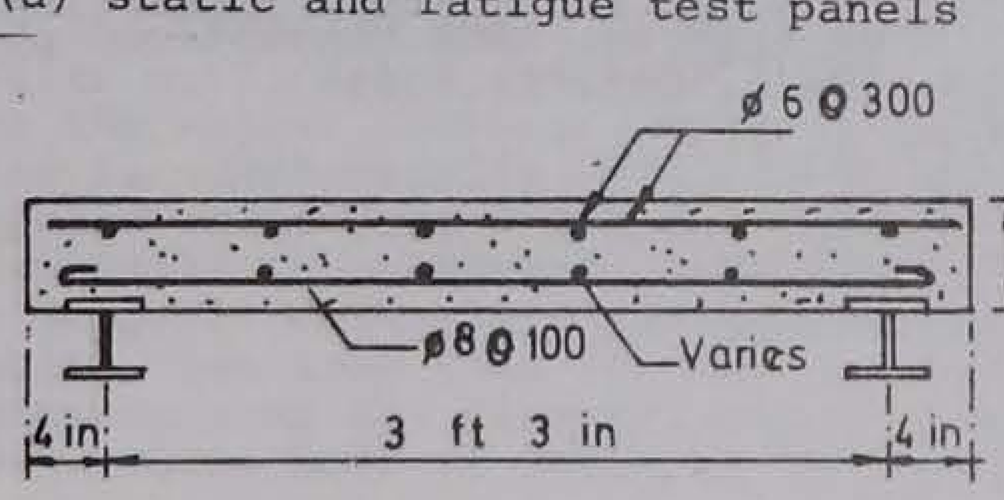


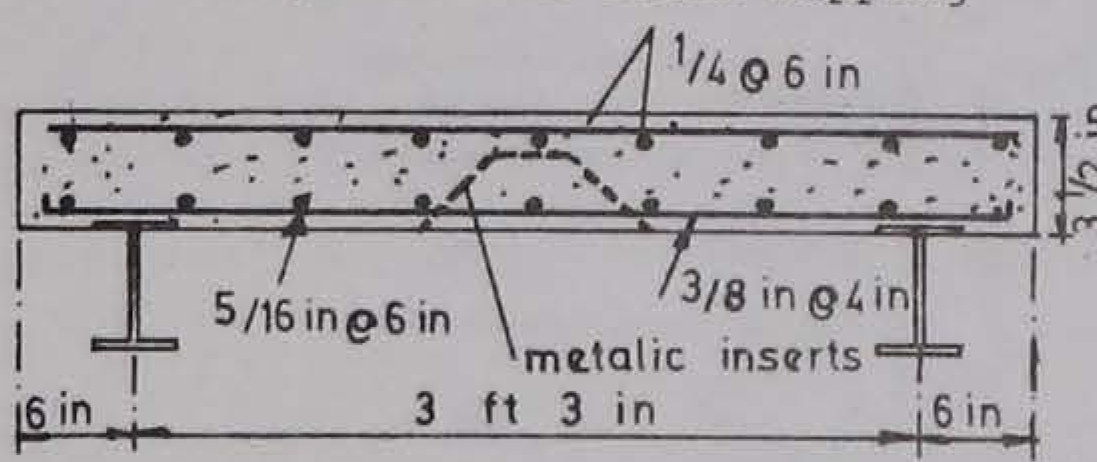
FIG. 1 - A Pothole Type Failure of Deck Slab



(a) static and fatigue test panels



(b) panels for crack mapping



(c) punching test panels with inserts

FIG. 2 - Details of Test Panels

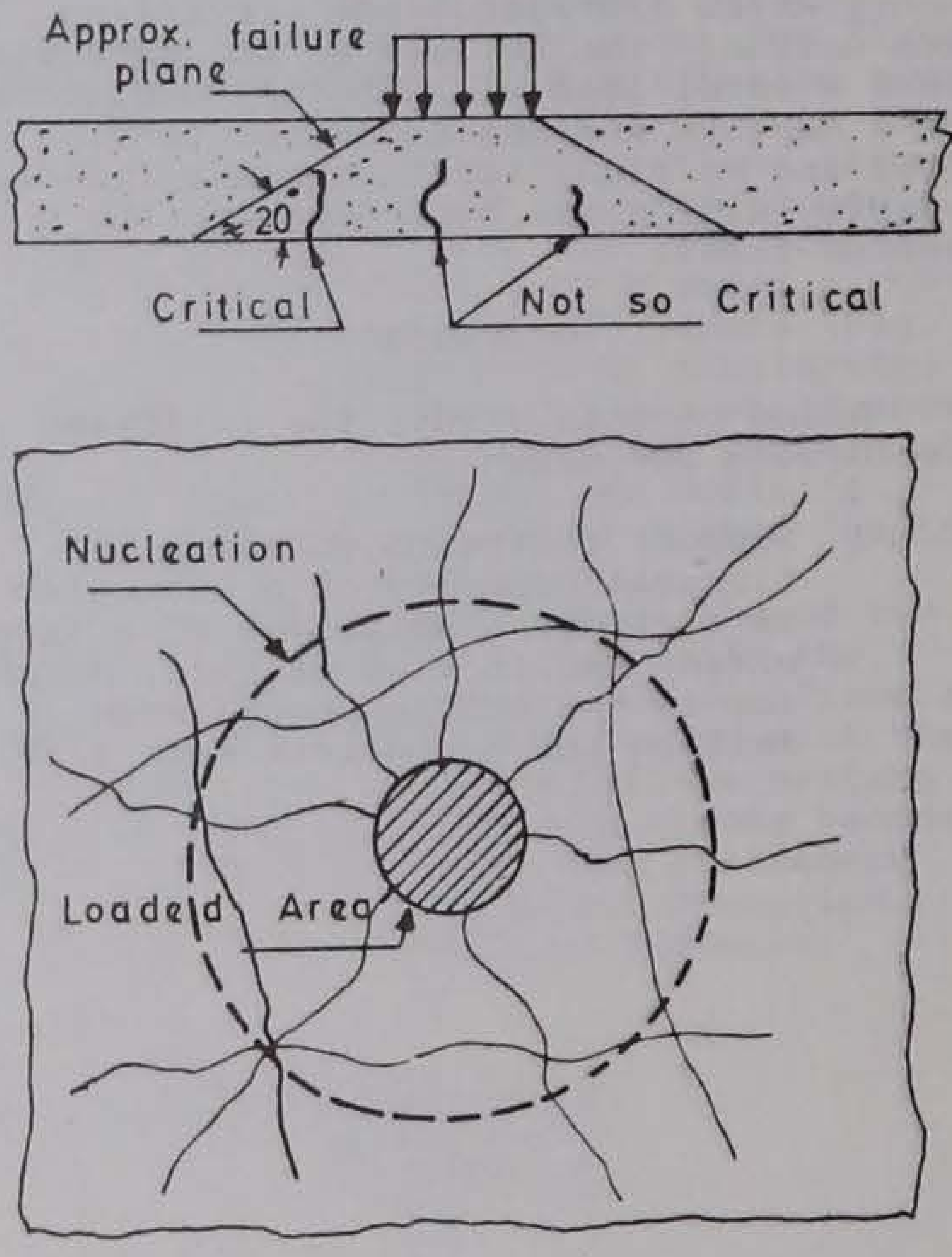


FIG. 3 - Punching Failure

CRACKING OF WELDED STEEL GIRDERS OF HIGHWAY BRIDGES

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SYNOPSIS

The main supporting elements of a 3-span continuous bridge (150', 180', 150') in New England were two longitudinal welded steel plate girders. A full height crack developed in one of these fracture critical girders, necessitating the closure of the Bridge to traffic. A research study was initiated of this failure. The results showed that the crack started in a "bad" weld between the web and bottom flange and propagated through the bottom 1-1/4" flange, the full 92" web into the upper 1-1/4" flange. The Bridge meets AASHTO catagory E fatigue category, and special techniques are suggested to improve the inspection and evaluation procedures for such structures in an effort to predict their useful fatigue life.

It is estimated that between 1950 and 1965 approximately 2000 bridges were built in the United States using welded steel plate girders as the primary structural elements. Some of these fracture critical (or non-redundant) bridges also possess welded details for stiffeners, connection plates, diaphragms and wind bracing attachments. Moreover, many plate girders have full section welded field splices.

These bridges, due to their fatigue sensitive details, are nearing the limit of their practical fatigue life. In a number of instances, cracks developed in the webs, flanges, secondary members and connections resulting in uncertainty about the integrity of these bridges and their safety.

A case in point was a non-redundant bridge in New England in February of 1987, after an extended cold spell, when a major crack in a main girder was discovered accidentally by a passing local policeman. The crack was located in the center span of a 3-span continuous bridge extending through the bottom flange and halfway up the 92" web. The officer notified the Department of Public Works and the bridge was closed to traffic within minutes, but not before the crack had propagated through the entire web and most of the top flange. A half inch of steel in the upper flange and the concrete deck that was connected to the other plate girder kept the bridge from total collapse into the river below.

A. G. Lichtenstein & Associates (AGLAS) were called in by the bridge owner to assist in evaluating the condition of the remaining members of the bridge and report on their condition. As a first step, AGLAS suggested that large mounds of sand and gravel be placed on the anchor spans to improve the stability of the cracked portions of the girder. It was too dangerous to send inspectors on the unstable superstructure for a "hands-on" inspection and testing. Therefore, a temporary bent was designed by AGLAS and erected by Contractor under the girder, next to the crack. AGLAS then performed an in-depth inspection and field testing of the entire structure. The inspectors found numerous cracked and damaged welds, connection plates and diaphragms. It became obvious that to repair and retrofit this bridge could not be accomplished with any reasonable degree of engineering confidence. The superstructure was eventually removed and replaced on the existing piers with a redundant multi-girder system.

AGLAS was given the added assignment of preparing a failure analysis of the cracked plate girder. Dr. Regis Pelloux of the MIT Department of Materials Science and Engineering and a research oriented testing laboratory (Manlabs) were engaged by AGLAS to assist in this study. It was intended to determine the probable causes and sequence of events that led to the cracking of the north girder. The gained knowledge could then be utilized in improving evaluation techniques of other similar fracture critical bridges to predict their remaining useful fatigue life.

The study included the following engineering activities:

1. Macrophotography of the fracture features.
2. Removal of 2" x 2" section ocontaining the fracture origin.
3. Removal of a 2" x 2" section of the flange-web junction for metallography and macrophotography of the welds. This section is located .5 inch behind the fracture plane.
4. Macrophotography of the fracture features before rust removal and after cleaning off the rust.
5. Polishing and etching of the T joint section followed by macrofractography, metallography and microhardness testing of base metal (flange and web) and weld joints.
6. Removal of samples for chemical analyses.
7. Removal of samples for Charpy bars (impact testing).
8. Scanning Electron Microscopy (SEM) analysis of the fracture features near the origin at magnifications from 25x to 10,000x. Microchemical analysis of the welds with an energy dispersive attachment (EDAX) to the SEM.
9. Fatigue life calculations as per AAHSTO specifications (17 years).
10. Fatigue life estimates utilizing new formulas proposed by Dr. Fred Moses under a TRB grant (32 years).

The primary findings and conclusions are summarized herewith:

* The steels in the web and flange of the plate girder meet the requirements of ASTM A373-54T and 58T.
* The microchemical analyses of the welds show high contents of Manganese and Sillicon. Hardness measurements confirm that the welds had a high Rockwell C number (Rc36) due to the high contents of Manganese and Sillicon.
* The Charpy tests show the flange steels met all AASHTO requirements for Zone 2, while the Charpy value for the web was below the requirements at 40°F.
* Photographic studies of the fracture planes and welds pinpointed the origin of the crack initiation.
* A thumb nail fatigue crack was initiated from a small weld defect (.10 inch deep) transverse to the north weld between the lower flange and the web.
* The fatigue crack grew to a depth of .5 inch through the weld before fast crack propagation took place under a combination of low temperature and live loads.
* Failure of the north girder was due to the propagation of a long vertical brittle (cleavage) crack on a cold day under live loads. This failure is typical case of brittle fracture of low carbon steels below the ductile to brittle transition temperature.

The Bridge had been inspected routinely by the owner about 8 months pior to the accident and no unusual conditions were found nor reported. The question lingers, could the improper weld and/or the beginning of a crack have been discovered doing a routine inspection? We think not. A more reliable method would be the utilization of an in-depth specialized inspection for fracture critical bridges. Such an activity would include the following:

1. Search the project files of the construction of the Bridge to review the shop drawings and x-ray films of the weldments.
2. A magnifying-glass inspection of the welds in the web where attachments were made. Slight horizontal cracks in the paint around weldments would denote questionable welds and possible cracking of steel.
3. Obtain traffic data and reliable information on the percentage and weights of truck traffic, and "regular" overloads.
4. Perform calculations of the stress range at the various fatigue sensitive locations of the bridge, by conventional or newly developed formulas.
5. In special situations place strain gauges at these locations and establish the stress range via a load test.

It is possible that even with this major effort and care the initiation of fatigue cracks may not be discoverd in time. The useful fatigue life calculations might also result in substantially diverse values. Still these special inspections must be undertaken as they will yield more reliable data than the routine inspections or, even worse, no inspections.

The phenomenon of cracking of welded plate girders has been recognized by FHWA, AASHTO, TRB and other research organizations. Much attention has been given to the special procedures described above. We should, however, continue the research and improve even further the reliability of inspecting fracture critical bridges because human life and millions of dollars are at stake.

CYCLIC TESTING OF FULL SCALE PIER WALLS

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SYNOPSIS

Pier walls are reinforced concrete wall elements used to support bridges, but the distinction between a column, pier, and pier wall is not clear. The present paper on pier walls offers the following: 1) a definition, 2) stiffness, strength, and performance characteristics, 3) applicability of code provisions, 4) specimens used in a test program, 5) test method, 6) conclusion, and 7) acknowledgement. The specifications pertaining to pier walls are unclear on some issues, but they require very close cross tie spacing throughout the height of pier walls. An objective of a test program underway at UCLA is to show that tie spacings can be increased without losses in seismic performance.

DEFINITION

Pier Walls are reinforced concrete wall elements which are based on footings or piles, and which provide vertical, transverse, and longitudinal support for the superstructure. The geometric quantities which define these elements are the height/thickness ratio and the height/width ratio. For pier walls these ratios satisfy the following limits

$$0 < \frac{h_c}{w} < 2.5 \qquad (1)$$

and

$$\frac{h_c}{t} \leq 20 \qquad (2)$$

in which h_c, w, t = clear height, width, and thickness of the pier wall respectively.

The width of pier walls is often equal to the width of the bridge, and typical heights are in the range of 30 ft, as might by appropriate for a low river crossing. Therefore the height to width ratio is one, more or less. A wall thickness of 18 in. is often used for a 30 ft. tall wall giving a typical height to thickness ratio of 20.

Reinforcement for pier walls consists of the following:

1. Two layers of horizontal tie reinforcement, each one with 2 in. cover, are placed parallel to the faces of the wall. Horizontal tie reinforcement is used primarily to resist inplane shear forces acting on the wall.
2. Two layers of vertical reinforcement are used; each layer is tied on the inside of the horizontal ties. Vertical reinforcement is used to resist out-of-plane bending and in-plane shear forces. A net or curtain is formed on each face by the horizontal tie and vertical reinforcement.
3. Cross tie reinforcement is placed perpendicular to the faces of wall. The cross ties serve as links between the two curtains of vertical and horizontal ties at the intersection of horizontal and vertical bars. The cross tie has a 135° hook on one end and a 90° hook on the other. The cross ties resist out of plane shear forces, provide confinement for concrete between the curtains, and give lateral support to the vertical reinforcement.

PIER WALL STIFFNESS, STRENGTH, AND RESPONSE

Pier walls have highly contrasting stiffness, i.e., high in plane and low out of plane. Consequently the response of the bridge due to earthquake inputs is highly polarized, with substantial response in the longitudinal direction of the bridge and negligible response in the transverse direction. For bridges without skew, in plane response of the pier wall corresponds with transverse response of the bridge while out of plane response of the pier wall corresponds with longitudinal response of the bridge. For discussions pertaining to bridge response in the present paper an unskewed bridge is assumed.

A similar polarization of strengths also occurs. Shear wall action dominates the behavior of the wall in plane; accompanying this are high levels of strength. Bending action is dominant during the out-of-plane deflections of the wall associated with longitudinal response of the bridge. The out-of-plane strength of the wall is limited to a low level by a mechanism which occurs when flexural hinges from simultaneously at the top and bottom of the wall.

An analysis of frequencies and strength coefficients of walls with differing heights and spans (Table 1) demonstrates the polarities and range of values. A lumped mass placed at the top of the pier wall was the idealization selected to represent the superstructure and uncracked section properties were used for Table 1 calculations. The results presented in Table 1 evince two properties of pier walls:

TABLE 1

Frequencies and Strength Coefficients of Pier Walls

Wall	Height (ft.)	Span (ft)	Transverse Frequency Hz	Transverse V_n/W	Longitudinal Frequency Hz	Longitudinal V_n/W
1	20	50	19.9	4.9	3.6	0.61
2	30	100	9.5	1.1	1.4	0.22

Notes:
1. Vertical reinforcement percentage, $\rho_v = 0.61\%$.
2. Wall Thickness = 18 in.; Wall width = 25 ft.
3. V_n/W or strength coefficient represents the shear strength associated with either shear or bending failure, whichever less, divided by the weight of the supported structure.

1. In transverse bridge response the frequency is large which indicates a rigid structure, and the strength coefficient in excess of one implies that failure in the transverse direction is highly remote.

2. When displacing in the longitudinal direction the frequency drops to 1.4 Hz which indicates a dynamically responsive direction for the bridge. A strength coefficient of 0.22 is too low for seismically active areas and yielding is likely when strong ground shaking occurs. If yielding does occur a closing of joints and pounding against the abutments will forestall collapse.

A longitudinal deck movement of 12 in. is the ultimate displacement which could occur in a three-frame pier wall bridge. This amount includes 6 in. for closing of joints and 6 in. for pounding against the abutments.

APPLICABILITY OF CODE PROVISIONS

Shear force and vertical reinforcement buckling associated with out-of-plane response of pier walls is the principal concern for cross tie reinforcement design. A 12 in. displacement in a 30 ft. tall wall is substantial, but out of plane shear failure and compression failure are unlikely to occur. For the wall configurations of Table 1 the axial and out of plane shear stress ratios satisfy the following limits

$$0.025 \le \frac{P_{uD}}{f_c' A_g} \le 0.047 \qquad (3)$$

$$0.50 \le \frac{V_n}{\sqrt{f_c'}\, wD} \le 0.75 \qquad (4)$$

in which P_{uD}, V_n = factored axial dead load and yield hinging shear respectively; f_c' = compressive strength of concrete; A_g, w, d = gross area, width, and flexural depth of the pier wall respectively. These ratios are in a low range especially if we consider that the wall is idealized in a severe way, i.e., as a column.

Bridge design specifications (1) require a column design about the weak axis and a pier design about the strong axis for pier walls. Reinforcement details required for columns by building codes versus bridge specifications are different.

There are three interpretations possible for design of lateral or cross tie reinforcement under the uniform building code (2) for a pier wall subjected to out of plane deformation. The interpretations are as follows:

1. Provisions (3) for reinforced concrete structures resisting forces induced by earthquake motions state that an element satisfying

$$\frac{P_u}{f_c' A_g} \le 0.1 \qquad (5)$$

shall be treated as a flexural member. Eq. (3) shows that pier walls will usually satisfy Eq. (5). The spacing of lateral reinforcement for beams in the plastic hinge zone is approximately d/4 and d/2 elsewhere in a beam. These provisions would pertain to vertical spacing of the cross ties.

2. Requirements (4) for walls state that vertical reinforcement need not be enclosed by lateral ties if vertical reinforcement area is not greater than 0.01 times the gross concrete area. Many pier walls satisfy this requirement.

3. Traditional lateral reinforcement design provisions (5) for tied columns if enforced for seismic zones 2, 3, and 4 require spacing on the order of d/2 in the plastic hinge zone and d elsewhere in the column.

Please note that other lateral reinforcement vertical spacing requirements based on vertical bar diameter and cross tie diameter also pertain, so the spacings given in the present paper are not exact but are good approximations of the requirements.

In contrast with building requirements, the bridge design specifications (6) require the use of confinement equations for selecting vertical spacing of cross ties throughout the height of the pier wall. The confinement equations are always applied irregardless of the axial load level. The vertical spacing which results from the application of the confinement equations is approximately d/4 throughout the height of the pier wall. A d/4 spacing represents too much cross tie reinforcement since axial and shear stress ratios are so low (Eq. 3 and 4).

SPECIMENS FOR TESTING

Full scale specimens 18 in. thick with two cross tie reinforcement configurations will be subjected to a constant 300 psi level of axial load and cyclic transverse displacements. An h_c/t value of 20 will be used. Since in plane shears will not be applied the h_c/w value is not significant, but instead the value of $w/t = 4$ is used so that confinement provided by horizontal ties is represented when transverse bending occurs.

The spacing required by the bridge design specifications (6), i.e., 3-1/2 in. in the plastic hinge zone and 4-1/4 in. elsewhere, will be used in one type of specimen. The other type will have a cross tie configuration which somewhat resembles the UBC requirement (5), i.e., a 12 in. vertical spacing is under consideration.

TEST METHOD

Pier walls experience a point of inflection at the midheight of the wall when longitudinal bridge response occurs because fixity exists at the top and bottom of the wall. Therefore 15 ft. tall cantilever specimens to be used in the test program will exactly represent a pier wall which is 30 ft. in height. At the base of the pier wall specimens will be a 4 ft. thick footing so that pull out conditions for the vertical reinforcement are accurately represented. The footing will be bolted against a reaction block.

Vertical load and transverse deflection will be imposed on the tip of the cantilever by servo controlled actuators. A cyclic displacement history will be imposed while axial load is held constant. The maximum transverse displacements to be imposed at the tip of the cantilever are 6 in. This corresponds to a 12 in. longitudinal bridge displacement.

CONCLUSION

Pier walls subjected to out of plane seismic responses are considered to possess the frailties of a column. At this juncture they must be designed in accord with column specifications (1). The bridge design specifications (6) require 2-1/2 in. to 4-1/2 in. vertical spacing of cross tie reinforcement and essentially the same spacing is required throughout the height of the pier wall. This is too much cross tie reinforcement for the pier wall because axial and out of plane shear stresses are small in these elements. If pier walls received a separate classification in the specifications then logical reinforcement details for the element could be spelled out. A full scale experimental test program is underway at UCLA to show that pier walls with wider cross tie reinforcement can be used. Suggested specifications for pier walls will also be developed.

ACKNOWLEDGEMENT

The pier wall test program is sponsored by FHWA/CALTRANS. Mr. Ray Zelinski, Senior Bridge Engineer, Office of Structures Design, CALTRANS is the contract monitor.

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EXPLOSIVE TREATMENTS FOR REDUCING RESIDUAL STRESSES IN WELDMENTS

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SYNOPSIS

Explosives were applied to butt-welded and fillet-welded specimens of ASTM A36 grade structural steel. The shock loads substantially reduced residual stresses in the weld heat affected zone and, in some cases, induced compressive residual stresses. These results suggest that explosive treatments hold promise for extending the life of welded structures.

INTRODUCTION

Residual stresses that result from welding increase susceptibility to failure by fatigue, stress corrosion and impact, and hence reduce the useful life of a welded structure. Measures that can be taken to reduce residual stresses and, hence, enhance weldment lifetime include mechanical peening, heat treatment, and applying a remelt pass to the weld [1,2]. Lesser practiced is the explosive treatment method, although this method has been established and applied in the Soviet Union [3-8], investigated in the United Kingdom [9] and in the Peoples Republic of China [10], and patented in Japan [11]. In this method, a strip or cord of high explosive is placed in contact with the weldment and detonated to produce a shock wave of predetermined strength in the weldment.

Explosive treatments can substantially increase fatigue life [5,8,9], brittle fracture resistance [7,8], and stress corrosion cracking resistance [8]. However, the reasons for mechanical property enhancements due to shock loading are unclear and have been attributed to shock wave induced changes in the residual stress state around welds [3,4], changes in the dislocation substructure [9] reductions in hydrogen trapping sites [7], and changes in the geometry of the weld toe [9].

In this paper, we discuss the effect of shock loads on residual stresses that are typical for welded steel bridges. Results indicate that explosive treatments are effective in reducing residual stresses, suggesting that the failure properties of welded structures could be enhanced. Work in progress will examine the microstructural effects of shock loads and will quantify the fatigue life enhancement of welds from shock loads.

EXPERIMENTAL PROCEDURE

Weld specimens were fabricated in butt-welded (Figure 1) and fillet-welded (Figure 2) configurations. The plate material was ASTM A36 grade structural steel, which is a plain carbon steel designed for general structural purposes in riveted, bolted, or welded construction of bridges and buildings. The nominal composition is 0.26% max. C, 0.04% max. P, 0.05% max. S, and the balance Fe. The specified tensile requirements are 250-MPa yield strength, 400- to 550-MPa ultimate tensile strength, and 23% elongation in a 50-mm gage length. Base metal for the weldments was cut from rectangular bar stock 12.5 mm thick and 76 mm wide for the butt-welded specimens or 152 mm wide for the fillet-welded specimens.

For the butt-welded specimens, a single V-groove weld preparation (60-degree minimum) was used with a 3-mm maximum root face and a 2-mm maximum gap. Manual shielded metal arc welds were applied using 3-mm E7018 electrodes at 130 amperes in the flat position. After five passes were made in the V-groove, the plate was turned over, a groove was cut on the back side with a carbon arc, and two final passes were made. No preheating was used. Fillet-welded specimens were made with a single weld pass starting at the midlength location. The fillet welds were made with the same weld process as the butt welds.

Shock treatments were performed with two configurations of sheet explosive and one configuration of cord explosive. The sheet explosive was "Detasheet C"* that was either 0.08 cm or 0.11 cm thick. The sheet explosive had a density of 1.48 g/cc and a composition of 63% PETN (pentaerythritol

*Detasheet is a registered trademark name of E. I. DuPont de Nemours & Co., Inc.

tetranitrate) and 8% NC (nitrocellulose), with the balance an elastomeric binder. The cord explosive was "primacord"* that was 3 mm in diameter, with 0.21 g/cm of PETN.

For the shock treatments with sheet explosive on butt-welded specimens, a 6.4-cm-wide strip of explosive was placed over the length of each specimen in the configuration shown in Figure 1. The detonation direction was parallel to the weld. Strips of 3.8-cm-wide sheet explosive were placed on the fillet-welded specimens as shown in Figure 2. The two strips of sheet explosive on each fillet-welded specimen were detonated from a single source: the detonation direction was parallel to the specimen width.

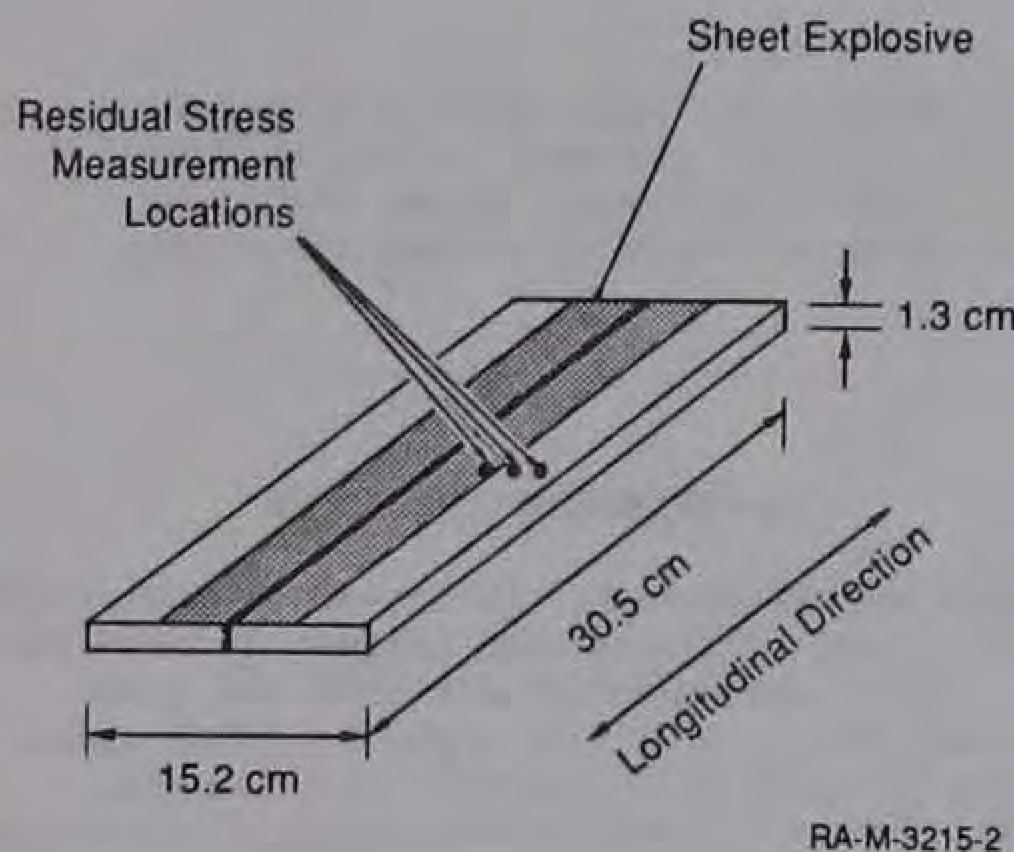

Figure 1. Butt-welded specimen.

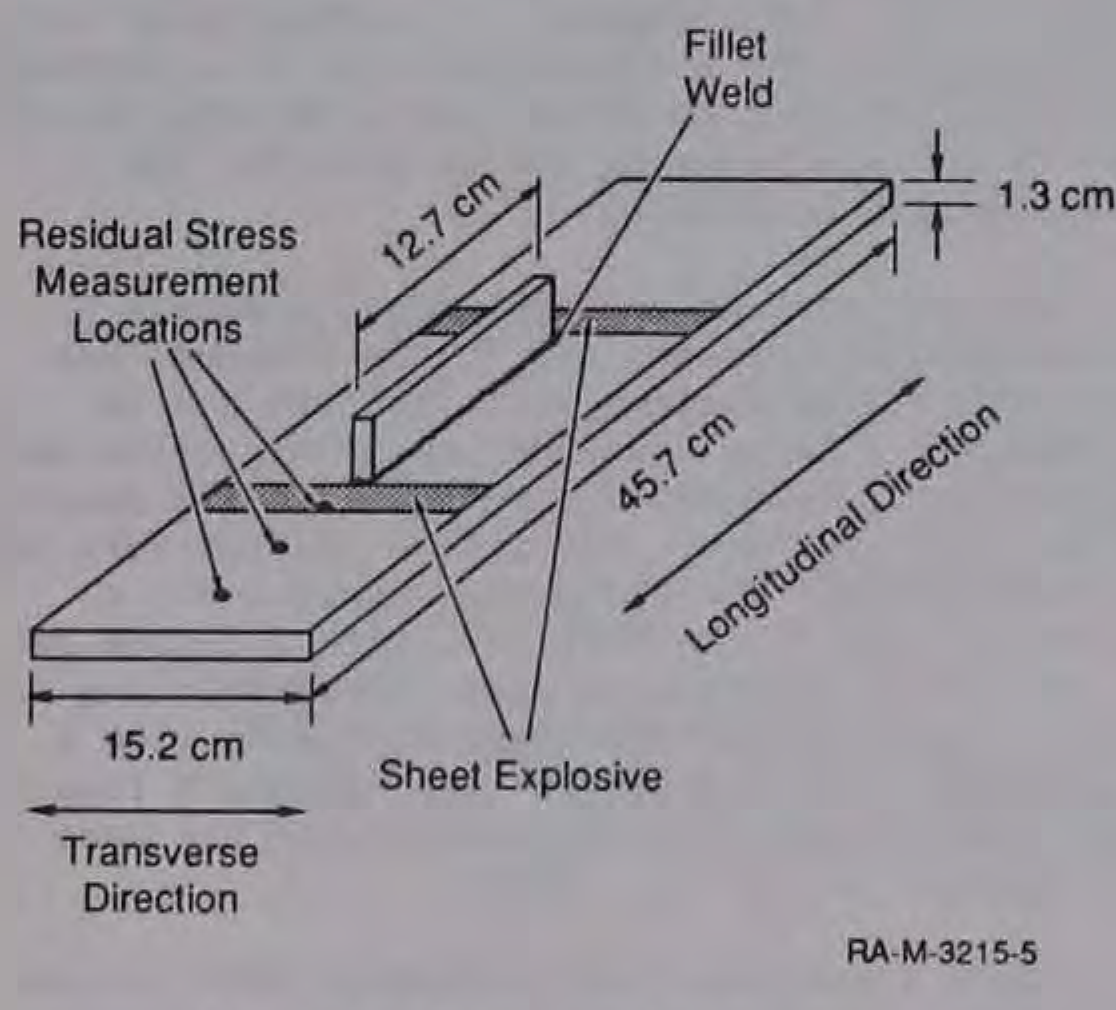

Figure 2. Fillet-welded specimen.

For the shock treatments with cord explosive on butt-welded specimens, one cord was positioned on either side of the weld in contact with the weld metal and the heat affected zone. The two cords of explosive were detonated from a single source. For the fillet-welded specimens, two cords were oriented in an analogous manner to the sheet explosive positions in Figure 2. The cord explosive was positioned positioned in contact with the weld and the base metal near the center line of the specimen.

During shock loading, specimens were supported along their length dimension by two steel support plates that were oriented parallel to the detonation direction. The support plates extended in from the edges of the specimen by approximately 1.3 cm.

Residual stresses were measured by the hole-drilling strain-gage method in accord with ASTM Standard E837-81 [12]. Except for a few of the measurements indicated in Figure 3, all holes were drilled to a depth of about 1 mm on the shock-treated side of the specimen. Because the hole-drilling technique is not accurate at residual stress levels near or greater than the yield stress [13], we have reported all readings greater than the nominal yield stress for A36 steel as equal to the nominal yield stress (250 MPa).

RESULTS

Figure 3 shows longitudinal residual stress measurements for as-welded and shock treated butt-welded specimens. Shock treatment with cord explosive reduced the longitudinal residual stresses to about 65% of the yield stress at locations near the weld (i.e., about 2 cm from the weld centerline). The 0.08-cm- and 0.11-cm-thick sheet explosives produced larger reductions in the longitudinal residual stress near the weld (i.e., to about 24% and 45% of the yield stress, respectively). All of the explosive treatments produced a more uniform longitudinal residual stress profile across the butt-welded specimens. Gradients in the longitudinal residual stress were greatest for the as-welded condition and least for the 0.08-cm-thick explosive. Residual stress measurements were also performed from the surface opposite the shock loaded surface for the 0.08-cm-thick sheet explosive tests. These measurements revealed compressive longitudinal residual stresses at all distances from the weld.

Transverse residual stresses present before and after shock loading butt-welded specimens are shown in Figure 4. The as-welded condition exhibited low compressive residual stresses of about -40 MPa. The 0.08-cm-thick explosive substantially increased the transverse residual stresses near the weld to about 175 MPa. The cord explosive and 0.11-cm-thick sheet explosive also increased the transverse residual stresses but to lower levels (50 MPa and 25 MPa, respectively). Residual stress measurements made on the side opposite the shock loaded surface revealed that shock loading increased the magnitude of the compressive transverse residual stresses near the weld to about -100 MPa.

*Primacord is a registered trademark name of Ensign-Bickford Company.

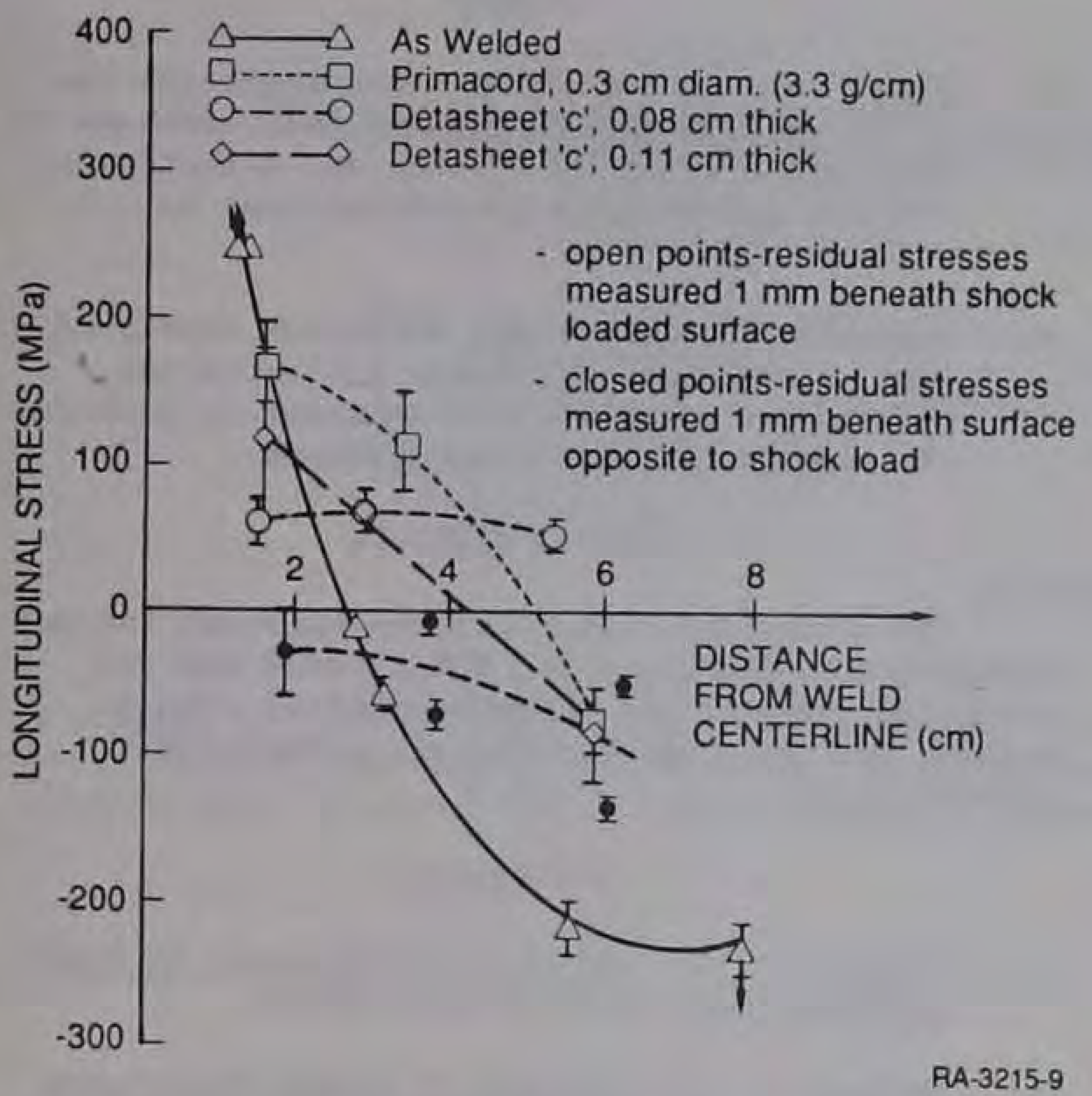

Figure 3. Longitudinal residual stresses in butt-welded specimens before and after various explosive treatments.

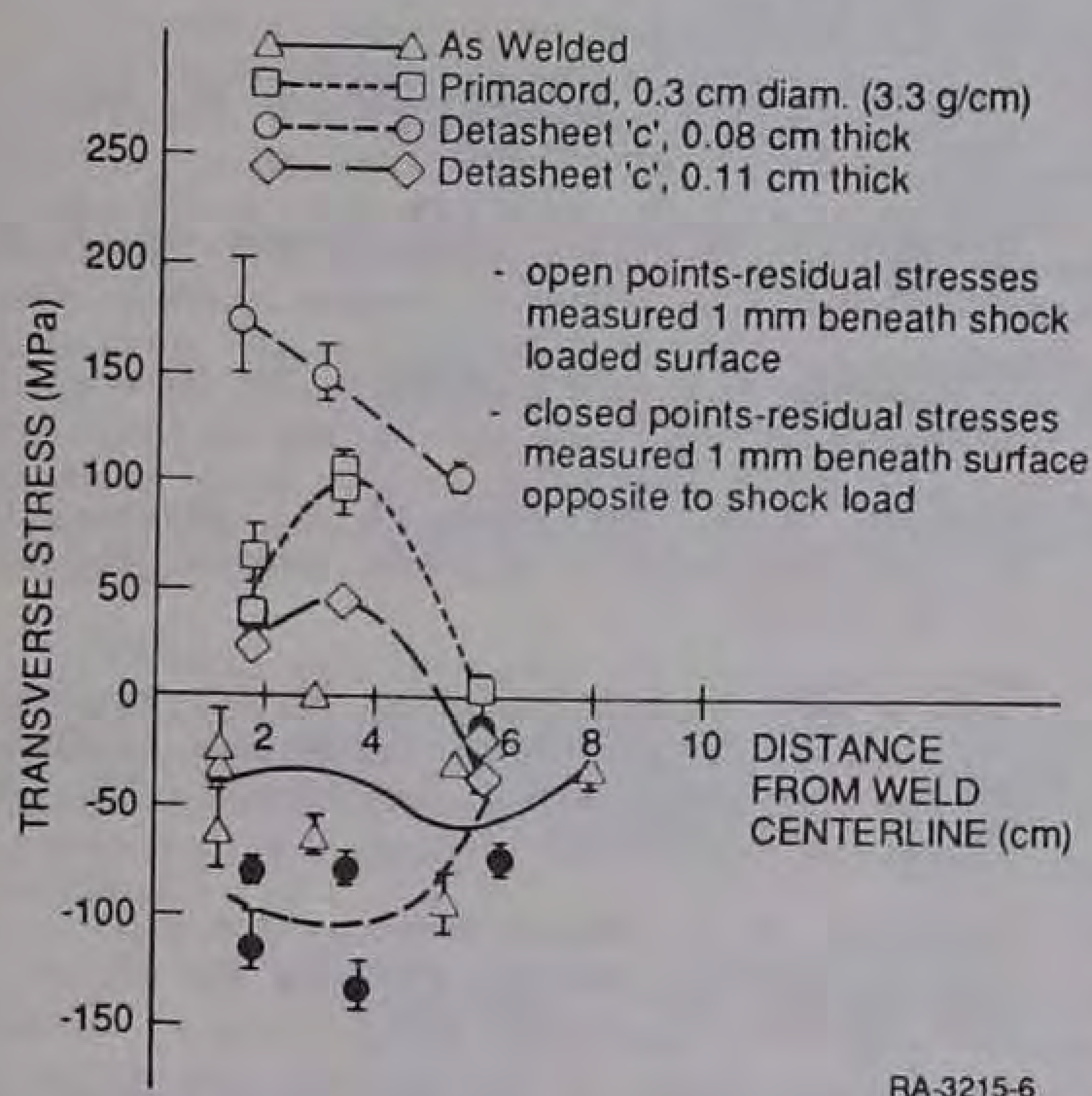

Figure 4. Transverse residual stresses in butt-welded specimens before and after various explosive treatments.

Shock loading the fillet-welded specimens with cord explosive reduced the longitudinal residual stress by about 20% near the weld but was less effective farther from the weld flange (Figure 5). The 0.08-cm- and 0.11-cm-thick sheet explosive reduced the longitudinal residual stresses near the weld by 30 and 80%, respectively. With increasing distance from the welded flange, longitudinal residual stresses present after the cord explosive treatment and 0.08-cm-thick sheet explosive treatment did not differ greatly from those in the as-welded condition. The 0.11-cm-thick sheet explosive produced a substantial compressive residual stress (-60 MPa) at a distance of 5 cm from the welded flange.

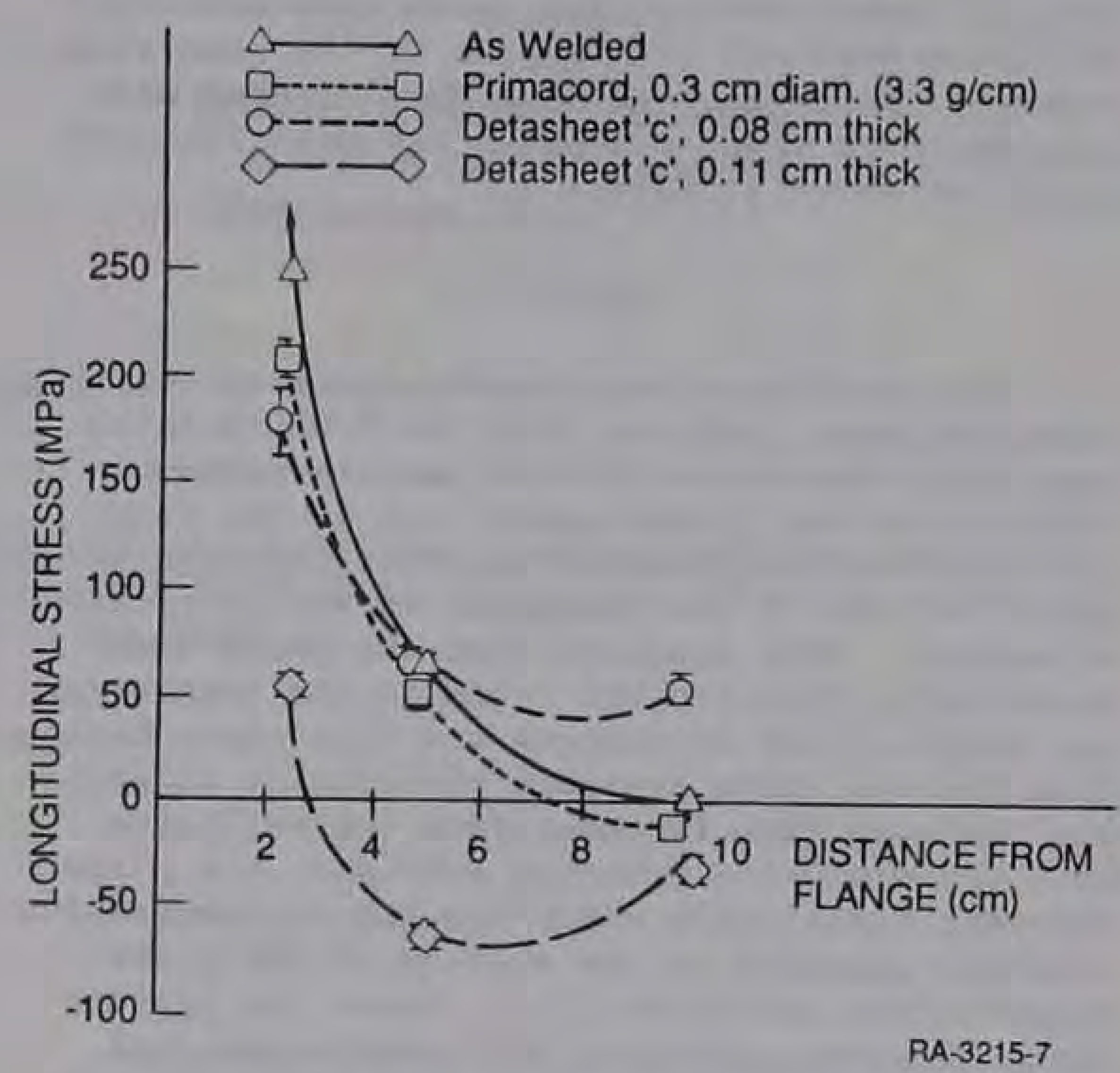

Figure 5. Longitudinal residual stresses in fillet-welded specimens before and after various explosive treatments.

The transverse residual stresses in fillet-welded specimens were compressive in the as-welded fillet welds (Figure 6). Cord explosive reduced the transverse residual stress to near zero next to the weld but did not have a large effect farther from the weld. The sheet explosive treatments produced substantial reductions in the transverse residual stress and, in some cases, produced low level tensile residual stresses where compressive stresses were originally present.

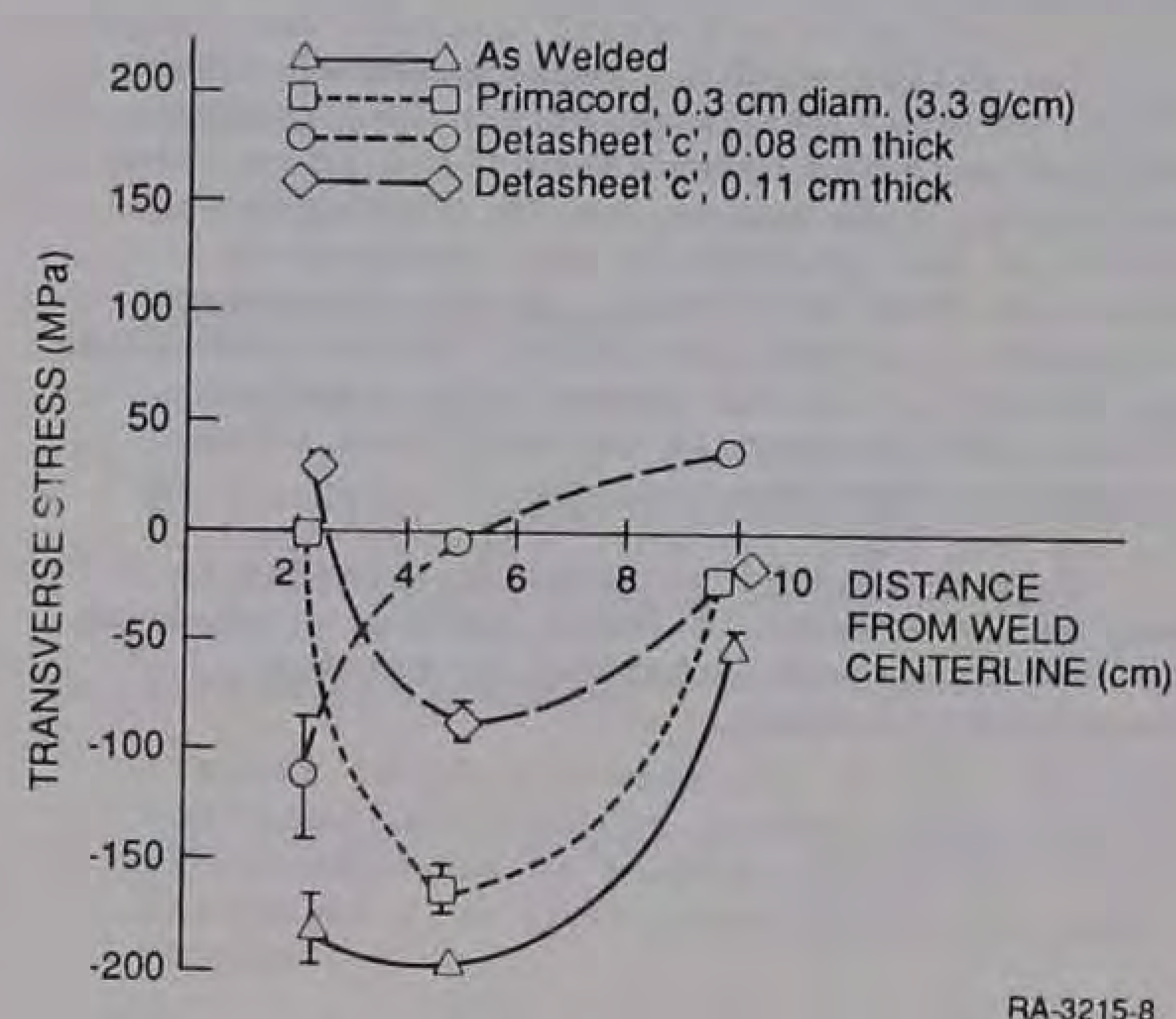

Figure 6. Transverse residual stresses in fillet-welded specimens before and after various explosive treatments.

Distortions in the butt-welded specimens due to shock loading were minor. Profiles of as-welded specimens perpendicular to the weld were slightly V-shaped with angles that varied from 0 to 1.5 degrees. Specimens shock loaded with cord explosive or with 0.08-cm-thick sheet explosive also exhibited profiles with angles in the same range; however, specimens that were shock loaded with 0.11-cm-thick sheet explosive had profiles with angles of about 3 degrees.

DISCUSSION

The residual stress measurements of the butt-weld specimens indicate that the 0.08-cm-thick explosive treatments produce tensile residual stresses on the shock loaded side of the weld specimens and compressive residual stresses on the opposite side of the specimens in the longitudinal direction. This suggests that the shock load plastically distorts the plate in the transverse and longitudinal directions and that plate bending from the explosive pressure plastically stretches the plate surface farthest from the explosive. Once the explosive pressure subsides, the plate recovers elastically which results in compressive residual stresses at the surface of the plate opposite the explosive (i.e., where the plastic stretching was greatest) and tensile residual stresses on the surface where the explosive had been applied.

The residual stress patterns in the butt-welded plates were altered less by the shock loads from the cord explosive than from the 0.08-cm-thick sheet explosive. This is probably due to the explosive pressures produced by the cord explosive. However, the shock loads from the 0.11-cm-thick sheet explosive altered the residual stress patterns in the butt-welded specimens to a lesser extent than the cord explosive. The reason for this is unclear. The 0.11-cm-thick sheet explosive produced large plate distortions (to 3 degrees) and hence would be expected to substantially alter the residual stress pattern.

The fillet-welded specimens showed little effect of shock treatment on the longitudinal residual stresses except from the 0.11-cm-thick explosive. This may be due to the stiffening effect of the specimen in the longitudinal direction from the flange. In the transverse direction, substantial residual stress reductions are achieved from all shock loads apparently because the specimen is not stiffened in the transverse direction.

This investigation is being extended to examine the effects of shock loading on weldment microstructure and mechanical properties (especially fatigue).

SUMMARY

- Shock loads produced by sheet explosive can reduce substantially the residual stresses in structural steel butt welds and fillet welds without producing major distortions to the weldment.
- Explosive thickness and explosive configuration (i.e., cord or sheet) influence the effectiveness of shock treatments in reducing the residual stresses in weldments.

ACKNOWLEDGEMENTS

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THE FATIGUE STRENGTH OF WELDED REPAIRS

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SYNOPSIS

A program of work was carried out in which girders were tested under fatigue loading. Fatigue cracks that occurred were repaired by welding under both laboratory and simulated site conditions. When the repaired girders were re-tested the original fatigue strengths were obtained.

INTRODUCTION

Steel bridges are susceptible to fatigue cracking caused by numerous cycles of heavy truck loadings. At the present time, many bridge engineers believe that bolted splices and member replacement are the only reliable methods of dealing with members with deep cracks. However many members cannot tolerate the loss of section required for bolted splices. Where replacement is possible long delays may be imposed on the contractor while waiting for fabrication of new members with the cost of the replacement and detouring traffic running into millions of dollars.

There was a need to demonstrate the reliability of welded repairs and to provide guidance for effective methods of welded repair of fatigue cracks in steel bridge members so that the economies of repair welding can be effectively realised.

In the NCHRP Project 12-27 Welded Repair of Cracks in Steel Bridge Members, a fatigue testing program was carried out in which girders containing fatigue cracks were repaired under the following conditions:-

1. In the laboratory.
2. Out of doors at a height of 8 feet with restricted access.
3. In the fatigue testing rig under dynamic loading.

When The repaired girders were re-tested the original fatigue strengths of the girders were obtained and the fatigue strengths were usually increased by grinding the toes of fillet welds and the surfaces of cope holes.

EXPERIMENTAL WORK

Details of Test Specimens

The fatigue testing program involved two types of fabricated girders containing several different details which were liable to give rise to fatigue cracking.

The welded girders were 7 feet long x 2 feet deep with 0.5in thick webs and 0.75in thick flanges, the flanges being 8in wide. Full depth vertical stiffeners were welded close to each end of the girders on both sides of the web.

Other details of the girders were as follows:

Type 1 Test Specimen:

a) a cover plate welded to the tension flange

b) 3 partial depth vertical stiffeners 14in long on each side of the web at the centre and 6 inches from the centre.

Type 1a Test Specimen

Similar to type 1 with the three central vertical stiffeners on each side of the girder extended to within 3in of the lower flange to give restricted access for welded repairs.

Type 2 Test Specimen

a) a transverse butt welded in the tension flange

b) a cope hole in the web at the position of the flange butt weld

c) 2 full depth vertical web stiffeners on each side of the web 10 inches from the centre of the girder.

d) a partial depth vertical web stiffener 14in long of each side of the web at the centre of the girder.

MATERIALS

Girders

The girders were fabricated from ASTM A588 Gd B steel.

Consumables

Repairs of fatigue cracks were carried out with covered electrodes complying with AWS A5.1 classification E7018.

METHOD OF FATIGUE TESTING

Fatigue testing was carried out in a 224,000lb capacity MAYES servo hydraulic bend rig with the test specimens loaded in 3 point bending under a load of 203,800lbs. The loading rates was typically 3.2Hz (192 cycles/min) so that the specimens were subjected to approximately 275,000 cycles per day.

Fatigue cracking was initiated from the ends of the partial depth stiffeners or from the edge of the cope hole after a minimum of 75,000 cycles in Types 1 and 2 specimens and after 450,000 cycles in the Type 1a test specimen.

When cracks had propogated to a length of at least 6 inches they were gouged or ground out and were repaired by welding. All groove repair welds were ground flush with the plate surface and some of the fillet welds replaced during repairs were subjected to grinding of the weld toes. The surface of the cope hole was also ground smooth after repair of associated cracks.

Welding repairs of the cracked specimens were carried out under the following conditions:-

1. In the welding laboratory.

2. Out of doors at a height of 8 feet with restricted access.

3. In the fatigue testing rig under dynamic loading.

The repairs under dynamic loading were carried out while the beam was deflected at the centre by 0.10 inches at a frequency of 50 cycles per minute.

RESULTS AND DISCUSSION

The results of the fatigue tests carried out on test specimens in which fatigue cracks were repaired either in the welding laboratory or out of doors under conditions of restricted access showed that there was no deterioration in fatigue strength.

Furthermore, the fatigue strength could usually be increased by grinding the weld toes, which had the effect of delaying the initiation of cracking. When repairs to fatigue cracks are carried out it is strongly recommended that the toes of any associated fillet welds should be ground either by a rotary burr or by disc grinding. The procedure for improving the fatigue strength of welded joints by grinding is described in a paper by Booth [1].

Repairs Carried Out Under Dynamic Loading

Some difficulty was experienced when repairing a crack under conditions of dynamic loading because the gap at the root of the groove was opening and closing so that the gap varied from nil to 0.1 inches. This caused strain on the solidifying weld metal resulting in hot cracking of the weld. This problem was overcome by depositing short lengths of weld metal, 4 inches long in this case, at each end of the crack where the opening of the gap was less than about 0.03in.

After the first side of the groove was filled with weld metal over lengths of 4in at each end of the crack the root of the weld was ground out from the second side and this side was then welded over the 4in lengths.

This effectively prevented the remainder of the crack from opening more than 0.03in so that welding could be completed under dynamic loading. This technique for welding grooves that have gaps that are opening and closing could be applied to longer grooves by welding from each end of the crack in lengths of approximately 4 inches and completing each 4 inch length of weld to the full section thickness before proceeding to weld the next 4 inch lengths. The ends of each 4 inch weld should be ground to provide a sound weld start before the next weld is made.

CONCLUSIONS

1. Two types of fabricated girder were used for fatigue testing and fatigue cracks were repaired by welding, both in the welding laboratory and out of doors on top of an 8ft high tower with severly restricted access for the welder. Testing of the repaired girders showed that the original fatigue strengths of girders could be obtained.

2. The fatigue strengths of repaired girders were usually increased by grinding of the toes of fillet welds.

3. When a welded repair was carried out under dynamic loading in which the gap at the bottom of a weld groove varied from nil to 0.1 inches the root bead cracked unding welding. Cracking of the root bead was prevented by depositing short lengths of weld, up to 4in long, from each end of the groove to lock the groove and prevent movement.

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REDUCED BRIDGE MAINTENANCE WITH CMA

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SYNOPSIS

The development of CMA, Non-Corrosive road salt replacement is traced from theory, practicality and economic adoption probability as infrastructure and public health values are considered.

The goal of infrastructural bridge deck preservation depends upon maintaining the bond of concrete to steel reinforcement steel. Past efforts of physical barriers to deicing chlorides have been denser concrete, various plasticizers in concrete and on the rebar itself, and cathodic protection, repelling the negatively charged chloride ion. The alternative substitution of chloride by organic anion as acetate is the approach in calcium magnesium acetate, CMA, a theoretical non-corrosive salt.

While confirmation of the physical parameters of ecological acceptance (caltans), non-corrosive (Prof. Locke, Univ. of Oklahoma, Kansas); melting kinetics and bonding ice to substrate (Prof. Cussler, Univ. of Minn.), the practicality of Iowa winter and road evaluation commenced by borrowing a six cubic foot portable cement mixer from Iowa State University Civil Engineering Dept. for blending and agglomerating (dolomitic hydrated lime, acetic acid, sand and a bonding agent) ingredients for CMA. The resultant particles had a mass of 75 lbs. per cubic foot, would supply traction upon melting thru ice. Confirmation of CMA properties complement the geological reduction of freeze-thaw cycles detrimental to Iowa roads. Under competitive contract of 150 tons of CMA on sand were produced and comparatively Iowa winter roads evaluated against a sand/salt mixture at 300 lbs. per lane mile disposal rate using the same spreaders.

The adoption of CMA requires an understanding that the real cost of deicing chemicals must include environmental corrosive degradation: bridge decks, autos, water contamination; salt is $1400 per ton not $25 per ton purchase price. CMA composed of 80% by weight acetic acid which is now derived from fosil petroleum can be a by-product of waste carbohydrate fermentation streams and/or corn as a glucose precursor. The combined activity of microbiologists, physical scientists expert in biotechnological extractive concentration are currently working for the reality of economic recyclably derived acetic acid.

Funding was initiated by Federal Highway Administration (selection of CMA, alternative to salt), Iowa Highway Research Board for two production grants, and TRB (Transportation Research Board) for annually supplying the information transfer media through the Winter Maintenance Committee.

STRUCTURAL AND SOIL PROVISIONS FOR APPROACHES TO BRIDGES

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SYNOPSIS

Approaches to bridges are designed to provide a smooth and safe transition from the highway pavement to the bridge deck and back to the highway pavement. Generally this transition area, regardless of pavement type, has been the cause of poor riding quality. Despite the widespread occurrence of bridge approach problems, only a small number of research studies have been performed on the subject. Most of these studies have been limited to problems associated with specific bridges sites. This paper compiles the different problems that have been encountered at approaches to bridges, summarizes corrective measures that have been used or suggested, and identifies major points that will be investigated in phase II of this study.

INTRODUCTION

Approaches to bridges are designed to provide a smooth and safe transition from the highway pavement to the bridge deck and back to the highway pavement (Fig.1). Generally this transition area, regardless of pavement type, has been the cause of poor riding quality. Pavement irregularities adjacent to bridges are unpleasant, unsafe, and destructive to vehicles and to the bridge structures. In addition, they require costly maintenance that usually involves mudjacking or patching the approach pavement. Maintenance operations are costly to the traveling public in both time and money.

Despite the widespread occurrence of bridge approach defects, only a small number of research studies have been performed on the subject. Most of these studies have been limited to problems associated with specific bridges sites. This paper compiles the different problems that have been encountered at approaches to bridges, summarizes corrective measures that have been used or suggested, and identifies major points that will be investigated in phase II of this study.

MAJOR CAUSES OF BRIDGE APPROACH PROBLEMS

A state-of-the-art and state-of-practice study that covered published and unpublished work in the U.S. and overseas was conducted. This included a comprehensive literature review and a survey of state highway agencies on design and construction practices currently used at approaches to bridges. The literature survey indicated that most problems occurring at bridge approaches can be associated with: differential settlement between the highway pavement and bridge deck; and rotation and/or lateral movement of the abutment (Fig.2).

1. Differential Settlement

Bridge abutments are usually founded on relatively stable foundations and practically speaking, cannot settle, whereas the highway pavement is located on an embankment and foundation that are potentially free to settle. Factors affecting differential settlement may be summarized as follows:

(a) Consolidation of the embankment foundation. A major portion of the approach displacement can be attributed to the post-construction consolidation of material within the embankment foundation. Conditions of the foundation material and loadings that affect the rate of consolidation include the degree of preconsolidation, the geometry of soil structure, soil properties (compressibility, permeability, stress history and void ratio), layer thickness, the dimensions of the embankment, surcharge, length of drainage paths, and rate of the construction of the embankment.

(b) Volume change of the approach embankment. Volume changes within the approach embankment may result from the rearrangement of soil particles, loses of moisture (shrinkage), increase in moisture (swelling) or ice and frost action.

(c) Lateral movement of approach embankment. Lateral movement of approach embankment is the result of shear strain within the embankment sublayers. It occurs mostly on approach embankments constructed on foundations that slope toward the ends of bridges and can be aggravated if a high water table elevation, weak foundation material, poor compaction and low shear strength between the embankment layers exist. Lateral movement of an approach embankment not only produces settlement of the approach pavement but in many cases it also causes damage to the wingwalls and abutments.

(d) Subsurface erosion. Erosion of soil particles could reduce the ability of the embankment to carry the weight of the approach pavement and, consequently cause the approach to settle.

2. Lateral/Rotational Movement

Some of the problems occurring at bridge approaches can be associated with longitudinal, rotational or vertical bridge abutment movements. Abutments have been observed to move both forward and backward, and to rotate forward and backward as well as to move vertically. The motion depends on the abutment type, abutment support system, foundation soil type, and construction sequence. Major factors which may lead to the rotation and/or movement of the abutment are:

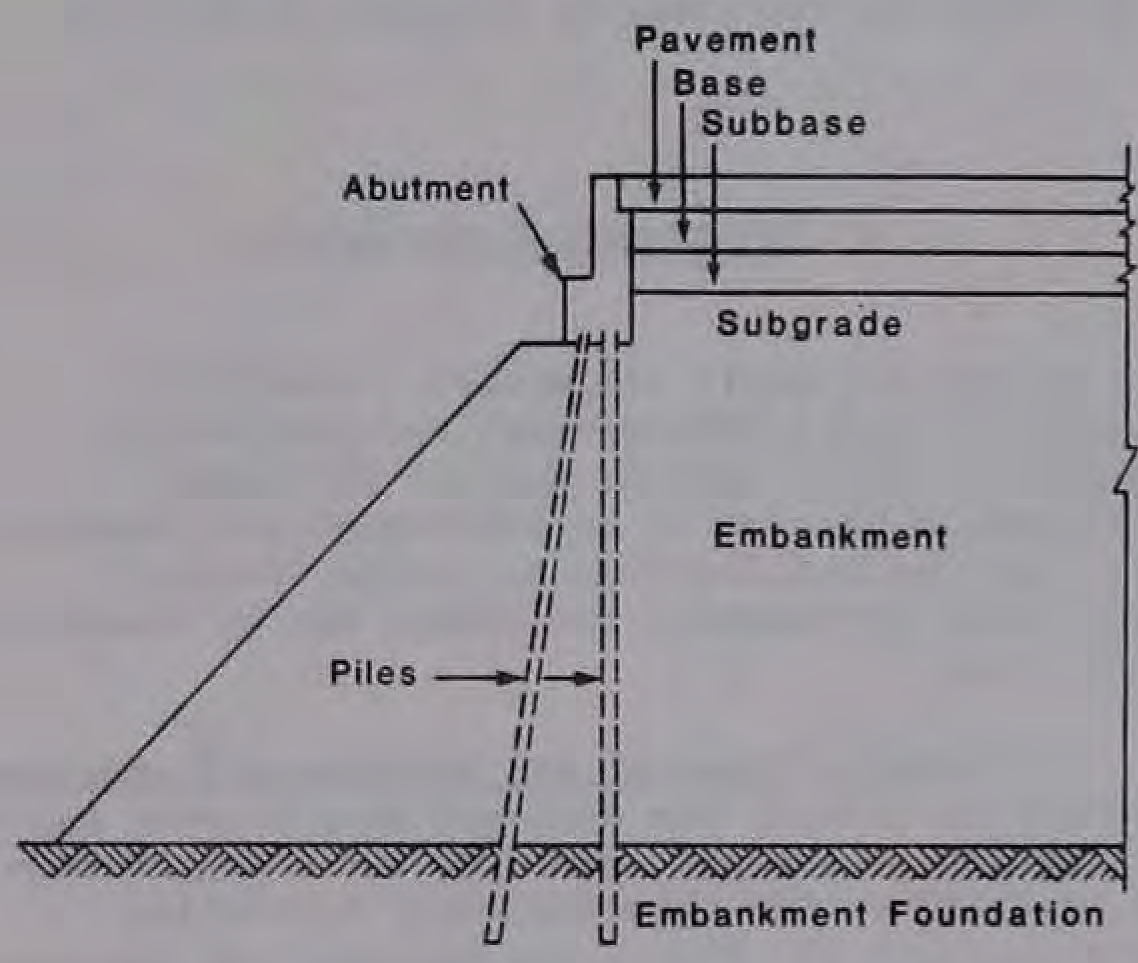

Fig. 1

(a) Slope failure. Embankment instability may cause lateral and rotational movement of the abutment. This movement may cause bending of the piles, tipping and cracking of the walls, and damage to the abutments and expansion joints.

(b) Seepage. Seepage may cause movement of the abutment due to a reduction in soil resistance and an increase in lateral pressure of the saturated soil in front of and behind the abutment, respectively. Cohesionless soil, particularly fine sands and silts are most susceptible to seepage failure.

(c) Thermal forces. Thermal expansion of the bridge superstructure and pavement may cause lateral displacement or tipping of the structure units. Jointless structures or where the expansion joints fail to operate are affected more by thermal forces.

(d) Foundation settlement. Uneven settlement under the abutment foundation caused by variation in the depth of compressible foundation layers may cause rotation of the abutment, and consequently cause cracks in the abutment walls and opening at the joints that connect the wingwalls to the abutment.

CRITICAL ITEMS IN DESIGN AND CONSTRUCTION

To prevent or minimize bridge approach problems, careful consideration must be given to design and construction techniques. Critical items in the design and construction of bridge approaches are embankment foundation, embankment and backfill materials, drainage systems, abutment type, construction methods, and structural details.

Several construction measures have to be considered in order to minimize post construction settlement when the embankment foundation contains weak and compressible soil such as saturated soft clays, compressible silts, organic clays, and peats. Use of surcharge is the most common method to reduce the magnitude of settlement after construction. If the use of surcharge is not economical or the time required for surcharging is greater than the time available, vertical drains may be used to increase the rate of settlement. Other measures that may be taken are waiting periods, removal of unsatisfactory material and the

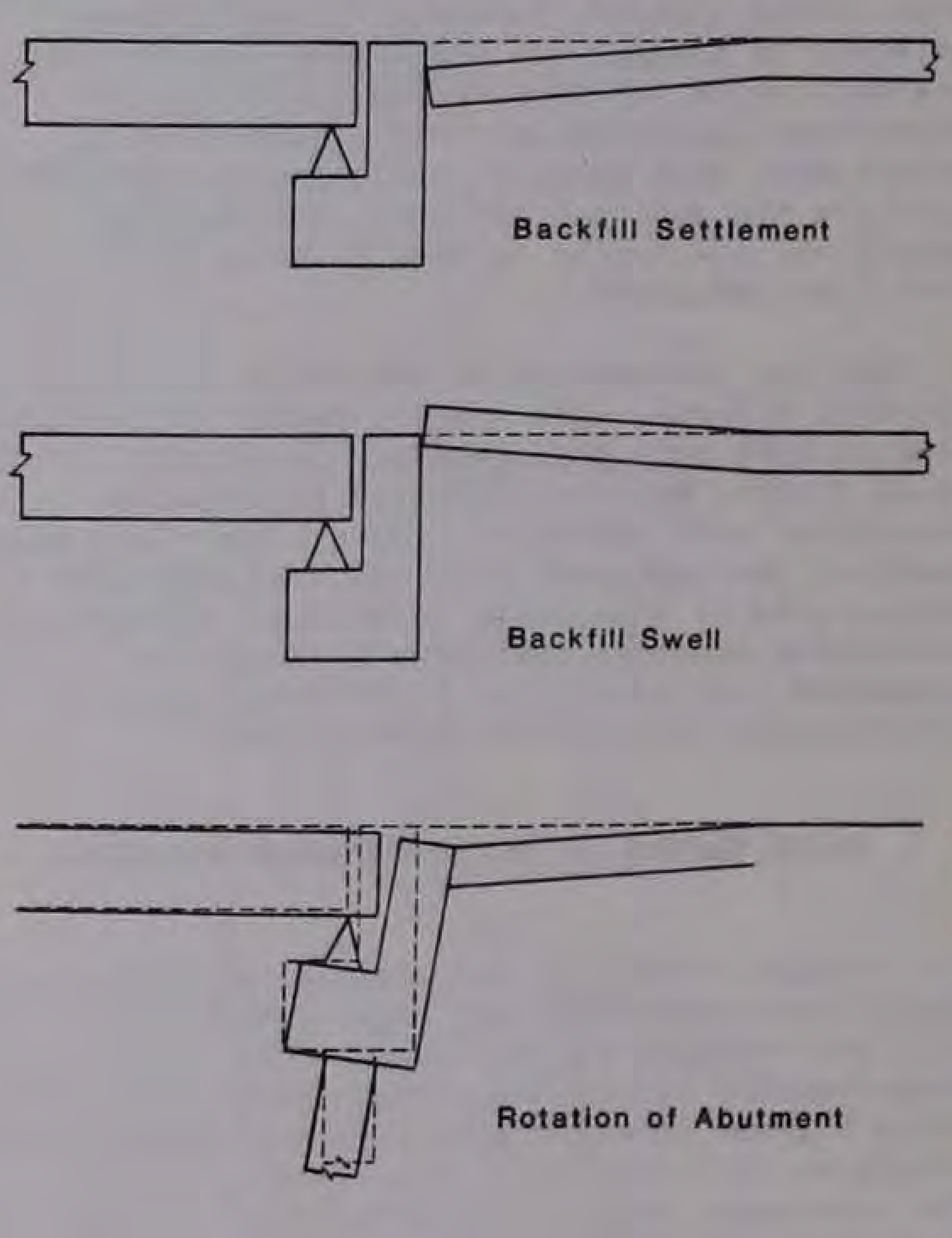

Fig. 2

use of lightweight embankment material such as furnace slug or lightweight concrete.

In embankment design the following parameters need to be considered: the suitability of various soil types as bridge approach embankment material and the compaction specification. Approach embankment, when possible should be constructed using selected material such as slightly cohesive granular soil with adequate strength.

Special attention must be given to control and to drive water away from critical areas around the abutment and beneath the approach pavement by providing adequate drainage systems. The removal of water from these areas is essential to prevent surface and subsurface erosion, and to reduce the lateral pressure on retaining wall due to an increase in the weight of the soils. Drainage pipes should be properly installed to avoid leaks, and adequately insulated to prevent clogging.

The type of abutment and its supporting system is a factor in bridges approach surface irregularities. The more confined the approach fill due to abutment type or wingwalls, the greater the approach settlement. This is due to the difficulty of achieving proper compaction in confined areas. Also significant differential settlements are more common for pile supported abutments.

The construction methods used for approach embankments could considerably affect the performance of bridge approaches. Benching of sloping ground to avoid lateral movement of the embankment, directing the surface water away from the abutments and retaining walls to avoid soil erosion during the construction, and controlling the lift thickness, moisture content, and densification level of embankment and backfill material are among the good practices that have been identified in approach embankment construction.

The careful design and construction of structural components in bridge approaches such as bridge deck, abutment, wingwalls, approach slabs etc., is essential to the construction of satisfactory approaches. Poor design details associated with approach pavement joints at the abutment, approach slab-pavement transition, abutment and wingwall intersections, expansion joints, etc., have resulted in rough riding and heavy maintenance requirements.

In addition to the methods described above, special abutments or transition structures may be used where excessive settlements are anticipated or when high traffic volumes are expected. One such structure is the reinforced concrete approach slab which is the most widely used means for controlling surface irregularities. Cellular abutments, cantilever end spans, and geotextiles may also be used to reduce bridge approach settlements.

APPROACH SLABS

Attempts to lessen rough riding condition near bridges have been made by constructing a reinforced concrete approach slab between the bridge deck and the highway pavement (Fig.3). One end of such slab rests on the abutment and the other end rests either on the fill or on a concrete slab called sleeper. The intent of such underlying support is to provide smooth transition between highway pavement and approach slab for traffic. The approach slab assists in bridging the soft zone directly in front of the abutment and in ramping up to the bridge. Most state highway agencies use some form of approach slab in design and construction of highway structures. These slabs seem to be successful in removing the bump from the end of the bridge, however, settlement or swelling may occur at locations away from the bridge causing bumps to develop.

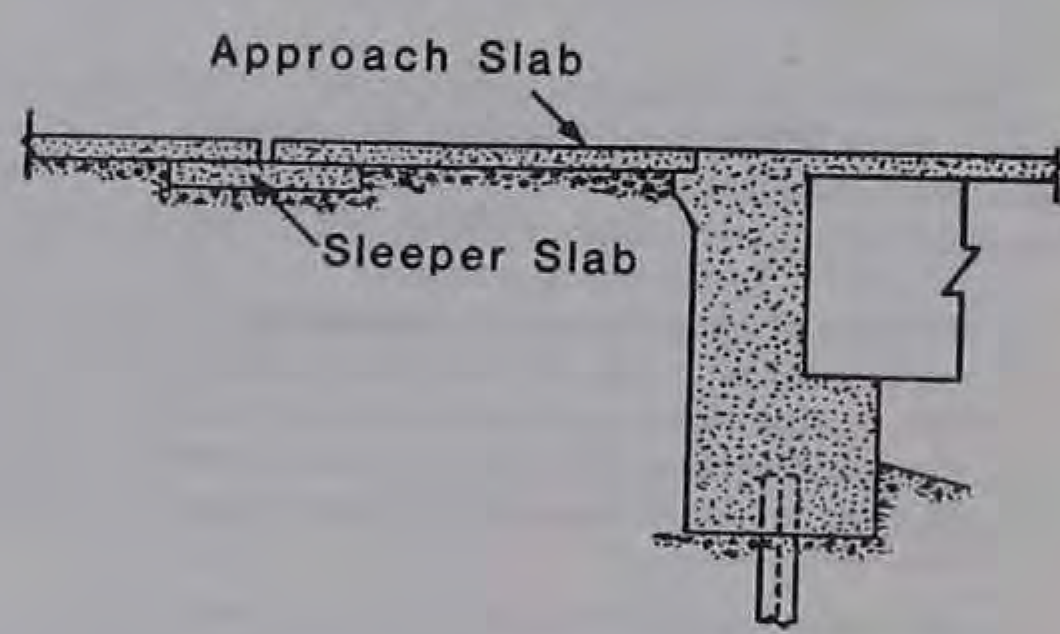

Fig. 3

Approach slabs range in length from 10 feet to more than 100 feet and from 9 inches to 18 inches in thickness. The length of approach slab needed to minimize maintenance has been a controversial issue for many years. A short slab with a small initial cost may over a long period of time require more expensive maintenance than a longer slab with larger initial cost. Approach slabs may also be designed in different shapes: flat, tapered, haunched, and T-beam. The use of flat approach slabs is more common.

Parameters affecting performance of approach slabs such as length, thickness, reinforcement, sleeper configuration, type of connection between the approach slab and the abutment, and the type of embankment material and its height will be investigated in phase II of this study using the finite element technique.

SURVEY OF STATE HIGHWAY AGENCIES

A survey questionnaire based on the findings of the literature review was prepared and sent to state highway agencies to obtain information on current design and construction practices concerning bridge approaches. Forty-six states responded to the questionnaire. One of the questions on the the survey questionnaire and a summary of the responses are given in the following tables.

Question: (a) Have you ever used any of the following special designs to reduce bridge approach defects? (b) If so, are they successful?

Table 1. Reinforced approach slabs

(a) Number of States		(b) Number of States		
No	Yes	No	Moderate	Yes
3	43	3	9	15

Table 2. Cellular abutments

(a) Number of States		(b) Number of States	
No	Yes	No	Yes
22	10	--	6

Table 3. Cantilever end spans

(a) Number of States		(b) Number of States	
No	Yes	No	Yes
24	10	7	--

SUMMARY

A state-of-the-art and state-of-practice study concluded that: Differential settlement between highway pavement and bridge deck, rotation and/or lateral movement of the abutment, and poor design of structural components are the major problems at approaches to bridges.

The biggest contributors to differential settlement between highway pavements and the bridge deck are subsidence of the original ground below the fill, and settlement within the fill mass. Differential settlement can be minimized by the following techniques: use of surcharge, use of drains, removal of unsatisfactory materials, use of lightweight embankment material, and extension of time between construction of the embankment and approach pavement, proper compaction specifications, and the use of moisture barriers to minimize soil expansion.

The rotation and/or lateral movement of the abutment can be minimized by benching of sloping ground, constructing proper drainage system to avoid soil erosion or saturation of the embankment, and by compaction of the soil around the abutment and walls which should be done at the same time in order to prevent tilting of these structures one way or the other.

Careful design and construction of structural components in bridge approaches are essential to minimize bridge approach problems. Poor design details have resulted in rough riding and heavy maintenance requirements.

The most promising solution to bridge approach problems is the use of reinforced concrete approach slabs. However, there are large discrepancies in their use by state highway agencies. A finite element parametric study is underway to identify the most appropriate approach slab design.

USE OF WELDED STEEL MESHES IN BRIDGE DECK

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SYNOPSIS

This paper discusses the investigation concerning the use of welded steel mesh as concrete bridge deck reinforcement. An experimental test program was conducted. The test program included tensile tests, chemical composition tests, fatigue tests, flexure tests, splice tests, pull-out tests, and punch-out tests. The results of the investigation show that welded steel mesh performs well with respect to all of the tests, except the flexure test where it has less ductility than the slabs reinforced with conventional reinforcement. However, the welded steel mesh reinforced slab deformed sufficiently to be used as bridge deck slab.

INTRODUCTION

Welded steel mesh (WSM) can be described as prefabricated sheets consisting of a series of parallel longitudinal wires welded at regular intervals to transverse wires, formed by an automatically controlled electrical welding process (1,2). The material used to fabricate the welded steel mesh is made from hot-rolled rods which were cold-drawn through dies. The cold-drawn process results in a reduced diameter of the wire and increases the yield and ultimate strengths.

The major advantage of WSM is the reduction in construction cost and duration. The reduction in construction time is due to the availability of welded steel mesh in large sheets that can cover larger areas in significantly less time than conventional reinforcement. Also, because the welded steel mesh is prefabricated, there is a small probability of error in placement in comparison with the conventional reinforcement. With regard to labor, the crew members required for the placement of welded steel mesh mats do not need to be highly skilled and are readily available. Also, less supervision is required for the placement of welded steel mesh in comparison to conventional reinforcement.

WSM has been used as slab reinforcement in buildings for decades. In spite of the economical advantages of using welded steel mesh, engineers are reluctant to use it for highway structures such as bridge decks. This is due to the wide spread belief that welded intersections will cause stress concentration resulting in poor fatigue characteristics (3). This is amplified by the lack of knowledge of the properties and behavior of this material. The establishment of the properties and behavior of welded steel mesh is considered essential in order to broaden the application of this material in structural elements.

Some aspects of the behavior of slabs, reinforced with welded steel mesh, in flexure that might be affected by the properties of the welded steel mesh are the bond properties, which depend on the bond developed along the longitudinal wires and on the anchorage provided by the rigidly welded transverse wires. The bonding properties may affect the strength of slabs in shear and the behavior at splices. Other aspects of the WSM that may affect the flexural strength and the mode of failure are the high tensile strength and reduced ductility of the wire. In addition, the use of WSM also changes the rotation and deflections characteristics, and the shear strength of the slabs (4). Because the amount of ductility in a slab is desired to give visual warning of imminent failure, the amount of deformation before the slab collapses is studied analytically and verified experimentally.

Another very important concern with respect to bridge decks is its punch-out shear resistance. The punch-out mode of failure refers to the ability of the slab to withstand a large load distribution over a small area. Although bridge deck rarely fail, the punch-out mode of failure is the most frequently encountered; consequently, the ability of WSM reinforced slabs to resist punch-out failure is studied.

RESEARCH PROGRAM

In an effort to understand the characteristics of welded steel meshes, an experimental test program was conducted. The test and analytical program included the assessment of tensile strength, chemical composition , fatigue strength, flexure strength, splice strength, pull-out strength, punch-out strength and construction feasibility of concrete slabs reinforced with WSM.

Tensile Strength

To establish information concerning the tensile properties of welded steel mesh specimens, two different testing methods were used. The first method is based on a load-controlled testing that was conducted using a 60 kip capacity testing machine. The second method is based on displacement-controlled testing using a 5 kip capacity testing machine.

A total of 101 tensile specimens were tested for this investigation. These specimens included local steels composed of rods, smooth wires, deformed wires, and epoxy coated deformed wires. In addition, German steel was tested that included tempered and non-tempered welded steel mesh and non-tempered bars without weld.

The investigation included a study of the effect of welded steel mesh deformation on strength, the effect of the cold-drawn process on ductility and strength, the effect of tempering on ductility and strength, and the effect of epoxy-coating on ductility and strength. A comparison of the performance of the local steel with the German steel concluded the tensile test investigation.

Chemical Composition &Micrographic Examination

The main objective of the chemical composition examination is to determine the percent carbon content in the steel used in welded steel mesh.The amount of carbon is an important factor and has a large effect on the properties of the steel. An increase in the carbon content results in an increase in the hardness, consequently reduces ductility and weldability.

The crystalline structure and grain dimensions of steel specimens were examined through an electric microscope. Material micrographic examinations were conducted to obtain mechanical properties such as strength, toughness, ductility, etc., and the effect of thermal treatment (tempering, welding, epoxy coating, etc.). The difference in "the amount of deformation" (percent cold work) between the examined specimens can be found from the deformed microstructure. As the percent cold work increases, the strength also increases.

Fatigue Strength

To understand the fatigue life and the fatigue characteristics of the welded steel mesh a fatigue testing program was conducted. In this test program, the stress ranges were taken as 50, 40, 30, and 20 ksi. The effect of tempering was studied by comparing tempered to nontempered welded steel mesh specimens. The effect of welding was studied by comparing specimens with welded intersections with bars without weld. Finally the local epoxy coated welded steel mesh was compared to the imported (German) tempered and nontempered welded steel mesh specimens. All of the above were compared to results of conventional reinforcement found in the literature.

The fatigue test program included a total of 42 specimens. These specimens included local epoxy coated welded steel mesh, tempered German welded steel mesh, nontempered German welded steel mesh, and nontempered German bars without weld.

No matter how carefully controlled a fatigue test is, a large variability in the results is expected. For this reason, a statistical approach to fatigue data analysis can produce the largest amount and most accurate information possible. The Weibull distribution is the most accepted and widely used mathematical model for the analysis of fatigue data. The Weibull distribution, which is also referred to as the third asymptotic distribution of smallest extreme values, was used in the statistical analysis of the fatigue data.

The effect of penetration, tempering, welding on fatigue strength and the position of the fatigue crack initiation were some of the studied variables. The performance of local welded steel mesh in comparison with conventional reinforcement and AASHTO category B was investigated. The welded steel mesh fatigue investigation was concluded with a comparison of the local steel with the German steel.

Flexure Strength

A one-way slab model was tested and analyzed to study the flexural capacity and ductility of bridge decks reinforced with welded steel mesh. Five slabs reinforced with welded steel mesh were tested. The reinforcement ratio, thickness, and mesh orientation were varied. An equivalent bridge deck reinforced with conventional reinforcement was tested and analyzed for comparison. The comparison gives an indication concerning the strength and ductility of bridge decks reinforced with welded steel mesh when compared to bridge decks with conventional reinforcement.

Splice Strength

To establish an understanding of the behavior of spliced flexural reinforcement, a splice test was conducted with the same amount of reinforcement used in the previously mentioned flexure test . The splices were designed according to AASHTO Specifications Article 8.33.5 (5). This test gives an indication of the bridge deck strength and ductility where the welded steel mesh reinforcements are spliced. It also serves as a check on the validity of the splice requirements of the codes. The splice is considered to be satisfactory if the structural element develops its ultimate flexure strength before bond failure or splice failure.

Two splice test slabs were tested. The first slab has the bottom two spliced welded steel meshes in contact with the transverse wires touching. The overlap length is 24 inches in this first slab. The second slab has the transverse wires separated by concrete with an overlap length of 22 inches. The behavior of the the specimen should be different due to the way they transfer the load thru the spliced zone. The first slab will transfer the steel stresses thru contact while the second slab transfers the steel stress mainly thru the concrete provided that the spliced steel remains on contact and does not separate. At high stresses the transfer of steel stress depends on the development length and thus the overlap length.

Pull-out Strength

With regard to the bond strength and the development length of welded steel mesh, a pull-out test was conducted to determine:

a) The affect of the local welding on the bond strength of a single bar embedded in concrete.

b) The affect of the cross bar on the bond strength of the bars.

c) The affect of the adjacent longitudinal bars, the transverse bar, and welding on the bond strength of the bar.

d) How the above cases compare to the pullout strength of a longitudinal bar without the transverse bar.

e) How bundled longitudinal bars with or without transverse bars compare to the above cases.

A total of 22 pull-out specimens were tested that included local and German steels. The pull-out specimens had four different welded steel mesh reinforcement arrangements to satisfy the above objectives. These arrangements consists of a longitudinal wire only, a longitudinal plus one transverse wire, a longitudinal plus two transverse wires, and, finally, a longitudinal plus two transverse wires and two adjacent longitudinal wires.

Punch-out Strength

The behavior of slabs subjected to a large load distributed over a small area,i.e., concentrated load, is studied in the punch-out test. This test is considered to be very important because the most frequent type of bridge deck failures are in the punch-out mode. The punch-out test is designed to compare the capacity of slabs reinforced with welded steel mesh to slabs reinforced with conventional reinforcement, and to determine the advantages, if any, of using welded steel mesh as bridge deck reinforcement in their capacity in punch-out shear.

Two slab models, one with local epoxy coated welded steel mesh and the other with conventional reinforcement, were used to study the punch-out resistance of bridge decks. The specimens dimensions are 6'-8"x16'. The continuity of the slabs was provided by partially constraining the long sides of the model.

Construction Feasibility

Construction simulation are used to study the feasibility of slab construction by using WSM. The simulation program breaks the construction into well understood tasks, and study the construction process by adding the element of uncertainty. The duration and cost of construction using different methods of construction are being studied. These methods include the use of manual labor, the use of different type of cranes, the use of concrete finisher to carry the WSM, and the use of pre assembled WSM cages. Preliminary analyses show that construction duration of the slab can be cut down from 12 days to 3 compared to the conventional method, and the cost cut by as much as 30%.

CONCLUSIONS

The experimental results of the tests described above were accompanied by a thorough analytical analysis from which the following can be concluded:

1. The deformation of wires causes a slight decrease in both the yield and ultimate strength.
2. The smaller the rod from which a wire can be produced the higher the yield and ultimate strengths and the lower the percent reduction in strength due to wire deformation.
3. Cold-drawing increases strength while decreasing ductility.
4. Tempering and epoxy coating result in a slight reduction of strength accompanying the significant increase in ductility.
5. The fatigue characteristics of welded steel mesh are affected by its thermal treatment (tempering, epoxy coating, welding, etc.)
6. The advantage of increased ductility due to tempering is counteracted by a decrease in strength and fatigue life.
7. The fatigue life decreases for a specimen with a welded intersection as opposed to a specimen without a weld.
8. The fatigue life for the epoxy coated local welded steel mesh exceeds those for high-yield deformed reinforcement bars tested by the Portland Cement Association.
9. All specimens tested have a fatigue life in excess of Category B of the AASHTO Specifications.
10. Although the epoxy coated local steel mesh has a higher penetration than the German steel, it has a higher fatigue life.
11. Because the applied pressure in the electrical welding process is considered a critical factor affecting the fatigue life, a carefully selected and regularly monitored applied pressure should be employed.
12. The same ultimate strength for a slab reinforced with WSM and a slab reinforced with conventional reinforcement can be achieved with a smaller reinforcement ratio of WSM. However a reduction in ductility accompaies the savings in steel.
13. The orientation of the WSM in the slabs affects the amount of ductility achievable. A significant gain in ductility was achieved by placing the transverse wires on the bottom surface. This increase in ductility was accompanied by more evenly distributed cracks that had the same propagation length at ultimate capacity.
14. There is enough visual warning (width of cracks, number of cracks, extension of cracks, and overall deflection) for the WSM to be classified as sufficient for use as bridge deck reinforcement.
15. The ultimate flexural capacity of spliced slabs is slightly less than slabs without a splice. This can be attributed to the rigidity of the mesh. The cracks form at the end of the spliced zone, and at the end of the bottom mesh.
16. The WSM has a significantly higher bond due to the rigidly connected transverse wires.
17. The ultimate strength of slabs in punching is affected mainly by the concrete strength because its failure is in punching shear with a cone like failure surface. The reinforcement has a minimal effect at ultimate Strength.

ACKNOWLEDGEMENT

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WIND PROBLEMS OF CABLE-STAYED BRIDGES DURING CONSTRUCTION

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SYNOPSIS

This paper examines the aeroelastic problems of cable-stayed bridges in the design stage. These include vortex-shedding, flutter, and buffeting. Analysis methods are presented for the latter two. Basic to these methods is a full set of bridge vibration modes and the 2-D static coefficients and flutter derivatives, which are first experimentally obtained in the wind tunnel and later incorporated into the full 3-D dynamics of the bridge. Among the most vulnerable configurations of such bridges is the erection stage, as has also been emphasized by other authors. Finally, analytical stability and buffeting predictions based on the flutter derivatives are corroborated via 3-D wind tunnel model tests.

INTRODUCTION

The wind problems of cable-stayed bridges in all stages of construction include primary assessment of the steady design wind forces, plus the aeroelastic stability problems of vortex excitation, flutter and buffeting. These problems can become particularly severe in certain erection stages of the structure. In these stages the deck is typically cantilevered on both sides of a single tower. One of the most severe conditions that may arise is the rocking of the extended deck about the tower as effective pivot during high winds.

Assessment of the wind forces acting in this design stage includes two- and three-dimensional wind tunnel model studies. The 2D studies provide a knowledge of lift, drag and moment coefficients, flutter derivatives, and both aerodynamic admittance and signature buffeting forces. Among the 2D results the flutter derivatives, in particular, provide a firm basis for assessment of aerodynamic damping, which plays an important role in both flutter and buffeting responses.

The 3D studies offer means for the calibration of predictive theory against dynamic model results.

The paper presents analytical flutter and buffeting criteria formulated around the flutter derivatives of the deck. Calculated buffeting results are presented for a representative twin-deck configuration in a typical construction stage. Comments about some interesting aspects of the wind stability of twin-deck configurations are presented.

TWO AND THREE-DIMENSIONAL WIND TUNNEL MODELING

Due to the limitations of state-of-the-art computational fluid dynamics, it is not yet possible to compute precisely the unsteady forces on a bluff body in a turbulent air flow. In particular, prediction of the dynamic response of a bridge to an incoming fluctuating wind depends at this juncture on the analog, rather than digital computer: the wind tunnel.

There are two major approaches to wind-tunnel modeling. The first is the construction of a full, three-dimensional model of the bridge structure, capturing as many of the important features of the local wind topology and dynamic characteristics of the bridge as possible within the scaling constraints. The measured response of the model to various incoming flow velocities can be directly measured, and scaled to the prototype dimensions. The effects of flow features, such as various levels of turbulence, can be investigated directly.

The second method incorporates a combination of an analytical approach and two-dimensional "section models" of the bridge. Coefficients required for the analytical model are determined from data measured from model section tests in a wind tunnel. These coefficients are then applied to the full-scale structure via the analytical model.

In this paper, the relative merits of the two methods will be discussed in the context of a specific example: the Houston Ship Channel (Baytown) Bridge. In particular, the importance of the combined analytical-experimental approach in modeling the actual physical mechanism of the bridge deck response will be discussed.

THE HOUSTON SHIP CHANNEL (BAYTOWN) BRIDGE

The Baytown Bridge is a twin-deck cable-stayed bridge with a main span of 953 feet. The main towers are 431 feet in height, and the deck is 185 feet above sea level. The deck overhangs adjacent the main span are supported by elevated piers. The configuration is shown in Figure 1. Initial wind-tunnel modeling and analysis showed that the completed structure was stable for all winds through a design wind speed of 120 mph. The critical flutter speed for the full bridge was estimated as over 135 mph, and the peak buffeting response as 2.7 feet at center span.

Before construction commenced, it was considered important to determine the stability of the partially-constructed bridge. The bridge owner, the Texas State Department of Highways, wished to ensure complete structural stability for full hurricane winds at the site during all stages of construction.

DYNAMIC ANALYSIS

A series of finite element analyses of the bridge at three critical stages of construction were performed by Greiner Engineering Services, Tampa, Florida using the GT-STRUDL program. A number of the important mode shapes and associated frequencies are shown in Figure 2.

A point of considerable interest, and an important consideration in the study reported herein, is the degree of coupling between the various degrees of freedom of the structure, particularly in the higher modes. For example, it may be noted that for Stage B Mode 8, there is both significant torsional *and* vertical displacement for the deck nodes. This coupling will be discussed presently, as it has considerable bearing on the aeroelastic stability of the structure.

WIND TUNNEL SECTION MODELING

In addition to the above analysis, a series of section model tests were performed by West Wind Laboratory of Carmel, California. The section models in this case were different from those used in for the full-bridge section tests as there were no "Jersey" barriers or fences, a condition consistent with the construction configuration. Typical flutter derivative data for the construction-stage sections are presented in Figure 3. Also shown are those for the full-deck models.

EQUATIONS OF MOTION

For a bridge of deck width B, subjected to a mean flow velocity, U, the modal equations of motion may be written as [1]

$$I_i[\xi_i'' + 2\zeta_i K_i \xi_i' + K_i^2 \xi_i] = \frac{B^2}{U^2} Q_i(s) \tag{1}$$

with $K_i = B\omega_i/U$ and $\xi_i' = d\xi_i/ds$, and $s = Ut/B$. The generalized inertia of the ith mode is given by I_i and the mechanical damping by ζ_i. The generalized force in the ith mode is given by

$$\begin{aligned} \frac{B^2}{U^2}Q_i(s) \quad = \quad & \frac{1}{2}\rho B^4 \int_0^\ell \{\ K[H_1^*(K)h_i^2(x) + P_1^*(K)p_i^2(x) \\ & + A_2^*(K)\alpha_i^2(x)]\xi_i' + K^2 A_3^*(K)\alpha_i^2(x)\xi_i \\ & + \mathcal{L}_b h_i(x) + \mathcal{D}_b p_i(x) + \mathcal{M}_b \alpha_i(x)\ \}\, dx \end{aligned} \tag{2}$$

where H_1^*, P_1^* and A_2^* are flutter derivatives for the deck, and $\mathcal{L}_b$, $\mathcal{D}_b$ and $\mathcal{M}_b$ represent the buffeting forces due to wind turbulence.

The formulation includes the effects of contributions from torsional (α_i), vertical (h_i) and lateral (p_i) motions of the bridge. The full details of the analysis for flutter and buffeting are under preparation in a separate paper [1]. Presented below are summaries of the important features of the analysis.

ANALYSIS FOR FLUTTER

The net condition for flutter in a particular mode, i, may be written in terms of the flutter derivatives as:

$$H_1^* G_{h_i} + P_1^* G_{p_i} + A_2^* G_{\alpha_i} \geq \frac{4\zeta_i I_i}{\rho B^4 L}\left[1 + \frac{\rho B^4 L}{2I_i} A_3^* G_{\alpha_i}\right]^{1/2} \tag{3}$$

The quantities G_{h_i}, G_{p_i} and G_{α_i} represent modal integrals over the partially-completed deck, given by

$$\int_0^\ell q_i^2(x)\frac{dx}{\ell} = G_{q_i} \tag{4}$$

where $q_i = h_i$, p_i, or α_i.

Proclivity towards flutter can be seen to occur when one, or more, of the terms on the left-hand side of Equation (3) becomes large and positive. If this occurs, the negative aerodynamic damping becomes larger than the positive structural damping, yielding a net negative system damping and therefore instability. In typical bridge-deck flutter situations, the A_2^* component of (3) commonly dominates the stability criterion when the mode in question exhibits significant torsional components of motion.

However, in the case of the Baytown bridge, the coupling between the modal component degrees of freedom results in modes with large amounts of torsional motion also having significant vertical translation. The positive contribution to instability due to the A_2^* coefficient described above is overcome by the large negative contribution of the H_1^* term. In other words, the positive aerodynamic damping supplied by the vertical deck motion more than compensates for the potential instability due to torsional flutter. This effect was noticeable in a number of modes, where the present method of study emphasized the various participating aerodynamic mechanisms involved.

Clearly, in these cases, the overall stability of the section was due to the geometric and dynamic characteristics of the structure. Aerodynamically, the bridge deck configuration was such that the leeward deck contributed important aerodynamic damping to the windward deck in torsion; stability was further ensured by important contributions to the damping by sway and vertical motions. The discriminations among contributions to stability could not be made simply by means of an overall three-dimensional wind tunnel study. Stability in such a case could be observed, but the reasons for this behavior would not be obvious. The analytical study thus emphasized the participating mechanisms involved.

ANALYSIS FOR BUFFETING

Taking the Fourier transform of the equation of motion (1) yields:

$$[C(K) + iD(K)]\bar{\xi} = \frac{\rho B^4 \ell}{2I_i}\int_0^\ell \bar{\mathcal{F}}(x, K)\frac{dx}{\ell} \tag{5}$$

where

$$C(K) = K_i^2 - K^2(1 + \frac{\rho B^4 \ell}{2I_i} A_3^* G_{\alpha_i}) \tag{6}$$

$$D(K) = 2\zeta_i K_i K - \frac{\rho B^4 \ell}{2I_i} K^2 [H_1^* G_{h_i} + P_1^* G_{p_i} + A_2^* G_{\alpha_i}] \quad (7)$$

$C(K)$ and $D(K)$ representing aerodynamically-modified frequency and damping, respectively, and

$$\begin{aligned} \bar{\mathcal{F}}(x, K) &= \bar{\mathcal{L}}_b(x, K) h_i(x) + \bar{\mathcal{D}}_b(x, K) p_i(x) \\ &\quad + \bar{\mathcal{M}}_b(x, K) \alpha_i(x) \end{aligned} \quad (8)$$

From equation (5) there may be developed an expression for the power spectral density $S_{\xi_i}(K)$ for the generalized coordinate ξ_i, i.e.

$$\begin{aligned} S_{\xi_i}(K) &= \left[\frac{\rho B^4 \ell}{2I_i}\right]^2 [C^2(K) + D^2(K)]^{-1} \\ &\quad \times \int_0^\ell \int_0^\ell S_{\mathcal{F}}(x_A, x_B, K) \frac{dx_A}{\ell} \frac{dx_B}{\ell} \end{aligned} \quad (9)$$

where $S_{\mathcal{F}}(x_A, x_B, K)$ is the cross power spectral density of $\bar{\mathcal{F}}(x, K)$ between spanwise locations x_A and x_B.

Buffeting was studied in the lowest, most flexible modes of each of the three stages. Buffeting response was checked in what can be described as "weather-vane" (where the decks rotate essentially as rigid bodies about a *vertical* axis through the center of the towers) and "see-saw" (where the decks rotate essentially as rigid bodies about a *horizontal* axis through the towers) modes. In all cases, it was found that the most critical mode was the "see-saw" or rocking mode, and the most extreme was an estimated peak-to-peak deflection at the tip of the deck of 49.3 in. in mode C-3. The results are summarized in Table 1. It should be noted that a correction has been applied to the stage B results to account for the finite aspect ratio of the relatively short deck and the resulting non-two-dimensionality of the flow field around the structure.

CONCLUSIONS

The tendency of a long-span bridge structure to exhibit aeroelastic instability is due to a complex interaction between the dynamics of the structure itself and the aerodynamic damping effects due to the wind. In the case of the higher modes of cable-stayed bridges, where significant coupling exists between the translational and torsional degrees of freedom, proclivity to flutter in torsion is often hindered, and indeed prevented, by the aerodynamic damping provided through the coupled translational motions.

The stability of the structure can often be directly attributed to this structural coupling, rather than to the aerodynamic design of the bridge deck itself. Such a conclusion is only possible by considering the analytical/empirical modeling technique described herein. Three-dimensional wind-tunnel studies, while most valuable in their own right, are unable, however, to differentiate among the physical processes involved.

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Stage B:

Mode No	Freq (Hz)	Rocking Pk to Pk (in)	Lateral Pk to Pk (in)
1	0.189	36.95	-
2	0.289	18.42	1.48
3	0.463	5.94	1.66
4	0.515	-	1.91

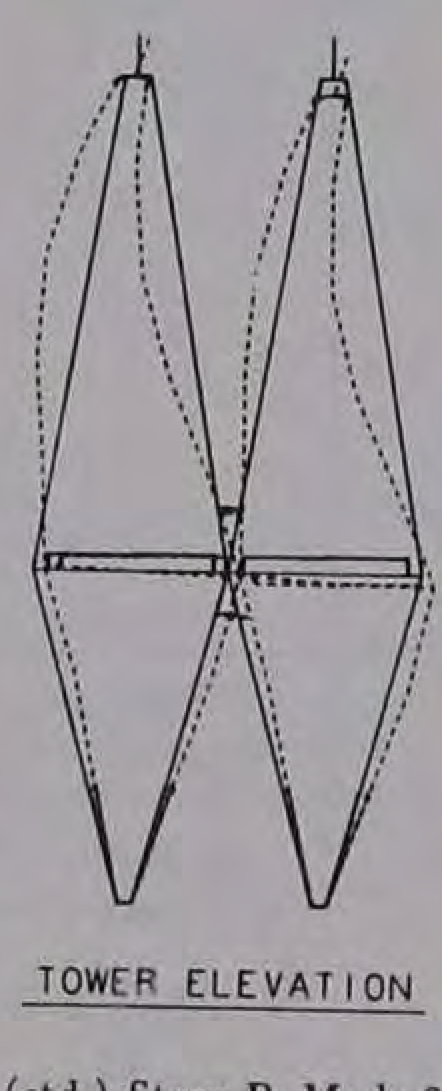
Stage C:

Mode No	Freq (Hz)	Rocking Pk to Pk (in)	Lateral Pk to Pk (in)
1	0.143	7.72	18.36
2	0.169	-	13.40
3	0.224	49.25	-
4	0.307	41.17	-
5	0.521	9.32	-
6	0.580	2.20	0.36

Stage D:

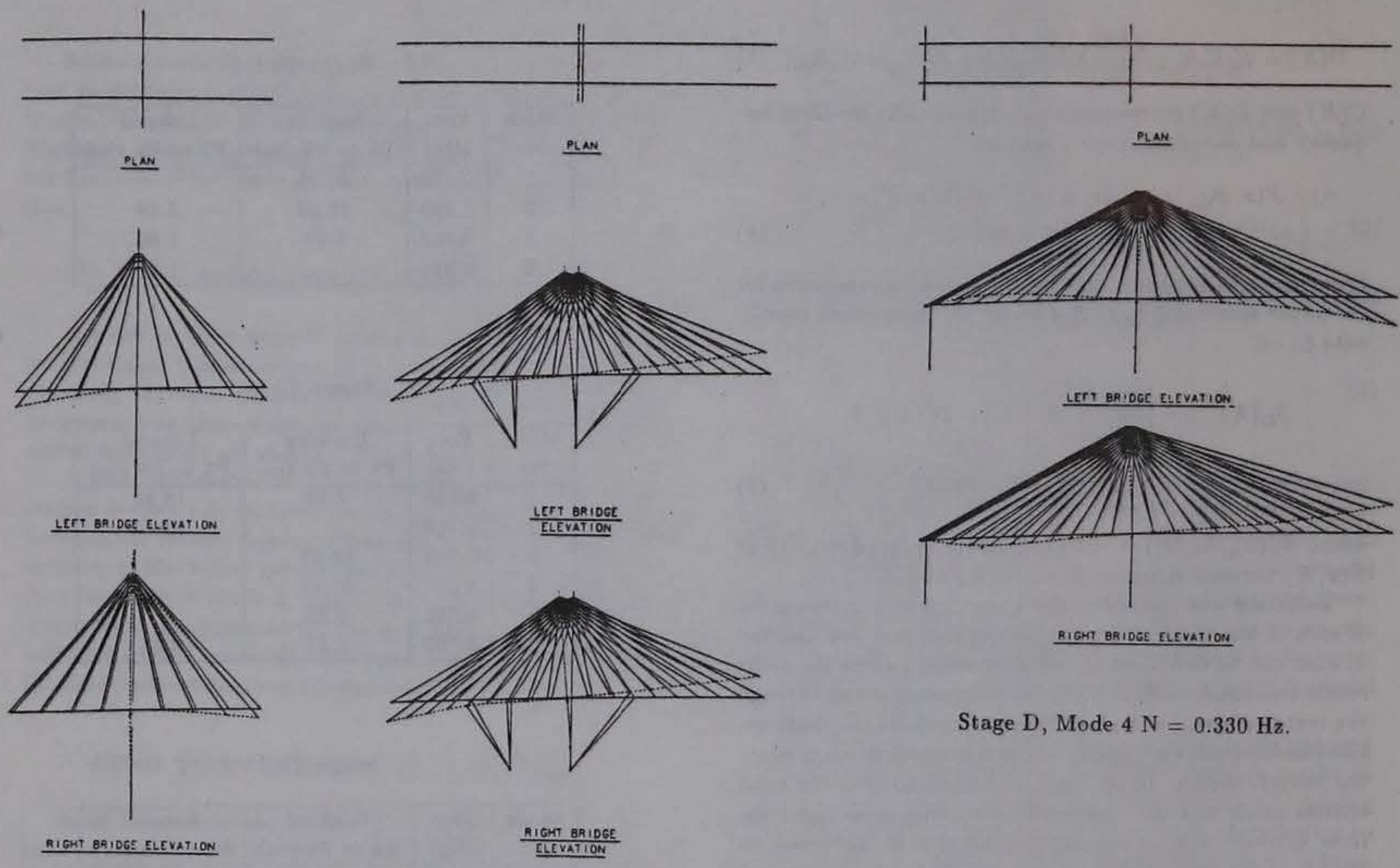
Mode No	Freq (Hz)	Rocking Pk to Pk (in)	Lateral Pk to Pk (in)
1	0.281	-	4.07
2	0.307	33.51	-
3	0.322	-	3.17
4	0.330	40.33	-

Table 1: Summary of Buffeting Responses: Baytown Bridge - Construction Stages



TOWER ELEVATION

Figure 1: (ctd.) Stage B, Mode 8 N = 1.17 Hz.



Stage B, Mode 8 N = 1.17 Hz.

Stage C, Mode 3 N = 0.224 Hz.

Stage D, Mode 4 N = 0.330 Hz.

Figure 1: Sample Mode Shapes of Partially-Constructed Bridge

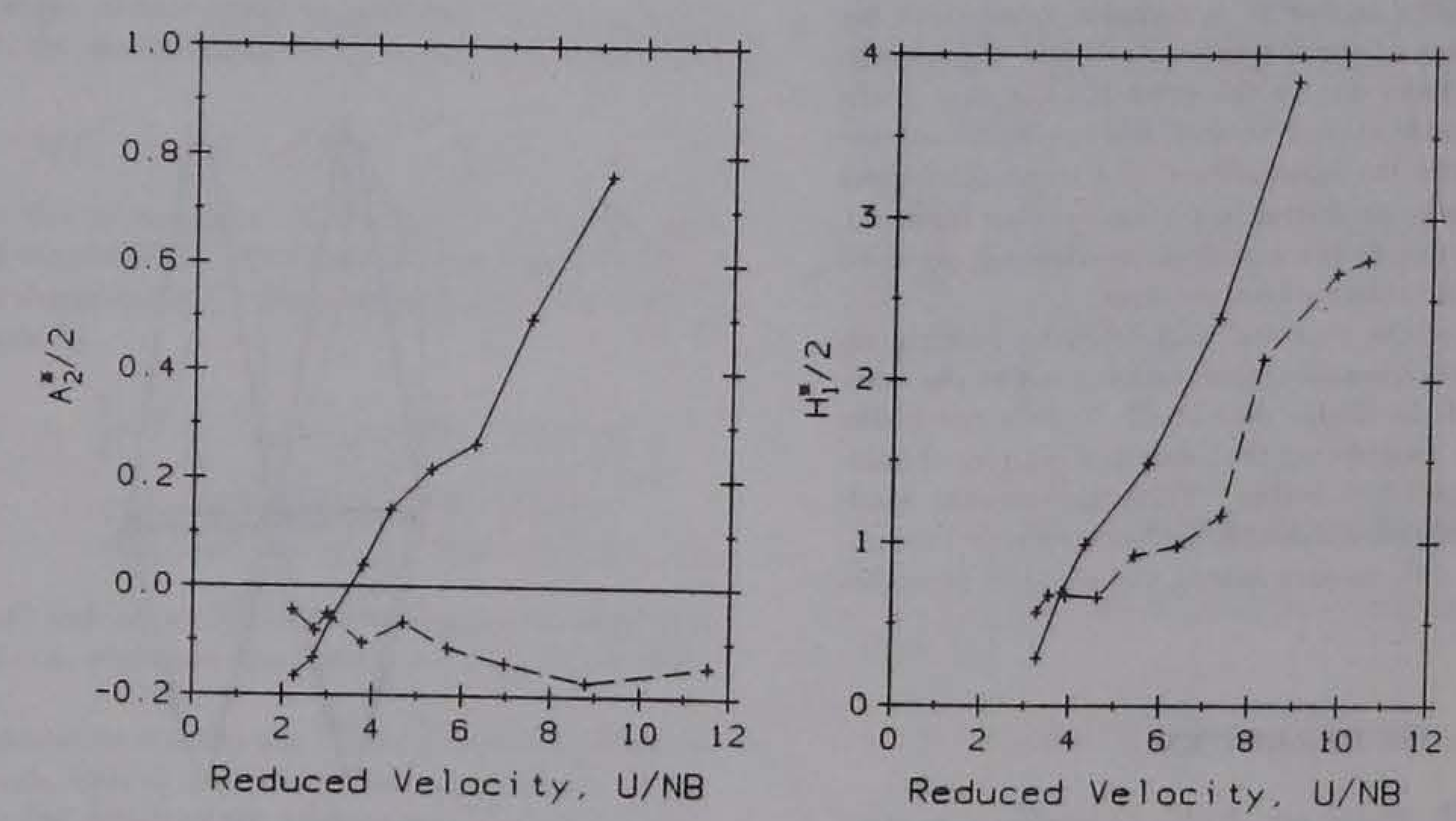


Figure 2: Flutter Derivatives for Deck During Construction
—— Windward Deck, - - - Leeward Deck

SESSION IIIB

AN AUTOMATED PROCEDURE FOR THE REGULATION OF OVERWEIGHT VEHICLES

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SYNOPSIS

A methodology for the evaluation of overload vehicles is being developed. A formula approach is calibrated against a rational method to yield ratings with a consistent reliability. The rational method is based on current rating practice. An automated approach is being developed based on a graphical information system (GIS) that links truck, route, and structure data to perform ratings of all structures along a requested route. The combination of the formula method and the GIS will give the permit authority both the speed and consistency presently unavailable.

ABSTRACT

Methods for the evaluation of overloads on bridge structures have been studied for many years. Several methods have been developed based on simple formulae involving the axle spacings and weights. All such formulae are approximate and often do not yield a rating or gross weight evaluation consistent with more exact procedures. A methodology to rate bridges for overload is being developed that is based on a formula procedure combined with a rational analysis founded on structural mechanics . This combination utilizes the speed of the formula method and the consistency of the rational method to yield a reliable rating.

The percentage of overload trucks that create an overstress situation is presently unknown. Thus, a data base of load effects from representive vehicles on various structure types and configurations is to be developed and subsequently used to compare the formula and rational methods. This comparison will be used to adjust the formula method to yield predictable rating results. In other words, a formula will be developed to permit an acceptable percentage of overload trucks to slightly overstress the bridges (beyond the operating level). This data base will provide more consistent data upon which to make overload policy decisions.

This methodology is to be implemented on an engineering workstation. The mainframe bridge rating program BRASS will be used to generate influence lines for inventory structures. These data will be archived for use by the workstation-based overload program. Because all the computational intensive finite element calculations are done apiori, interactive response times are expected. A graphical information system will be developed to link the graphical representation of a truck's route to the necessary structure data for the rating. A significant task will be the linking of truck, route, and influence line data into an efficient graphical information system.

BACKGROUND

The passage of overweight vehicles on Wyoming's bridges is controlled through a permit system which is based on the axle configuration diagrams. The usage of these diagrams are very fast, often giving truck operators an immediate response to an overload request. But the accuracy of this methodology is quite variable for different structures and load configurations. Further, many situations arise that are not addressed by the these diagrams. Such cases are forwarded to the Bridge Design Branch for analysis. This usually results in a time consuming and costly rating of the structures along the requested route. Wyoming, like many other states, is experiencing a marked increase in the number of permit applications for very heavy, multiple-axle loads that are not addressed by the present methods(1).

Bridge engineers have sought efficient methods for evaluating the load effect on bridges for many years. One of the first efforts was in the 1946 AASHO specification. The load was limited by single axle weight, tandem axle weight, and by gross weight as a function of axles configuration. The gross weight was not to exceed

$$W = 1025(L + 24) - 3L^2$$

where W is the weight in pounds and L is the out-to-out dimension of the extreme axles in feet(2).

The Federal Highway Act of 1956 had similar but less restrictive provisions for loading. Research that progressed in the subsequent years culminated in the U.S. House document that contains the well known "Formula B"(3,4). In conjunction with this recommendation, the single and tandem axles load limits were increased. The Formula B is

$$W = 500\ (\ LN/(N+1) + 12N + 36\ )$$

where N is the number of axles in the vehicle or load group, W and L are defined above. This equation has been amended and extended in many different ways by individual states, such that state to state laws vary considerably(1). A methodology is needed that is easy to use, based on sound analytical rationale, and can be easily modified to accommodate the changing condition of the inventory, bridge construction practice, and departmental policies.

A recent report prepared for the Federal Highway Administration by the Texas Transportation Institute(TTI) attempts to address this situation. A new bridge formula is proposed which is illustrated in Figure 1 (5). This procedure is based on simple span analysis and therefore is conservative for continuous bridges. Note the form of the proposed formula is similar to previous equations, is quite simple and is easy to automate. The formula is based on a 1.05 stress factor for HS20 bridges and 1.30 for H15 bridges.

In a recent report by the Wyoming Highway Department (WHD) Bridge Design Branch(1), the TTI HS20 formula was modified to increase the allowable stress to 1.30 times the inventory design stress. This factor is quite close to the operation level for most materials(1). The current WHD practice, the TTI formula, and the WHD modified formula (hereafter referred to the WHD formula) are compared in Figure 1. The WHD formula is currently under study as described herein. As apparent in the Figure 1, the WHD formula uses the higher stress level. The WHD tested this WHD formula using real and contrived loadings on 23 HS20 and 13 H15 bridges in the WHD inventory. These rating are based on girder-line analyses using the AASHTO distribution factors. The average rating factor for the HS20 bridges was 1.11 with a standard deviation of 0.23. This indicates that the WHD formula may be too permissive within the framework of current rating practice. The work described herein will generate the necessary data to more accurately calibrate the formula method.

The formulae described above works well for trucks with limited configurations moving across structures that are simple spans or have influence lines similar to that of a simple span. Deviation from the relatively "standard" cases causes a large variability in the permitted loading. This is illustrated in the WHD study(1) with approximately one third of the bridges with rating factors outside of the range 0.88 to 1.34.

This project will use the formula method combined with ratings based on the actual geometry, material and cross sectional properties, etc. This merger of approximate and rational analysis (RA) provides an efficient and consistent methodology for the evaluation of overloaded vehicles.

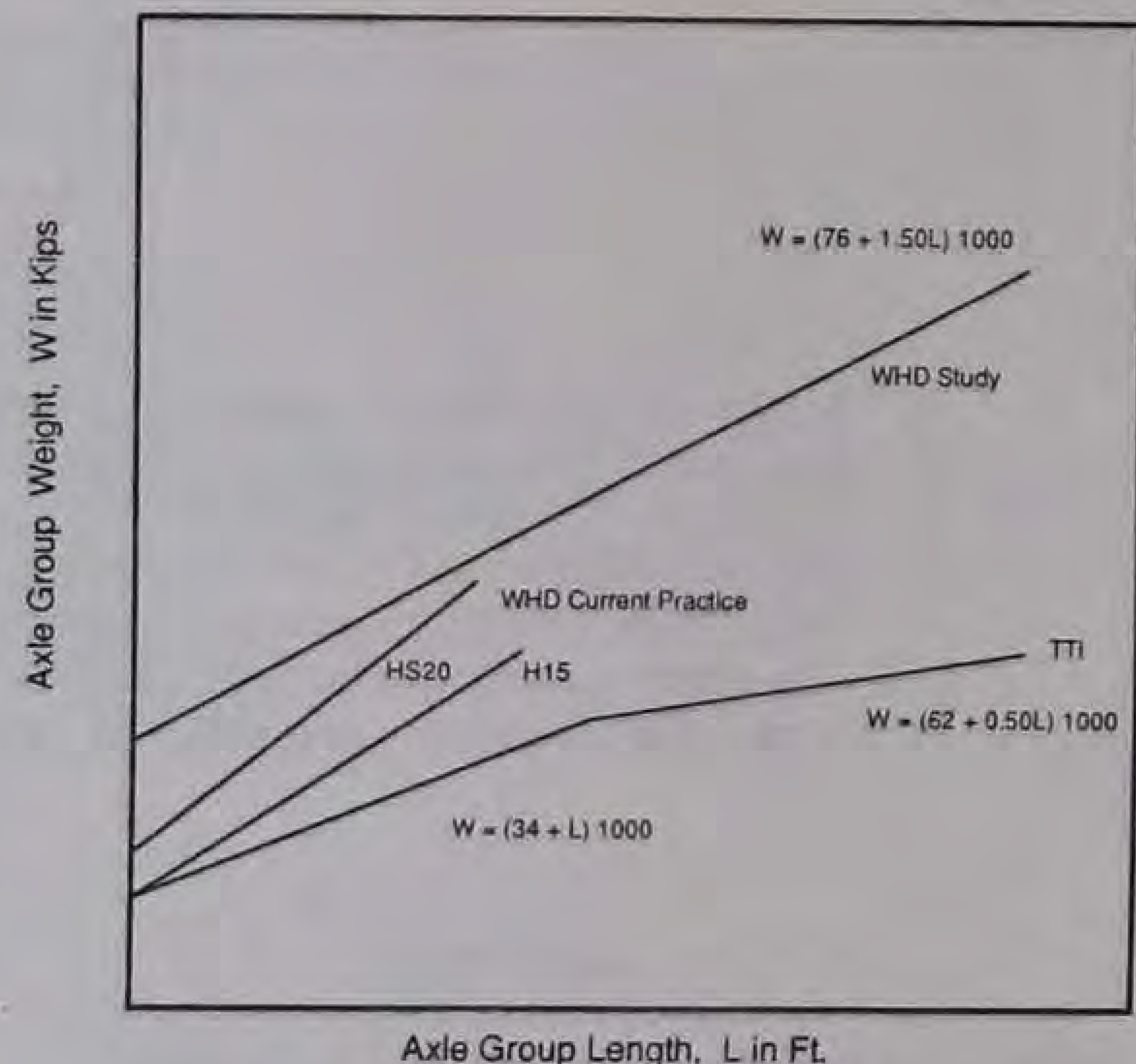


FIGURE 1. Overload Formulae

PROJECT TASKS

The project is to develop an interactive overload rating system for the utilization in the permit process. The project tasks are fourfold:

1. Develop formula procedure to rate overload vehicles within prescribed confidence limits.

2. Develop an interactive user interface for the implementation of the formula developed above.

3. Develop a rational rating procedure based on structural analysis to compliment the formula analysis procedure.

4. Develop a graphical informational system (GIS) to link truck, structural analysis, and routing data for the RA method.

Formula Procedure

It is well recognized that simplified formulae can not accurately predict *all* overload situations on *all* structures. The proposed work is to develop a formula based method (hereafter referred to as formula analysis, FA) that can be used to properly rate overloads for a *predictable percent* of the occurences. Presently, the percentage of overload vehicles that are overstressing the structures is unknown.

Interactive Interface/Formula Analysis

The user interface will prompt the operator for the data required on the "Special Permit for Operation of Oversize Loads" form. The user next rates the vehicle using the FA method. If acceptable, then the vehicle is permitted. If unacceptable the route is established and the truck is rated by the RA method. The procedure is graphically depicted in Figure 2.

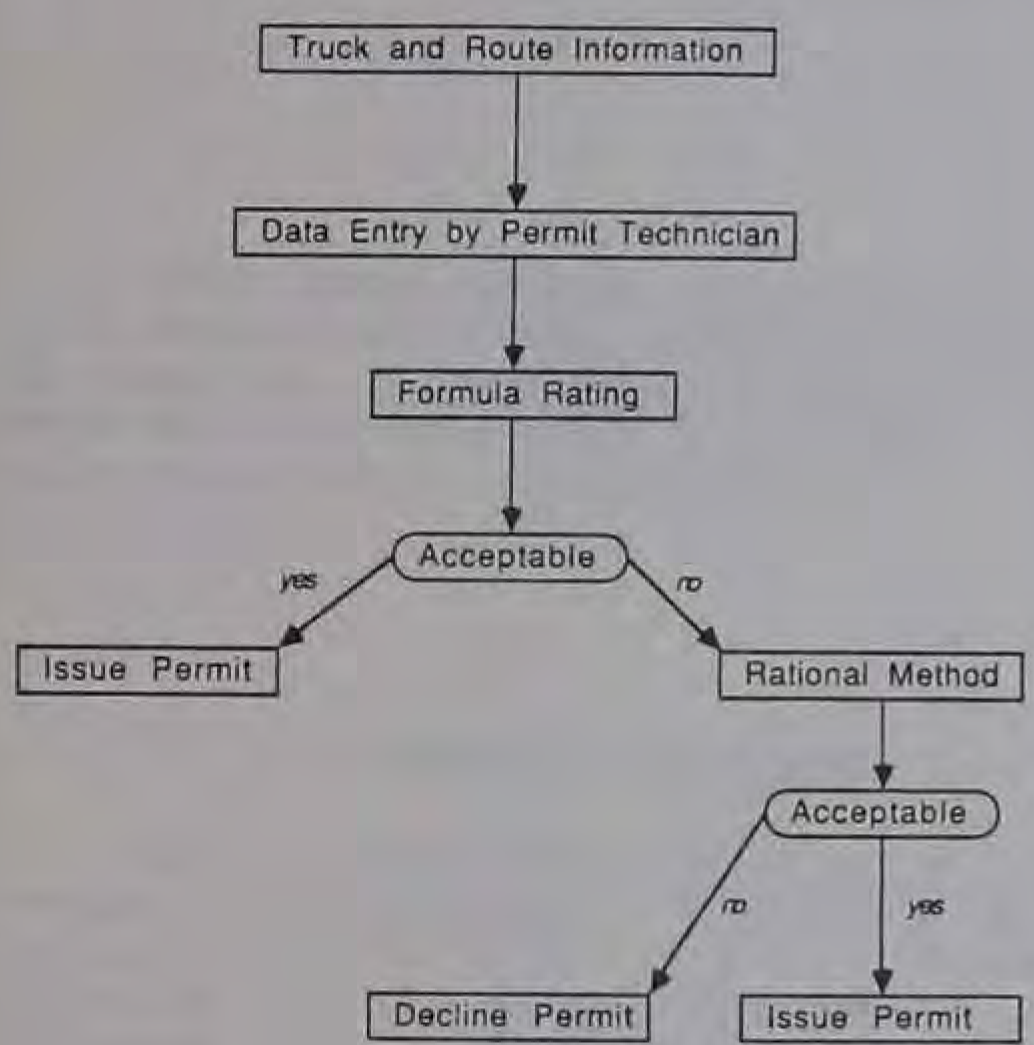


FIGURE 2. Permit Procedure

Rational Analysis

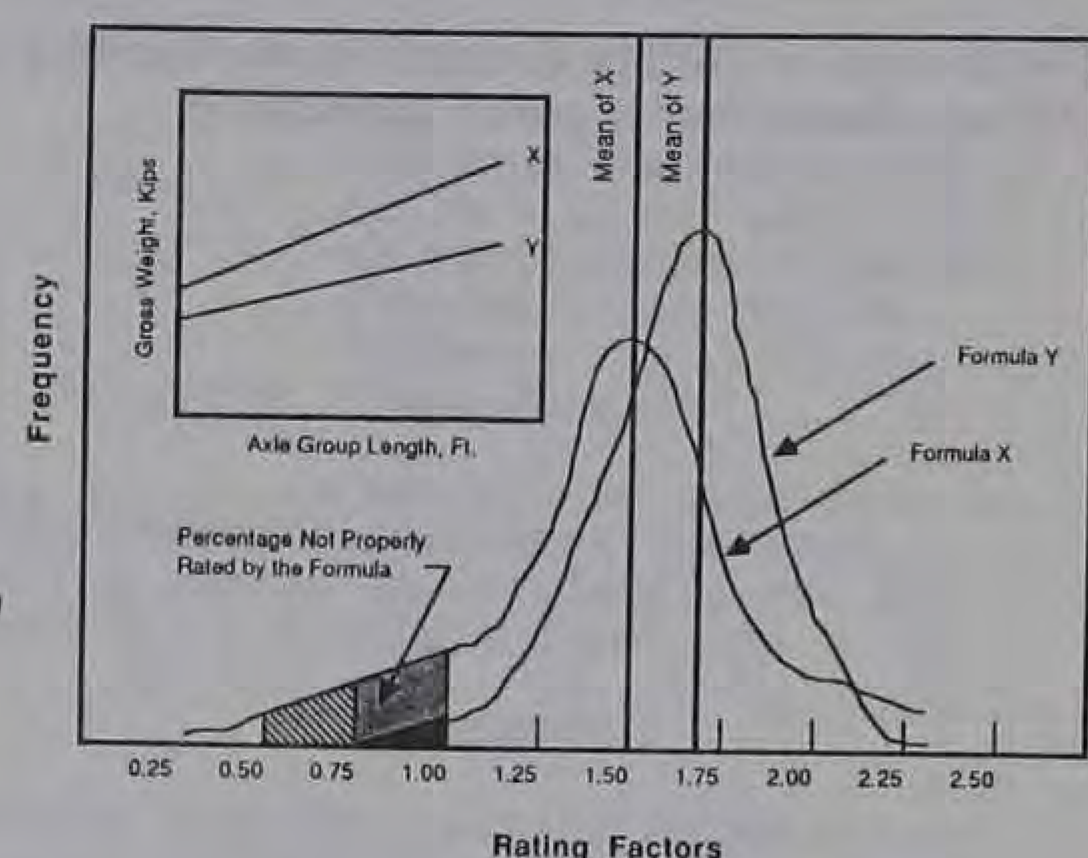
The rational analysis is performed only if the structure does not meet the FA criteria. This analysis will be based on influence line data for rating factors generated by BRASS which is based on girder-line analysis. The geometry, material and cross sectional properties, dead load, and distribution factors are included in the influence lines. The influence line data base is established apriori by analyzing structures for a unit loading. This data base can be established for the entire bridge inventory and accessed by overload program. Efficient structures for this type of data have been developed in previous research efforts and are readily available.

Graphical Information System

A input system will be required to link the truck, route, and structural analysis information. Because the route is best defined graphically, a map will be used to mark the route and define the data links necessary to address the structural information consisting of influence functions generated by the finite element analysis. At the time of this writing two methods are being investigated. The first employs a CAD package to define the maps, mark routes and produce a file of roadway segments that can be subsequently linked to structure information. The second option is to write a program that overlays a pixel map representation of the required maps. The route is defined in the program and the necessary links are made. An elaboration of the advantages and disadvantages of each is beyond the scope of this paper.

FORMULA CALIBRATION

Numerous FA and RA analyses will be used to calibrate the FA to achieve statistically reliable results. As illustrated in Figure 3, the more permissive formula (Formula X) will have more rating factors below the acceptable level. By adjusting the formula parameters (say to Formula Y), the frequency distribution function will shift accordingly. Hence the formula can be adjusted to meet prescribed highway department policy. In other words, statistically, this line can be set such that a structure rated with the FA method will unlikely have rating factor of less than a target value. The probability of a rating factor being less than unity may be statistically based. For example, if a five percent (0.05) probability of a FA rating falling below unity combined with a one percent (0.01)



Note: Formula Y is the most conservative and has the smaller area under the curve below the rating factor 1.00. The intercept and/or slope is calibrated to yield an acceptable percentage of formula analysis that may incorrectly assess the load effect. The two lines illustrated show two discrete possible formulae.

FIGURE 3. Calibration

probability of a FA rating falling below a 0.90 are the criteria, then the FA curve(equations) can be adjusted (Figure 1) so that these criteria are met. Clearly, the bridge administrator can "fine tune" the procedure per policy measures regarding stress levels and type of analysis as dictated by the FA acceptance criteria.

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ACKNOWLEDGEMENT

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BRIDGE DECK REPAIR AND PROTECTION DECISION MAKING: ONGOING RESEARCH

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SYNOPSIS

Past research has developed forecasting models for bridge deck deterioration. Since such models are based on mean response values, they are useful at the policy making level. In contrast, such forecasting is of little assistance for maintenance decisions with regards to a site specific bridge. Herein, an overview of a proposed effort to handle site specificity is presented.

PREVIOUS RESEARCH

Bridge deck deterioration is a serial sequence of processes, Figure 1. The initial process is a diffusion of the chloride ion throughout the mature concrete deck. After some time period a critical chloride concentration level is reached at the depth of the reinforcing steel. Subjected to the critical chloride concentration the reinforcement steel begins to corrode. The corrosion results in material expansion which in turn leads to spalling. Eventually, the spalling becomes sufficiently extensive to warrant failure.

The computation of the time to failure, t, is the summation of the three process times, Δt_i,

$$t = \sum_{i=1}^{3} \Delta t_i \qquad [1]$$

Consequently, researchers have developed formulas (analytical and empirical) for evaluating the process times, Δt, (1,2,3). For example, the chloride ion diffusion was found to follow Fick's second law (1),

$$\frac{\partial C}{\partial t} = D\,\frac{\partial^2 C}{\partial x^2} \qquad [2]$$

where:

- C = chloride ion concentration
- t = time
- x = depth into the concrete
- D = diffusion constant

The solution to this differential equation was taken as (1),

$$C_{(x,t)} = C_o[1-\mathrm{erf}(x/2\sqrt{Dt})] \qquad [3]$$

where

- $C_{(x,t)}$ = chloride concentration at depth x after time t for an equilibrium chloride concentration at the surface.
- erf = error function

If $C_{(x,t)}$ is set to the critical chloride concentration then the time period for corrosion initiation can be computed for a specific depth, x.

THE PROBLEM

While such forecasting formulations are useful for policy making, they become ineffectual in site specific decisions. With the mean values provided by the present forecasting formulations, decision makers can plan the needed manpower and resources for future patching, overlays and replacement for an infrastructure. However, the question of what repair should be performed for a specific bridge remains unanswered since its deterioration state is unknown.

In fact use of such forecasting for site specific decisions could be quite erroneous. As an example, if deterioration can be assumed linear, the importance of site specifics become clear, Figure 2. (It must be stated that the linear deterioration function is solely for illustrative purposes. The authors are not advocating the linear form as accurate at this time. Indeed, the functional forms remain a research item.)

At the initial time zero, each deck is at a specific deterioration level. The initial state depends upon such variables as workcrew experience and the construction environment. While deck 1 may be the product of a highly experienced crew working under favorable conditions, deck 2 may have been constructed by an inexperienced crew under adverse conditions. Consequently, deck 2 does not have the initial quality of deck 1, ($d_2 < d_1$), Figure 2. On the other hand, deck 1 may be subjected to a highly salt-contaminated environment versus a milder salt-contaminated environment for deck 2. Such environmental variations result in different deterioration rates (slopes) Figure 2.

With respect to Figure 2, it becomes evident that traditional inspection/ maintenance decisions can be less than optimal. During the early service periods, deck 2 would have a higher percent deterioration level than deck 1. Consequently, deck 2 would be expected to reach the predetermined failure level earlier and so receive higher priority. Likewise at some medium time, both bridges would have the same maintenance priority.

The problem becomes even fuzzier when the question of failure is added to the maintenance decision-making process. In other words, what constitutes failure? Traditionally, deck failure has been defined at some specified percent of surface spalling. However, it is conceivable that the percent of spalling may greatly influence the selection of an acceptable repair technique. Perhaps, at a lower level of deck deterioration a less expensive repair may enhance the service life more so than the application of an expensive repair once a maximum allowable deterioration level is reached.

Such problems are the results of the inadequacy of traditional analysis in addressing initial quality and service environment which influence the deterioration rate.

PROPOSED RESEARCH

The crux of the problem lies in the development of a formal means for dealing with such vague variables as workcrew experience, construction environment, severity of salt-contamination and the definition of failure.

With regard to applicable vague variables, some preliminary studies have been conducted. An expert encounter session was arranged to acquire data [4]. The opinions were grouped into categories. For example, the dependability of a repair technique was rated as "good" versus "very good." Also, in conjunction with such opinion, the experts were asked to quantify the term good or very good on a scale ranging from zero to ten [5]. In turn, the typical categories used in deck evaluation were defined as fuzzy triangular numbers, Figure 3. Simple fuzzy algebra was employed to see if a rating scheme would be feasible.

The focus of the preliminary study was a feasibility evaluation. While the results did indicate the concept as feasible, the membership function definitions require substantial study [6]. Questions with regard to applicable fuzzy variables, quantification scheme and fuzzy logic employed are presently unanswered.

Research is proposed to dress existing databases, that is, what is available, and of that, what is useful. For example, the salt-contamination environment needs to be defined. Based on experience and historical records the service environment is to be delineated into fuzzy sets. In turn, deterioration functions are envisioned for the categories delineated, Figure 4.

RESEARCH OBJECTIVE

Based on the fuzzy categories identified, an evaluation scheme for repair techniques is to be developed. The initial use of the evaluation model is for ranking the reliability of various repair techniques for specific states of deck deterioration. Eventually, the evaluation model is expected to be an integral part of an optimizing maintenance scheduling program.

The skeletal concept of the expected evaluation model is given by Figure 5. The time deterioration model is the product of previous research, as discussed. The ongoing research seeks to add the fuzzy data, (environmental conditions and initial structural integrity). The output is a statement of the service state for the bridge deck.

With a stated bridge deck service state, the effectiveness of a repair technique may be determined. The repair upgrades the initial deck integrity. Consequently, the time-deterioration model upgrades the bridge deck service state. This circuitous analysis will provide a comparison of possible repair methods. In other words, a ranking would be defined.

With the effectiveness ranking completed, a decision aid for maintenance scheduling would be the next development.

In addition to environmental and initial conditions, the deterioration model conclusion (service state) would be updated with site inspections. The service state determined would then r the input for selection of repair from the previous determined ranking.

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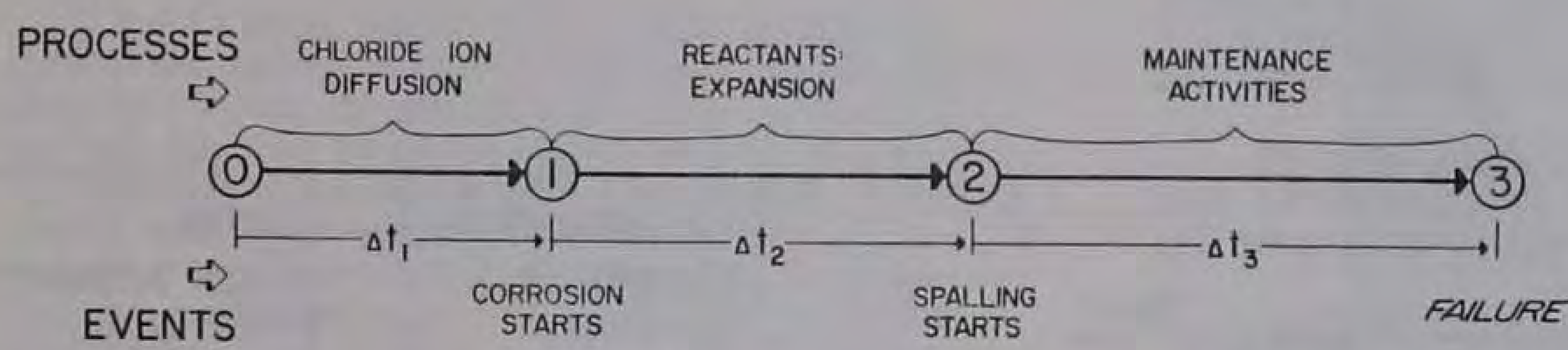

Figure 1: Bridge Deck Deterioration

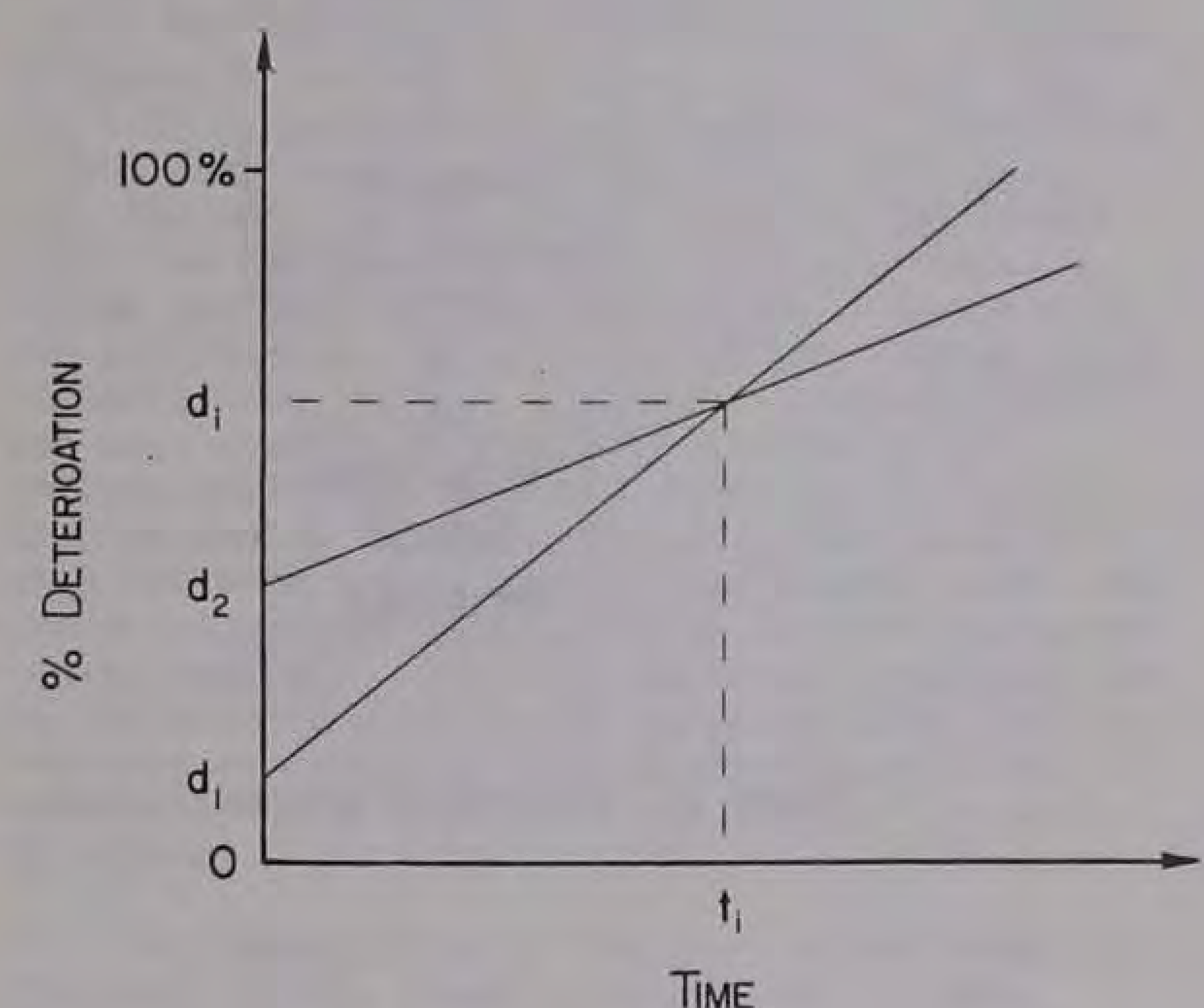

Figure 2: Linear Deterioration Models

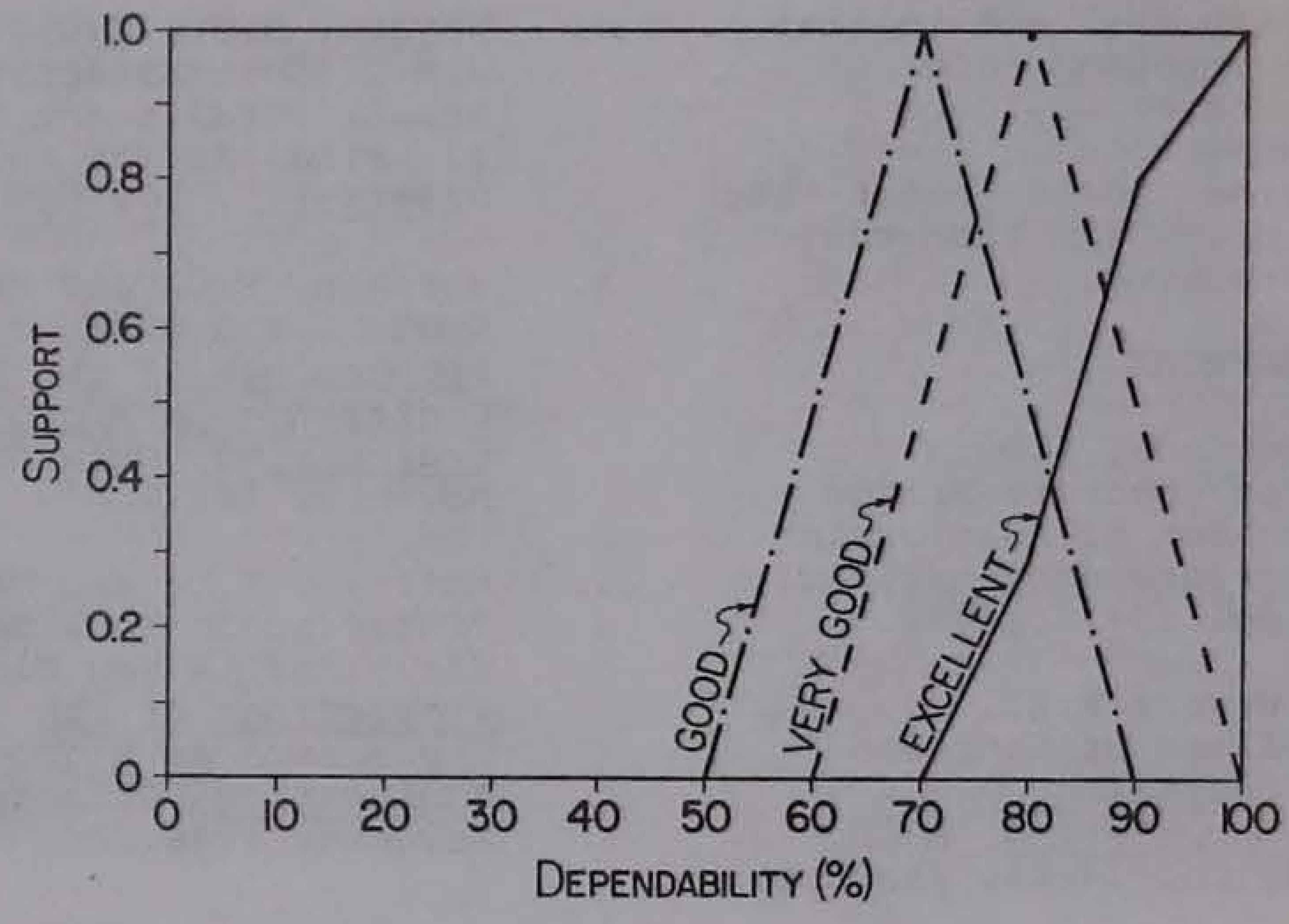

Figure 3: Membership Functions

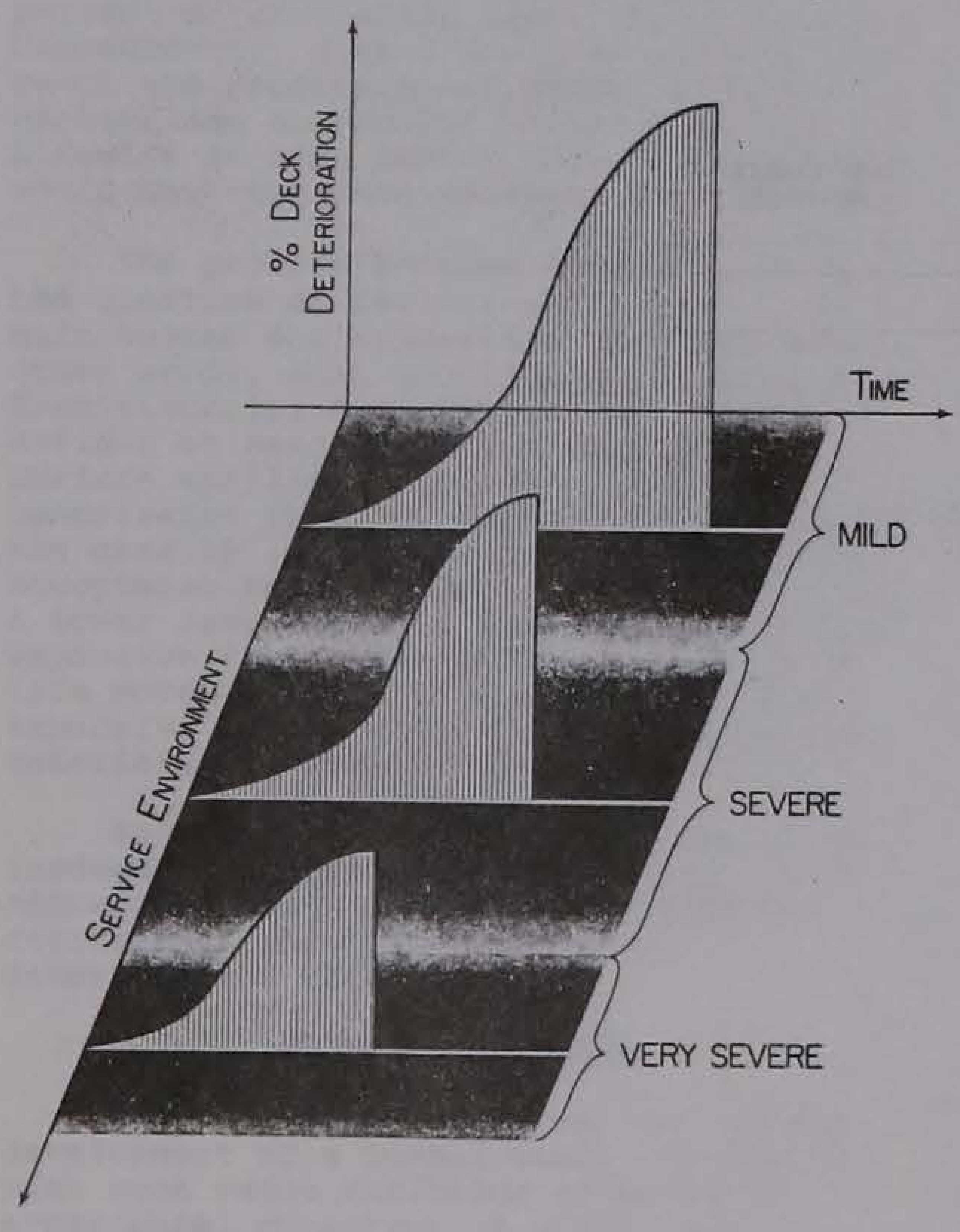

Figure 4: Fuzzy Categories and Deterioration Functions

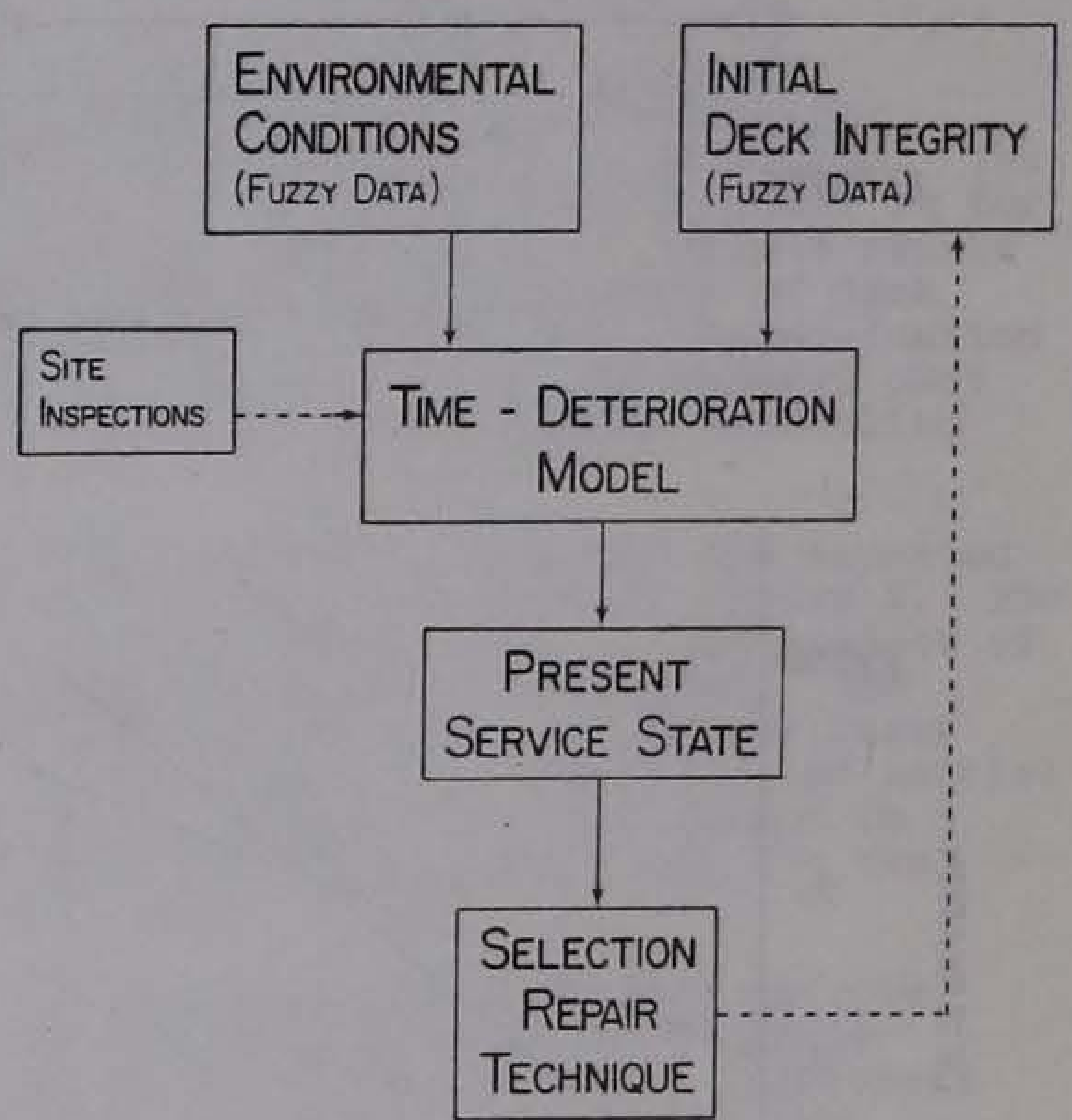

Figure 5: Evaluation Scheme Concept

DEVELOPMENT OF AUTOMATED BRIDGE RATING PROGRAM USING FINITE ELEMENT TECHNOLOGY

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SYNOPSIS

The finite element method is widely accepted as the most accurate method of analyzing slab and girder bridges. The inaccuracies of the standard empirical design and rating procedures are well known. However, even for relatively simple geometries, the work in developing a finite element model and interpreting the results is extensive using typical finite element packages. Work is underway at the University of Florida to make it possible for bridge engineers to obtain the results of finite element models for a wide class of bridges with a minimum amount of data input.

INTRODUCTION

The Florida Department of Transportation (FDOT) has been sponsoring research at the University of Florida in the general area of the analysis and testing of bridges for several years (1,2, 3). Recently, the Federal Highway Administration (FHWA) has begun supporting the work (4,5,6). The thrust of the work has been to take advantage of the three-dimensional nature in which a bridge distributes the vehicle loads to the components of the bridge such as girders, slab and stiffeners.

The work consists of two phases. Phase one was aimed at giving the FDOT a tool for bridge rating that would give them a reasonably accurate estimate of the Lateral Load Distribution Factor (LLDF) for the vast majority of bridges in service in the state of Florida, which were simple span. The LLDF is an empirical concept and is thought of as the portion of a vehicle's loads, divided by two, for which a particular girder in the bridge should be designed. With the LLDF available, a bridge designer or rater can do their analysis of the longitudinal distribution of the loading using standard beam theory. A computer program, SALOD, was developed to be used with a database of influence-surfaces prepared using the finite element method to implement Phase I. The SALOD program has been in use by FDOT for several years and the developmental work on SALOD for FDOT is essentially complete. However, SALOD was developed for an IBM mainframe TSO environment and some interest has been expressed by FHWA in developing a "PC" version of SALOD that would be usable in a number of states.

The second phase of the work is now underway. The goal of this phase is to develop a computerized procedure that can be used to do a complete bridge rating of almost any girder slab bridge. This is to be accomplished using a bridge rating system called BRUFEM. BRUFEM consists of a preprocessor, a finite element module and a postprocessor. A pilot form of BRUFEM is presently available to FDOT (6).

In this summary paper, the general concepts used in developing the two phases of the research are briefly discussed.

LATERAL LOAD DISTRIBUTION PROGRAM SALOD

The function of SALOD is to predict a lateral load distribution factor (LLDF) for design and rating of simple span bridges. A database of influence-surfaces was developed using finite element models. The finite element models considered the three-dimensional nature of the distribution of individual wheel loads throughout the full superstructure. The database was developed to represent existing simple span bridges in the State of Florida.

Figure 1 shows a cross-section of a typical girder-slab bridge and can be used to describe the type of model used in the analyses. In order to keep the database of influence-surfaces manageable in size, a number of simplifying assumptions were made. First, edge stiffening affects were neglected thus only the portion of the bridge inside the basic model was considered. Composite action between the slab and girder was considered by using the composite moment of inertia computed using standard effective width formulae for the girder elements. Thus, the slab elements could be simple flexural plate bending elements.

Skew was neglected as were interior diaphragms. Influence surfaces were developed only for flexure which predominates in the design of most such simple span bridges. These assumptions closely paralleled those used in developing the Ontario Highway Bridge Code (OHBDC) procedures.

However, their analyses were based on orthotropic plate theory or grid analyses. Separate recommendations were made for shear (2).

The user of the SALOD program inputs information about the specific bridge and the vehicle or vehicles crossing the bridge. The SALOD program then interpolates from the appropriate stored database of influence-surfaces and places the vehicle system in the position to develop the maximum value of LLDF based on flexure for each girder. With the output of the LLDF available, the user can do his design or rating in the standard way. Figure 2 shows a plot of LLDF for a series of prestressed girder bridges with all parameters held fixed except span length (2). The dashed curves show the results of SALOD solutions for two standard HS20 trucks. The lower of the SALOD curves is multiplied by 0.9 for comparison with the OHBDC solution which includes a probability reduction factor for two vehicles. AASHTO does not consider a reduction for two vehicles.

The lower SALOD curve compares well with the OHBDC solution; both curves showing the same pronounced variation with span length. The AASHTO solution completely ignores span length and has a constant value. The AASHTO solution is seen to cross the upper SALOD curve at about a span of sixty feet. Thus, the AASHTO solution becomes increasingly conservative with increasing span length.

The prior curves were for two standard HS-20 vehicles. SALOD is even more useful in obtaining LLDF for cases with nonstandard vehicles; since the work done in developing AASHTO and OHBDC criteria was based on standard vehicles. Figure 3 shows an axle layout for a special purpose vehicle. A seventy-five foot span bridge with 5 Type IV AASHTO girders spaced at 6.17 feet was analyzed using SALOD. The SALOD results for the special vehicle alone gave a LLDF of 0.67. A second SALOD solution assuming the special vehicle could cross the bridge simultaneously with an HS20 gave a LLDF of .951. The AASHTO solution for this case would be 1.12. Clearly the use of SALOD is a powerful tool for the rating of bridges for nonstandard vehicles.

In addition to the theoretical studies on SALOD, a series of full-scale load tests were made to confirm the modeling techniques. Details on the load tests are available in Reference 3.

SALOD has been in use by Florida DOT for several years now. They use the program to obtain lateral load distribution factors in conjunction with the rating of simple span bridges subjected to standard and overload vehicles. The use of SALOD has allowed FDOT to avoid unnecessary posting of serviceable bridges in many instances.

BRIDGE RATING PROGRAM BRUFEM

It was felt that to consider skew, edge affects, girder continuity and the other affects that were neglected in developing the SALOD database would require a quantity of influence-surfaces that would be prohibitively expensive to generate and store. Thus, the BRUFEM program is being developed to allow direct modeling and rating of individual bridges using the finite element method.

The modeling techniques in BRUFEM are more extensive than those used to generate the SALOD database. The edge affects outside the basic model, as shown in Figure 1, can be considered. Composite action may be modeled either as described above for SALOD or by using plate bending and stretching elements which are specified eccentric to the girder elements. Thus, the composite action can be considered directly in the analysis, taking into account the nonuniform distribution of lateral compressive forces in the slab, commonly called shear flow.

THE BRUFEM SYSTEM

BRUFEM is a bridge rating system using a finite element model as the basic analysis tool. BRUFEM consists of three programs.

1) A preprocessor that develops a three-dimensional finite element model of the full superstructure from a relatively small amount of input data about the bridge's geometry and stiffness parameters. Typical data includes type of construction (steel girder, standard AASHTO prestressed girder, bulb-tee, etc.), girder spacing, edge stiffeners, skew angle, and material properties.

   Information about the vehicle system is also input and equivalent nodal loads are generated for the vehicle system. Of course, it is necessary to place the vehicle system in a number of positions to determine the critical rating. Thus, a large number of load cases must be developed.

   The preprocessor prepares a datafile for input to a finite element program.

2) A finite element program to solve the model created by the preprocessor. Previously, the McAuto STRUDL program was used. This selection was made primarily because of the availability of this program on the FDOT system and the fact that this version of STRUDL had the necessary plate bending and stretching elements and the capability to handle fairly large models. The finite element program outputs the results to an output file.

3) A post-processor that reads the output file from the finite element program and does the bridge rating based on the appropriate service level or strength criteria. At the present time, the development of the post-processor is restricted to post-tensioned continuous bulb-tee concrete bridges. Field studies have been conducted on the Eau Gallie bridge to confirm the modeling techniques used in the BRUFEM program on a four-span post-tensioned continuous bulb-tee bridge. Details on the test are available in Reference 5. Reference 6 contains documentation on the present version of BRUFEM.

NEW FINITE ELEMENT MODULE FOR BRUFEM - SIMPAL

In order for the FDOT to utilize BRUFEM as their primary bridge rating tool, a large amount of work is required on the post-processor. Also because of the large number of finite element solutions required, it is necessary that the solution time for the finite element model be minimized. One popular new approach to reducing the time for finite element solutions is through the use of vector or parallel processors. Also, some interest has been shown by other agencies in using BRUFEM as a bridge rating program. If BRUFEM is to be widely used as a bridge rating tool by a number of groups throughout the country, it would be advantageous to have a public domain finite element module for the analysis phase.

SIMPAL (7) is a public domain finite element analysis program which is modularly constructed and uses database techniques. Execution on different computers only requires recompilation using a FORTRAN 77 compiler. Work is nearing completion now on the modification of the preprocessor to accommodate SIMPAL instead of STRUDL. Simultaneously, a vectorized option is being added to SIMPAL to allow for faster solutions when a parallel processing machine is available.

COMPLETION OF BRUFEM

It is intended that the BRUFEM program be developed into a complete bridge rating package for simple and continuous span bridges of pretensioned and post-tensioned concrete girders, reinforced concrete slabs and girders, and structural steel girders. The full development of BRUFEM should take approximately four years.

Year 1 - Add bridge types of pretensioned simple span AASHTO and bulb-tee girders, reinforced concrete tee beams with simple or continuous spans, and reinforced concrete slabs with simple or continuous spans. All of these could have either rectangular or skewed decks.

Year 2 - Train FDOT personnel in use of BRUFEM. A preliminary workshop would be held early in the year. Then individual seminars would be held in each of the districts with UF personnel aiding individuals in rating actual bridges. Out of this interaction will come necessary revisions to the program to make it more readily useable by FDOT personnel. A final workshop will be held to make FDOT personnel competent to use BRUFEM for all girder and slab type bridges, except structural steel.

Years 3 and 4 - Extend the BRUFEM rating capabilities to include structural steel girder bridges of simple and continuous spans. Composite action could be considered or neglected. Effects of shoring and segmental construction would be considered. Seminars would be scheduled to make the FDOT personnel competent in using the full range of the BRUFEM program as the primary rating tool for the FDOT.

Full completion of the project would give the FDOT the ability to quickly and accurately rate the load capacity of the vast majority of the bridges in the state of Florida. In addition to the software development, work will continue in verifying the modeling techniques by theoretical and experimental means. Potential savings to the state of Florida from a more efficient use of its inventory of its bridges are quite large. Due to the large development costs and time involved, it does not seem prudent for each state to develop such extensive systems independently. The BRUFEM system could serve as an important part of a national bridge rating system.

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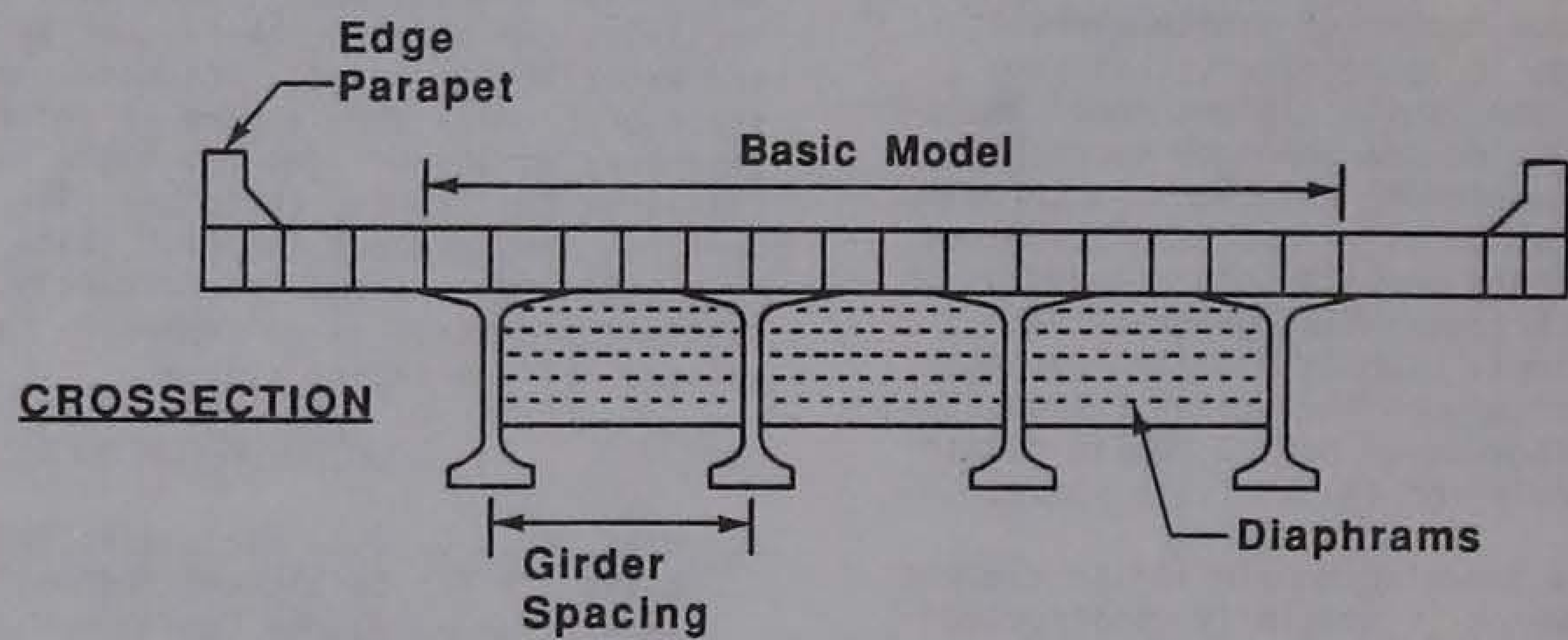

Fig. 1 Typical Girder-Slab Bridge Model

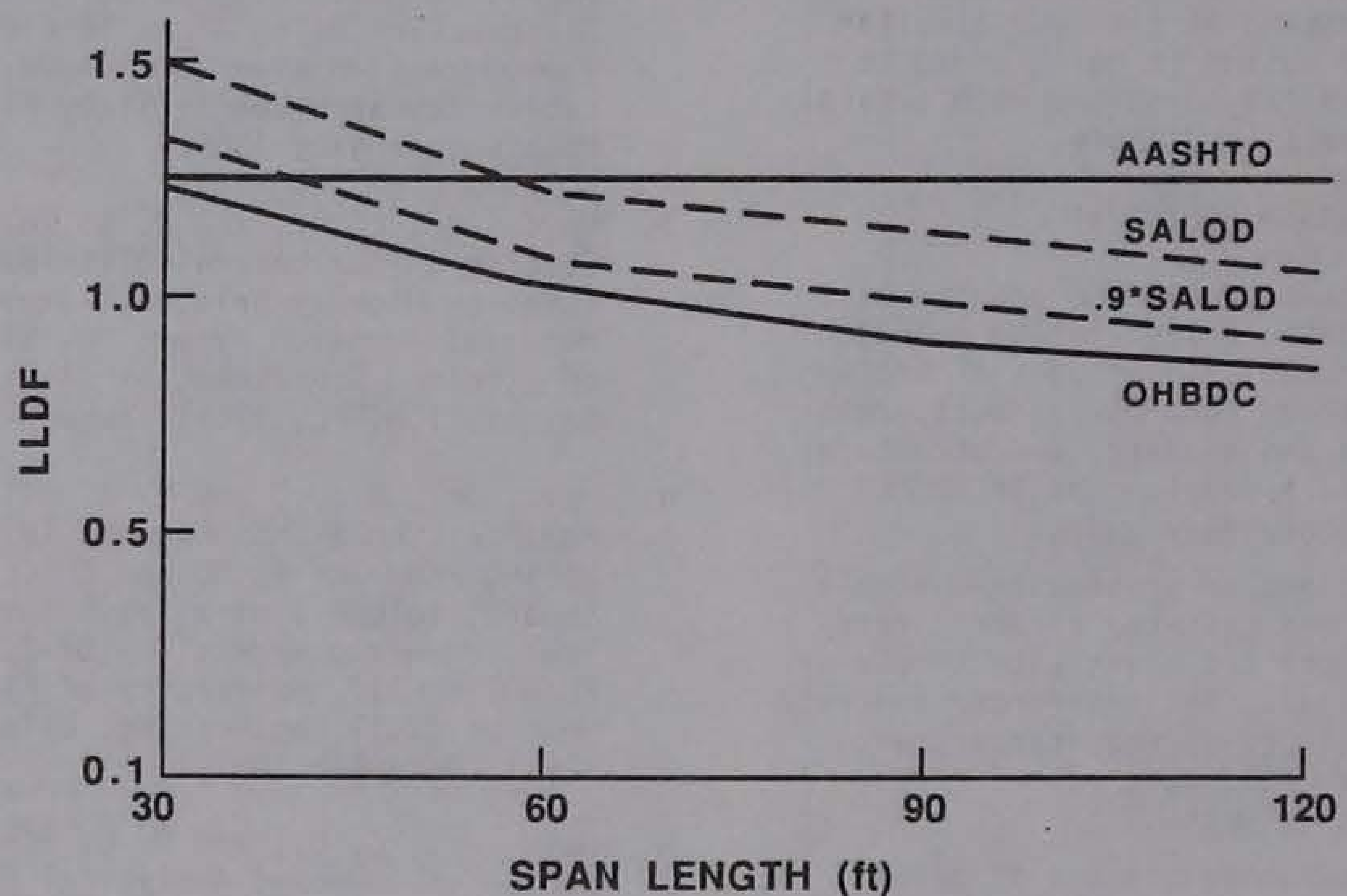

Fig. 2 Variation of LLDF with Span Length

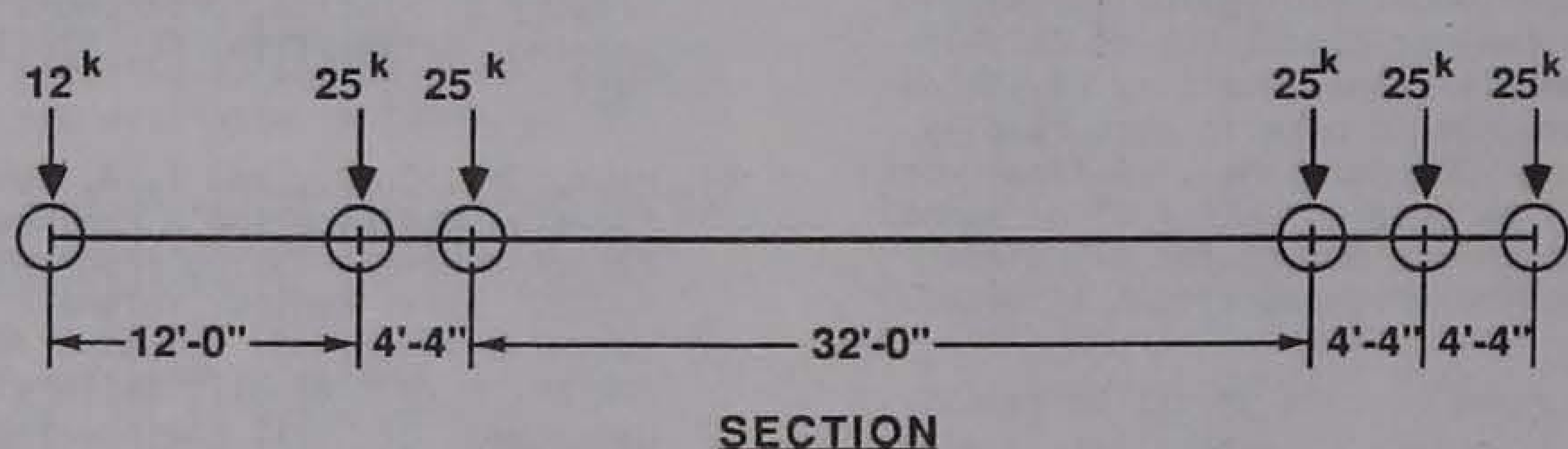

Fig. 3 Axle Configuration for Special Vehicle

INELASTIC RATING OF STEEL BEAM AND GIRDER BRIDGES

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SYNOPSIS

A rating method is being developed for steel plate girder and composite bridges which will incorporate the inelastic capacity of the section, and will utilize the inelastic redistribution of loads due to yielding and three-dimensional behavior. The final product will be a rating manual similar to the current AASHTO "Manual for Maintenance Inspection of Bridges", but reflecting more accurately the difference between designing new bridges and rating existing ones.

INTRODUCTION

The most recent data available from the National Bridge Inventory (NBI) indicates that approximately 42 percent of the bridges in the U.S are either structurally deficient (24%) or functionally obsolete (18%). Steel beam and girder bridges are the most common type of structure in the inventory, comprising approximately 35 percent of the total. The useful life of these bridges could be increased if more realistic and reliable rating procedures are employed. The development of such procedures would have a significant impact on the bridge management systems currently under implementation by many states.

Under the auspices of AASHTO-sponsored NCHRP Project 12-28 (12), a rating procedure utilizing the inelastic capacity of steel section is being developed by the authors [1]. This probability-based rating procedure will incorporate the inelastic deformation capacity of steel sections, and will utilize the inelastic redistribution of loads due to yielding and three-dimensional behavior. Steel beam and girder bridges are in general highly redundant structures (grillages), capable of substantial load redistribution particularly after the initiation of yielding. Allowing some minor damage to occur under controlled overloads should not impair the ultimate strength or serviceability of these structures.

CURRENT RATING PROCEDURES

Rating of bridges in the U.S. is performed almost exclusively by the AASHTO "Manual for Maintenance Inspection of Bridges", henceforth referred to as "Manual" [2]. While the rating vehicles vary from state to state, the basic analytical procedures and limit states used are almost uniformly those in the AASHTO specifications.

The Manual recognizes two broad rating procedures, Allowable Stress Design (ASD) and Load Factor Design (LFD), and two rating levels, Inventory and Operating. The inventory level may be regarded as the maximum vehicle allowable on the bridge without requiring a special permit, while the operating level is the absolute maximum permissible load to which the bridge may be subjected.

The limit states employed in rating are the same as those used in the current AASHTO design specifications: Serviceability, Overload, and Ultimate. The serviceability limit state in rating is generally governed by fatigue, and is the subject of a separate NCHRP project. The limit states relevant to this research are those of overload and ultimate strength.

The rating factor, RF, for ASD is given by:

$$f_{DL} + (RF)\ f_{LL+I} < f_{S,A} \qquad \text{(Eq. 1)}$$

where f_{DL} is the dead load stress, f_{LL+I} accounts for the live load and impact stresses and $f_{S,A}$ is the allowable stress. A rating factor greater than 1.0 indicates that the bridge can support the vehicle used in the rating process.

The controlling parameter in rating by both the ASD and LFD procedures is the flexural stress in the beam. Typically the allowable stresses $f_{S,A}$ are 0.55 F_y for the inventory level, and 0.75 F_y for the operating level. This implies that even under the operating level the probability of having any yielding in the beam is small.

The rating factor, RF, for LFD is given by:

$$\gamma[M_{DL} + \gamma_L \text{ (RF) } M_{LL+I}] < M_{S,U} \qquad \text{(Eq. 2)}$$

where γ and γ_L are load factors, M_{DL} is the moment due to dead loads, M_{LL+I} accounts for the moments due to impact and live loads, and $M_{S,U}$ is the ultimate strength of the member.

For the inventory level, the usual values are 1.3 for γ and 5/3 for γ_L, resulting in a total live load factor of 2.17. For the operating level the load factor γ_L is unity. The usual $M_{S,U}$ allowed is the plastic capacity of the beam (M_p) when the cross section is compact.

The LFD procedure requires that the RF obtained be checked for serviceability, with γ equal to 1 and the ultimate capacity reduced to $0.8M_y$. Thus it is unlikely that significant yielding of the structure will occur even under the operating load. It is likely, however, that rating by the LFD procedure will give larger capacities than rating by ASD. Both ASD and LFD calculations are based on linear, elastic analysis.

EVALUATION OF RATING METHODS

From the standpoint of rating steel beam and girder bridges, the current AASHTO Manual has several important shortcomings. Foremost amongst them are:

(1) The limit states used for rating are the same as those used in design, implying the same reliability for both processes. This is inconsistent with the use of actual dimensions and properties in the rating process and does not recognize that the bridge has survived many of the uncertain events for which load factors are needed in design.

(2) While emphasizing the importance of an accurate and complete inspection report, no explicit provisions are given to quantify this information in the rating process. For example, information on the road surface condition, the control of truck weights, and the current maintenance program could play a significant part in rating the bridge. While Sect. 5.3.1 of the Manual allows for use of these data, most rating engineers do not utilize it because of lack of more specific guidelines.

(3) Currently used impact factors are probably conservative. The current AASHTO impact factor, based on the length of the bridge, does not recognize the importance of parameters such as the natural frequency of the bridge and the surface conditions.

(4) The distribution factors used by AASHTO are in general very conservative. They are based on girder spacing and do not recognize the influence of the aspect ratio of the bridge, the ratio of longitudinal to transverse stiffness, and the influence of cross-bracing.

(5) No recognition is given to the ability of a redundant structure to redistribute loads as first yield begins to occur. This redistribution is due to the two-dimensional action between bridge elements, plastic force redistribution, unaccounted-for restraints and redundancies, induced automoments from previous overloads, and post-buckling strength of plate elements amongst other factors.

(6) The rating of the bridge is done on an element-by-element basis, with the weakest member controlling the capacity of the structure. Thus the redundancy and reliability of the system is not utilized.

PROPOSED RATING PROCEDURES

Recently Moses and Verma (NCHRP 12-28(1)) have proposed a new rating method based on extensive statistical and probabilistic studies of bridge loadings and resistances [3]. This NCHRP report uses the same procedures for structural analysis, limit states, and truck load configurations as the AASHTO Manual. The general limit state equation is:

$$\gamma_D D_n + \gamma_L \text{ (DF) } L_n \text{ (1+I) (RF)} < \phi R_n$$

where γ_D and γ_L are the dead and live load factors, D_n and L_n are the nominal dead and live load effects, DF is the distribution factor, ϕ is the resistance factor, and R_n is the nominal resistance.

While the method proposed by Moses et. al utilizes many of the current AASHTO provisions, including the distribution factor of S/5.5 for steel beam bridges, it is also a significant departure from the AASHTO Manual in the fact that (1) it places much emphasis on judgemental factors, and (2) suggests quantitative values for these factors based on extensive statistical work.

In the simplest fashion, the main differences between the current AASHTO and the proposed appoach are as follows:

(1) The use of resistance factors (ϕ). For redundant steel members $\phi = 0.95$ for members with no deterioration, $\phi = 0.85$ for members with slight deterioration and some section losses, and $\phi = 0.75$ for significant deterioration and section losses. These ϕ factors are further modified to account for inspection and maintenance work.

(2) The live load factors are based on the volume of truck traffic (ADTT), enforcement and control of overloads. The γ_L factor varies from 1.4 to 1.95 as the ADTT increases and the level of enforcement decreases.

(3) The impact factors are not related to the span length, but depend on the condition of the wearing surface. They vary from 0.1 for surfaces in good condition or with only minor deficiencies, to 0.2 for surfaces with major deficiences, and up to 0.3 for surfaces no longer functioning as designed.

(4) The rating factors obtained can be corrected if the critical condition is given by loading more than one lane or if the distribution factors used are based on more sophisticated

analysis techniques.

(5) The rating factor given is a single number, and the difference between operating and inventory levels is eliminated.

(6) A target reliability of 2.5 for redundant and 3.5 for nonredundant structures is used throughout the study.

(7) The work is calibrated to the AASHTO rating vehicles (Type 3, 3S2, and 3-3) and use of H or HS trucks is explicitedly discouraged.

Table 1, under the heading "Inventory", compares the rating factors for a continuous three-span noncomposite steel girder bridge subjected to a Type 3S2 truck loading. The bridge consisted of W30x280 beams spaced at 6 ft. on center. Each span was 100 ft. in length. The bridge was rated by the current AASHTO procedures as well as by three different cases of the proposed rating method. The case labelled "Best" assumes a low volume roadway, reasonable control of overloads at the source, reasonably vigorous enforcement, wearing surface in good condition or with only minor deterioration, and uncorroded members. The case labelled "Intermediate" assumes moderate truck traffic, limited sources of overloads, occasional enforcement, wearing surface with major deterioration, and slightly corroded members. The case labelled "Worst" assumes that weight limits are difficult to enforce, moderate to high ADTT, many likely sources of overloads, wearing surface not performing as designed, and corroded members. The "Intermediate" case probably best represents what could be called a "typical" bridge.

TABLE 1 - COMPARISON OF RATING METHODS

	AASHTO ASD	AASHTO LFD(1)	AASHTO LFD(2)	NCHRP BEST	NCHRP INTERM	NCHRP WORST
INVENT.	1.34	1.49	1.69	2.79	1.91	1.19
(% ASD)	(100)	(111)	(126)	(208)	(142)	(89)
SHAKED.	----	----	----	3.02	2.05	1.27
(% ASD)	----	----	----	(225)	(153)	(95)
PLASTIC	----	----	----	3.07	2.09	1.41
(% ASD)	----	----	----	(229)	(156)	(30)

(1) LFD governed by serviceability strength
(2) LFD governed by ultimate strength

The rating factors are also shown in parentheses normalized to the current AASHTO ASD rating procedure. The data shows that the current inventory rating procedures are conservative, as only the worst case has a rating factor below that given by AASHTO ADS. Even in that case, however, the rating factor of 1.19 indicates that the bridge is still satisfactory.

The influence of the judgmental factors on the outcome is clearly indicated by the range of rating factors for the three NCHRP cases. It varies from 1.19 to 2.79, a 234 percent difference. This is important because inspection and maintenance procedures are not uniform, and what may seem to one inspector to be a minor problem with the wearing surface may be classified as a non-performing pavement by another inspector. For optimum performance of this procedure uniform inspection criteria need to be developed and followed.

NEW LIMIT STATES

Extension of current rating procedures to the inelastic range requires first the development of new limit states to which this rating will be performed. The common limit states of serviceability, overload, and ultimate will need to be modified if some yielding of the structure is considered acceptable. For the purposes of this discussion the following limit states will be considered:

(a) Serviceability (SR): this limit state is reached before any part of the structure reaches yield, and is typically associated with unacceptable deflections, cracking, fatigue problems, or perceptible vibrations.

(b) Elastic limit (EL): this denotes the load at which yield is first achieved in any member of the structure. This limit state is difficult to calculate precisely because of residual stresses, fabrication inaccuracies, initial out-of-straightness, three-dimensional load distribution, and actual connection restraint.

(c) Shakedown (SD): this denotes the condition that occurs when a load exceeding the elastic limit is imposed upon the structure causing permanent damage. This loading causes a residual moment field to be introduced in the structure. After this load has been applied once, the structure will respond elastically to any successive application of an equal or lesser load. Shakedown is bounded by the following two limit states:

(1) Incremental collapse limit (IC): this limit state will occur under cyclic loadings. Successive applications of the same load cause the damage and deflection to increase, leading eventually to collapse.

(2) Alternating plasticity limit (AP): this limit state is reached when the cyclic loading on the structure causes certain members to yield alternately in tension and compression. After several cycles at this very high stress range the material will fail in low-cycle fatigue.

(f) Static collapse limit (SC): this represents the ultimate load that the structure can carry statically, and can be calculated from plastic analysis and ultimate strength principles.

Our current codes consider the service limit state to vary somewhere between the service range (SR) and the elastic (EL) limit states described above, while the ultimate limit state corresponds to the formation of the first plastic hinge. The overload limit state as currently defined may actually correspond to a shakedown condition (SD); most likely, however, it is just above the elastic limit (EL).

Table 1 shows shakedown (SD) and plastic collapse loads (SC) for the bridge example discussed in the previous section. The same load factors and assumptions as for the inventory case were used for these calculations to insure uniform reliability. These loads were calculated only for the proposed procedure, as plastic collapse and shakedown are not contemplated by the current AASHTO specifications. As can be seen from the numbers, the difference between the shakedown and collapse loads is small, less than 4 percent. This result has been obtained for several other bridges rated by the authors and represents a departure from the conclusions available in the literature.

The difference between the rating factor by the proposed method and the shakedown limit is less than 10 percent, another result verified by several other rating exercises. This indicates that, utilizing the proposed load and resistance factors and a shakedown analysis, an approach to inelastic rating of bridges may be developed. The load calculated in this manner would insure an adequate margin of safety while utilizing the inelastic capacity of the structure.

A major obstacle to the use of such a technique is the unfamiliarity of most engineers with the concept of shakedown and adaptation [5]. For the example discussed above, however, the computations are very simple. They involve the determination of the location of the critical sections from the elastic moment envelope and the postulation of a simple collapse mechanism involving two hinges for an exterior span and three hinges for an interior one. The calculations are then carried out on a span-by-span basis by solving a set of three or four simultaneous equations which provide the critical load and the residual moments. One equation represents the equilibrium of the structure under the residual moments alone, while the remaining equations represent the equilibrium at each critical section under the residual moments and the applied external load.

Two important issues need to be explored before shakedown can be used in inelastic rating. The first is the dependence of shakedown on temperature gradients in the structure. While the plastic collapse load is independent of such gradients, the shakedown load is not. The second issue is a definition of limit states based on an acceptable level of damage. Calculations for strength alone may not be sufficient to insure that an inelastic rating procedure is adequate.

INELASTIC DISTRIBUTION FACTORS

All current design and rating procedures are based on linear elastic analysis. Although other methods can be utilized, this is seldom done unless an unusual or expensive structure is involved. The fixed distribution factors (proportion of total longitudinal moment due to a single line of wheels carried by a single girder) used by AASHTO cannot account for differences in the geometry of the bridge nor the differences in torsional stiffness of common composite bridges.

The distribution factor, D, is computed as D=S/K, where S is the spacing of the girders in ft. and K is a constant. AASHTO gives a fixed value of K=5.5. Using elastic analysis, other studies [4-6] have shown the AASHTO value of K to be quite conservative particularly for long, narrow bridges. Using elastic orthotropic plate analysis, Sanders [7] has calculated ranges of K from 4.42 to 7.52.

The AASHTO approach can be modified for the inelastic range as suggested by Heins [6] by computing an inelastic distribution factor, $(DF)_p$, as a function of γ, where γ is the ratio of the number of girders to the number of lanes:

$$(DF)_p = 3.45 - 1.809\gamma + 0.315\gamma^2$$

In contrast to the elastic case, the inelastic distribution factors correspond to values of K from 5.90 to 9.45. Heins developed the equation using elasto-plastic analysis of grillages; consequently, inelastic distribution factors may be developed for particular classes of bridges. For most common ranges of composite bridges, the corresponding inelastic value of K is approximately 7.0. Such inelastic distribution factors can be employed to develop even more accurate rating procedures.

SUMMARY

The final product of this project will be a rating manual similar to the current AASHTO document [2], but reflecting more accurately the differences between designing new bridges and rating existing ones. In its simplest form it will use a shakedown analysis combined with inelastic distribution factors to determine more realistic rating factors for bridges.

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INVESTIGATION OF BRIDGE DECK MOVEMENTS IN THE NEW ATCHAFALAYA RIVER BRIDGE

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SYNOPSIS

Poor performance of bridge-deck joint sealing systems is a continual, expensive highway maintenance problem. The problems stem from a failure to properly assess short- and long-term joint movementsor other factors. The work reported in this paper is part of an overall effort to develop rational design methodologies for joints in modern highway bridges. This paper presents: (a) details of the instrumentation designed and installed to periodically monitor the short- and long-term movements of the new Atchafalaya River Bridge; and (b) the measured movements at selected bridge deck joints.

INTRODUCTION

Highway bridges generally require either expansion or contraction joints between sections of the deck or between the deck and the approach roadway. Several types of proprietary joint seals have been developed and used for highway bridges. These can be broadly classified as either compression seals, membrane seals or cushion seals.

Frequently the joint-sealing systems have not functioned as intended. The seals have either ruptured; pulled out or fallen out of their proper positions; squeezed out of position; leaked at their splice points; or the anchor bolts securing the seals have loosened or failed. In short, the joint seals have proved a continual and expensive maintenance problem for highway departments, and a nuisance to the highway user. Since various states have reported contradictory behavior in specific joint-sealing systems, the problems may not be inherent with the systems. Rather, the problems may stem from improper design criteria for the bridge joint, improper installation practices, differences in bridge type used in one state from another, differences in environmental conditions, or other factors.

An inspection of several recently constructed brides in Louisiana disclosed numerous problems in all of the joint-sealing systems used for these bridges. The current practice for the design of expansion joint seals for most state highway bridges is based on elementary formulas [1] of strength of materials, and these may not realistically predict actual joint movements in modern bridges.

Trends in highway bridge construction--such as the use of precast, prestressed concrete girders, and creation of multiple spans continuous for live loads--complicate the prediction of deck joint movements. Systematic, detailed studies of joint movements will lead to the development of rational design methodologies for joints in modern highway bridges including criteria for eliminating joints entirely in some cases. This paper deals with the initial phases of a project effort undertaken by the authors to systematically study the joint movements in the newly constructed Atchafalaya River Bridge in Louisiana. This paper presents: (a) details of the instrumentation designed and installed to periodically monitor the short- and long-term movements of the bridge; and (b) the measured movements at selected bridge deck joints.

DESCRIPTION OF INSTRUMENTATION

Three types of instrumentation were utilized on the bridge: Optical, LVDT's and thermocouples. Since the objective of this research was to evaluate the longitudinal movements that might affect expansion joint design, the instrumentation was designed to provide experimental evaluation of this behavior.

Bridge Layout

The bridge selected for instrumentation is the Atchafalaya River crossing at Krotz Springs, Louisiana. This bridge is typical of bridge construction in the southeastern United States. The bridge consists of approach spans (9 on the east side and 10 on the west side) and the river crossing. The original bridge was built in the 1930's and needed replacement due to deterioration of timber piling and the narrow lanes. A second bridge was constructed in 1976 so that each bridge carried two lanes of traffic from the four lane divided highway. Traffic was diverted to the newer bridge while the older bridge was removed and replace by the new one. All instrumentation was placed on the east approach spans. This approach consisted of eight spans of prestressed concrete girders and one plate girder span. Including the abutment and the beginning of the river crossing, five expansion joints were placed in this part of the system. The river crossing was a steel thru-truss and was not instrumented. The west approach was similar to the east approach and was not instrumented.

Optical Measurements

The optical measurements were taken using a total station theodolite surveying instrument. Reflectors were placed at each support as shown in Fig. 1. These reflectors were placed during construction so that an initial data base could be established. Readings were taken on a regular basis as soon as the roadway slab pouring was initiated. The pattern chosen for the reflectors allows for vertical, longitudinal, and rotational movements of all joints and piers.

LVDT Measurements

The disadvantage of the optical reading was that the time required to obtain a complete set of readings was 3 to 4 hours. Considerable temperature changes usually occur over this period of time resulting in movements during the data gathering process. The advantage of these measurements was that they could be taken during the construction process so that data could be obtained immediately after the slab was poured. The LVDT measurements can be taken almost instantaneously. These gages were placed after the completion of construction. The measurements to be obtained were similar to that of the optical at each expansion joint. Fig. 2 shows a typical layout of the LVDT's at an expansion joint. The layout was designed to provide longitudinal expansion and rotation at each joint.

Thermocouple Placements

Since the longitudinal movements of bridges are sensitive to temperature, it is essential to obtain temperatures of the materials at the time of other measurements. The thermocouple placement is shown in Fig. 3. For the roadway slab, the thermocouples were placed near the top, center and bottom of the slab at the time of pour. Thermocouples were placed at the top, center and bottom of the prestressed concrete girders on the exterior after these girders had cured. These thermocouples were covered with a hydraulic cement after bonding with epoxy.

Electronic Monitoring

A total of 36 LVDT's and 27 thermocouples have been installed on the east approach. These gages are connected to a Hewlett Packard 9000 series microcomputer and data acquisition system. A user oriented software program was developed for the project. The program is capable of taking a complete set of readings within one second. A set of readings can be triggered on either a fixed time interval or on a specified change of a data point or combination of data points. In addition, an on-line plot of any two variables can be obtained. Results of the optical readings during the first nine months of this project and the LVDT readings obtained after the completion of construction will be reported in this paper.

MEASURED JOINT MOVEMENTS

As indicated in the preceding sections, theodolite measurements of the movements of the ends of the girders on either side of the expansion joints were made at regular intervals soon after the girders were installed on the piers. These measurements provide a complete record of the movements in the joint during and after the concrete bridge deck was cast and curved. The movements of joint 1 together with the temperatures at the time of measurement are presented in Fig. 4 for a five month period. This figure indicates that, with minor exception, the bridge girders on either side of the joints tended to move in a similar fashion, i.e., either elongate or shorten. The influence of the temperature on the joint movements is difficult to establish from these readings because of the significant shrinkage occurring in the structural system during this period. A complete analysis of the measured movements will be made shortly after evaluating data from the creep and shrinkage cylinder tests.

CONCLUSIONS

The three types of instrumentation utilized to monitor the movements in the concrete deck of the bridge structure appear to be adequate to provide the necessary data for a rigorous evaluation of the factors affecting the bridge deck movements. The theodolite measurement of the joint movements taken over a period of nine months show that thermal movements over 50°F temperature changes are on the order of 0.2 inches at the concrete joints and 0.5 inches at the steel joints. Creep and shrinkage movements have not as yet been determined since the experimental data available for evaluation of the creep and shrinkage effects in the bridge deck on the joint movements has not yet been analyzed. The installation of the LVDT's permit a complete evaluation of the bridge deck movements particularly the effect of temperature on the response of the bridge structure.

ACKNOWLEDGEMENTS

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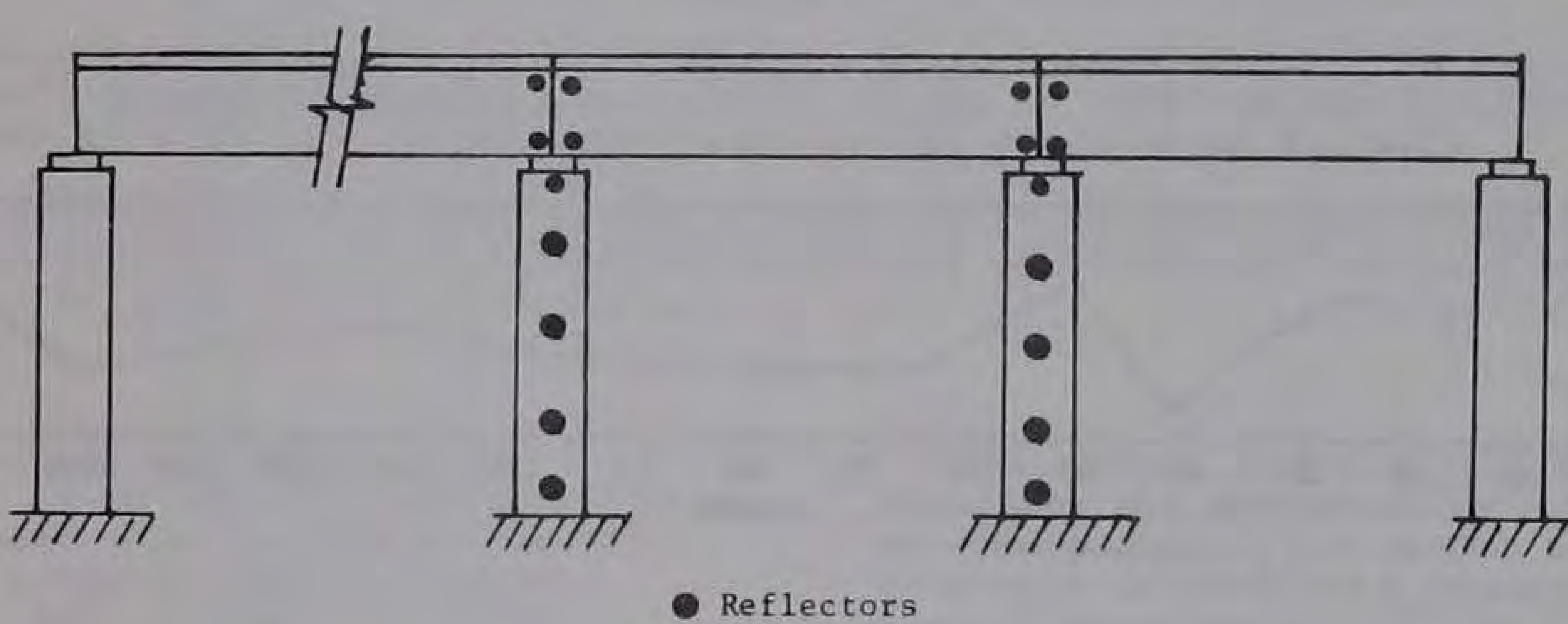

Figure 1. Evaluation Showing Typical Optical Measuring Points

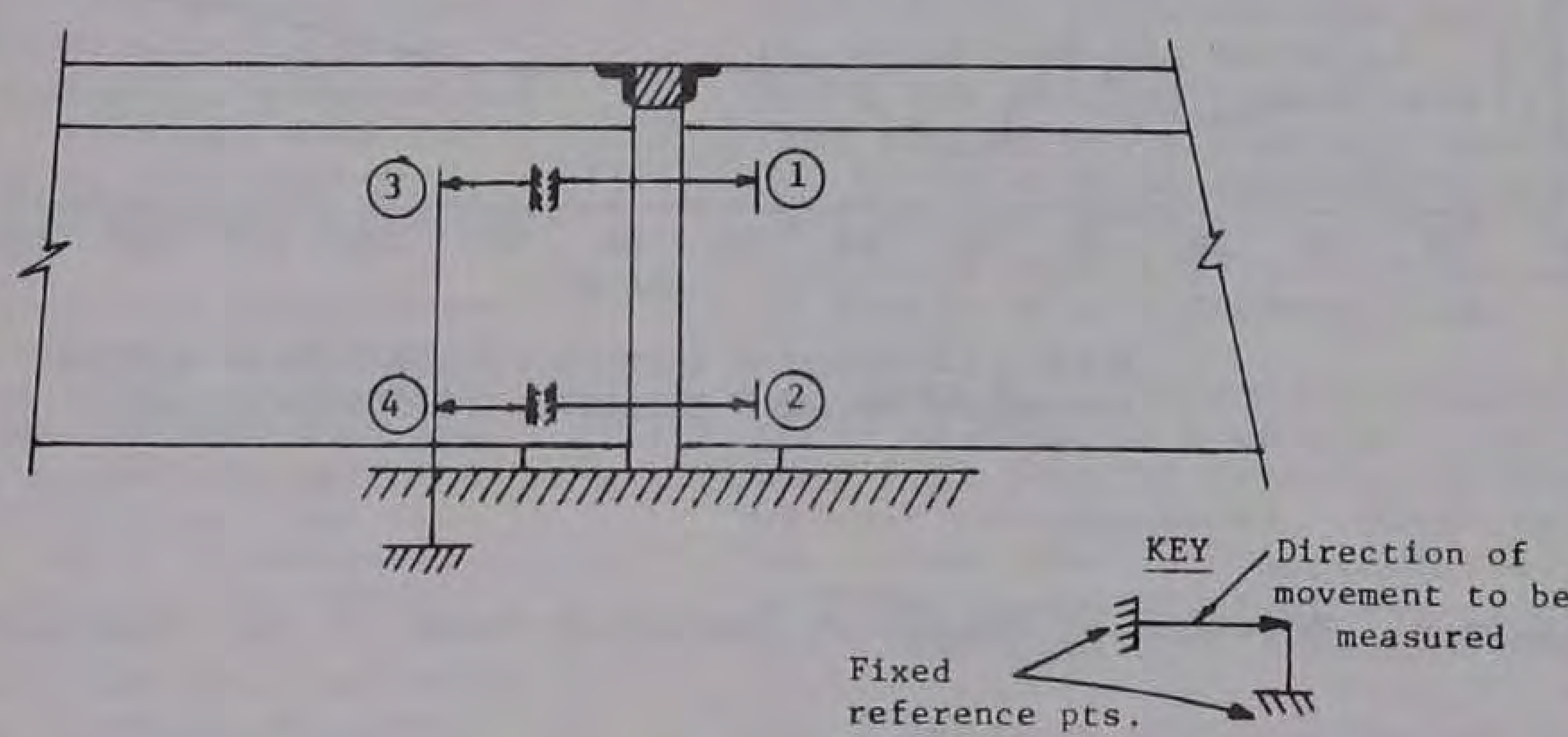

Figure 2. LVDT Array Package for Typical Joint

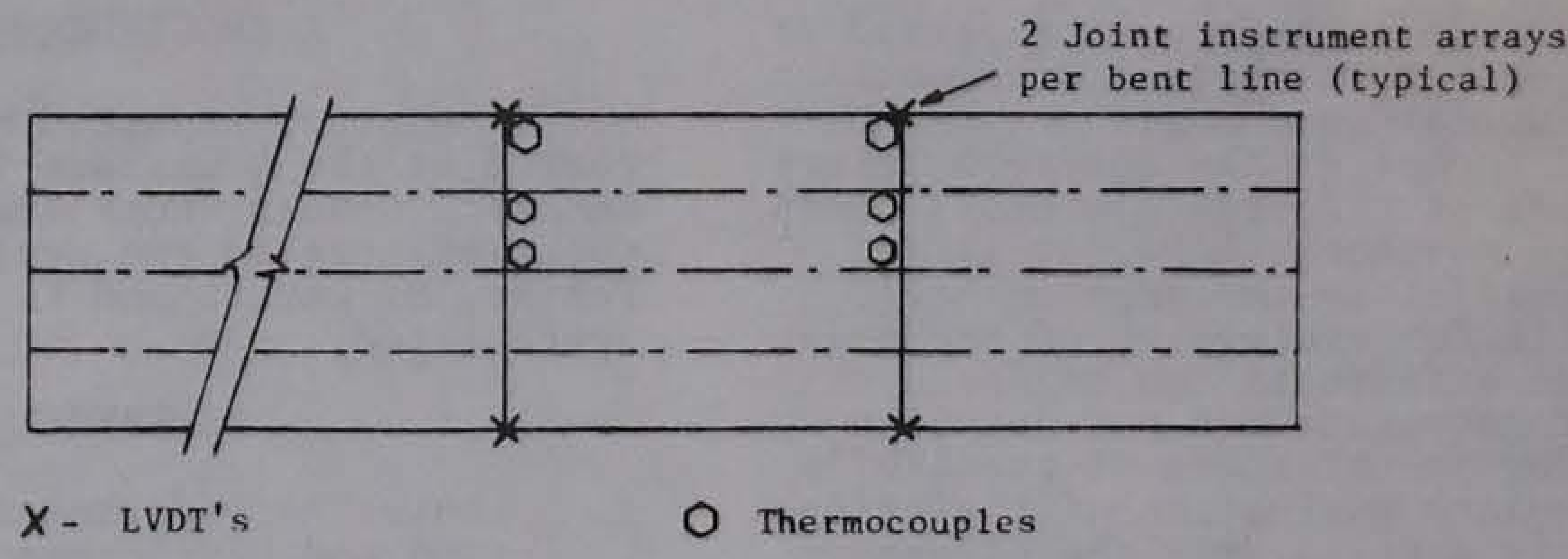

Figure 3. Layout of Joint Instrumentation Arrays at a Bent Line and Location of Thermocouples

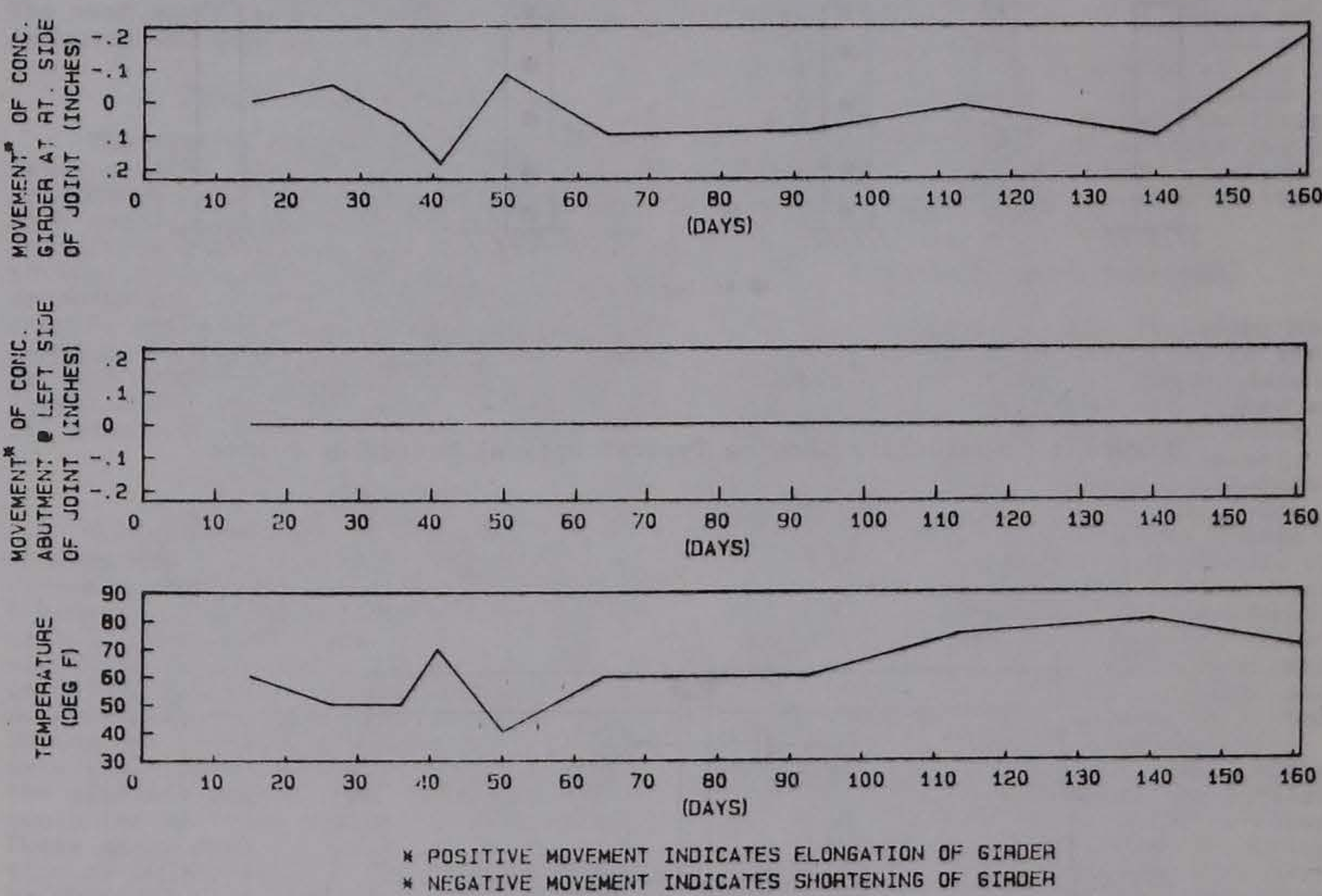

Figure 4. Movement of Grider at Expansion Joint #1 vs. Time and Temperature

KANSAS BRIDGE STILL SURVIVES SINK HOLE

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SYNOPSIS

A sink hole expanding under Interstate 70 in Russell County, Kansas, has subjected a Reinforced Concrete Box Girder bridge to rotation and a differential settlement of 2.91 feet between abutments during a period of 24 years.

This structure continues to carry traffic with no significant signs of distress, showing the flexibility and durability of reinforced concrete when subjected to very slow deformations.

During the construction of Interstate 70 in the mid 1960's, areas of settlement were noticed in the oil fields in a region covering several counties. One of these areas was adjacent to the Interstate and to a local road over the Interstate. This particular area of settlement or sinkhole is located near the small town of Gorham in Russell County, Kansas, and has induced unforeseen distortions into the bridge over the Interstate.

Investigation has revealed that a salt bed varying in thickness from two hundred to four hundred feet underlies the area approximately 1300 feet below the surface. It is being slowly dissolved and carried away by water flowing down an improperly plugged oil well hole and into the void left in oil bearing strata by oil removal. This well has been abandoned since 1957.

The bridge under observation is a Reinforced Concrete Box Girder with spans of 51'-68'-68'-51'. Single Column Piers resting on Spread Footings and Pile Bent Abutments are founded on a weathered shale formation. See the attached sketches for a cross-section and an elevation view of the bridge.

This structure has been subjected to settlement and rotation as shown in the included graphs with little or no visible distress. Hairline transverse cracks appear in the hubguard over each pier, but this is a commonly found crack on structures of this type in Kansas.

The record of the settlements and rotations are presented here in the form of five graphs. The graphs begin with plan data or with data shortly after the time of construction in the Summer of 1965 and continue through January of 1988.

Graph No. 1 shows the original profile grade, several later profile grades, the current profile grade, and a cross-sectional view at each abutment indicating the current amount of rotation.

Graph No. 2 shows the longitudinal centerline elevations at each substructure unit of the structure for each time the structure was surveyed. Abutment No. 1 has settled approximately 2.32 feet at the centerline and Abutment No. 2 has settled approximately 5.23 feet at the centerline. This graph indicates that the rate of settlement is stable at the present time.

Graph No. 3 shows the relative settlements in feet in each span. There has been 2.91 feet of relative settlement between the abutments. Note that Pier No. 1 has gone down approximately 7" in relation to Abutment No. 1; Pier No. 2 has gone down approximately 11 1/2" in relation to Pier No. 1; Pier No. 3 has gone down approximately 10 1/2" in relation to Pier No. 2; and that Abutment No. 2 has gone down almost 6" in relation to Pier No. 3.

Graph No. 4 shows the torsional

rotation as a per cent slope change of the structure at each substructure unit. This is based on gutterline elevations and calculated by the rotation of the top slab.

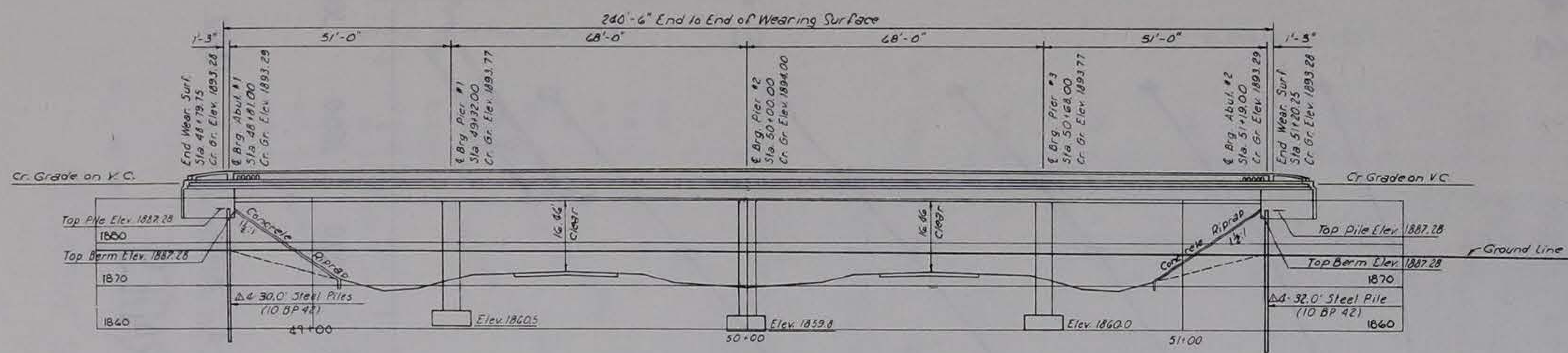
Graph No. 5 shows the relative rotation between the various substructure units. This graph and the previous one would tend to indicate that the rate of rotation is stable at the present time.

Attempts to analyze this structure for the stresses caused by the settlement and rotation have not been successful since they are time related. Had these distortions taken place rapidly, instead of over two decades, the structure would surely have failed in any number of ways and would have been removed or replaced. The apparent reason that the structure shows no outward signs of significant distress is that the concrete is undergoing creep.

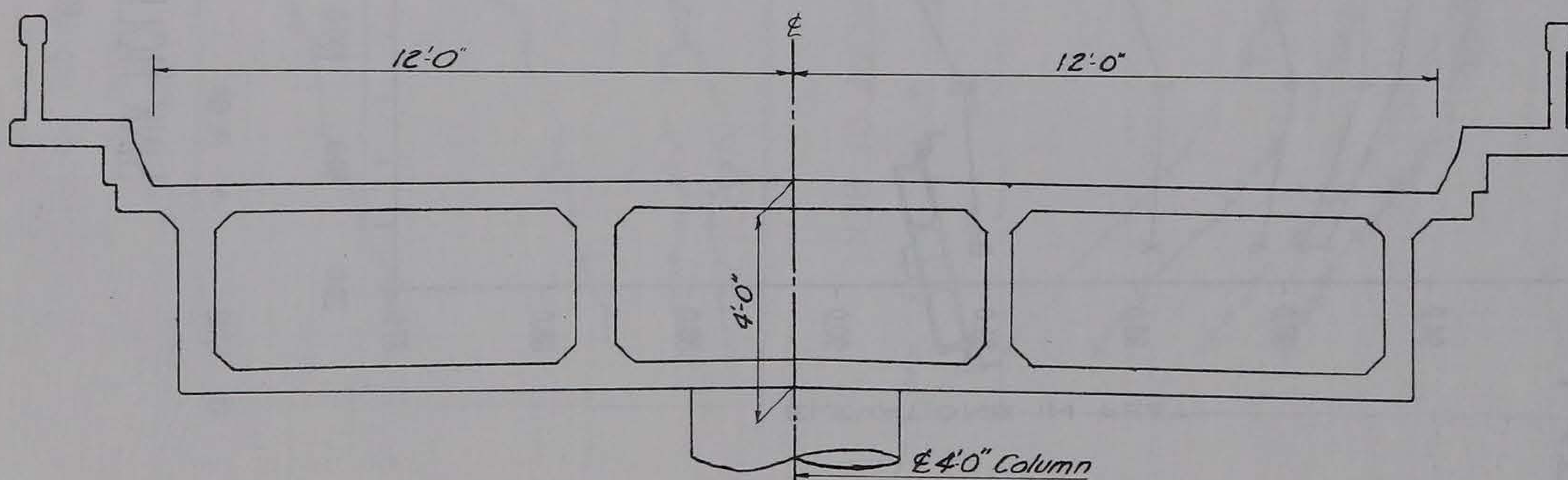
Despite the settlement this structure has been subjected to, it continues to carry normal traffic. This attests to the flexibility and durability of reinforced concrete when subjected to very slow deformations.

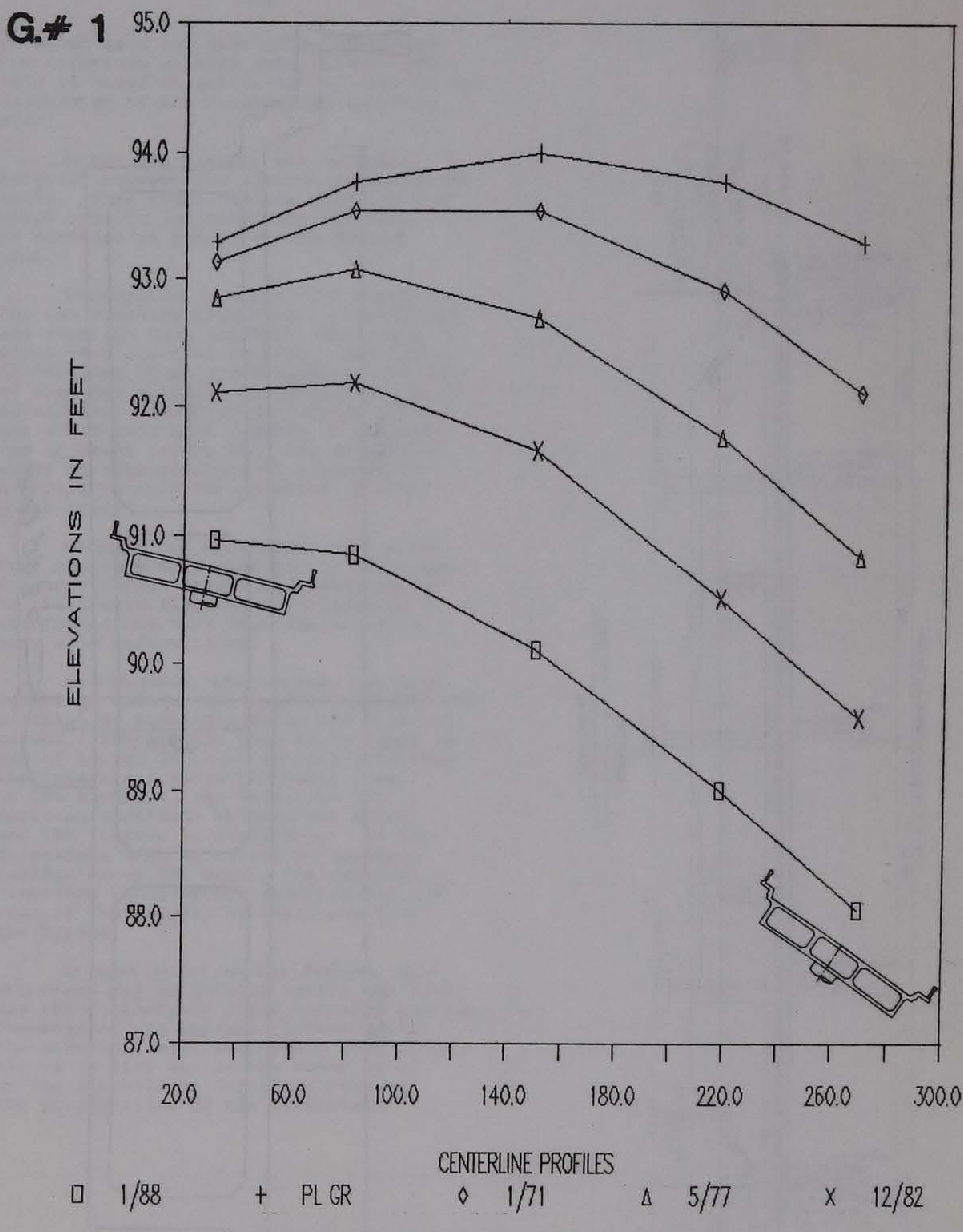
In the past, the roadway has been raised in some areas to alleviate problems of changing sight-distances and vertical curves. The problem soon to be faced is one of having to raise the grade to keep the Interstate above the water level in the sinkhole. At this time the vertical clearance between the bridge and the roadway is acceptable. If the Interstate must be raised to maintain traffic above the water, the vertical clearance could become unacceptable and require the removal or replacement of the bridge.

At some point in the future, this structure may be totally within the sink and the distortions could actually reverse themselves. It appears, however, that the structure will not remain long enough for us to find out as the water level in the sinkhole is currently nearing the shoulderline of the Interstate.



ELEVATION
51'-2@68'-51' Cont. R.C. Box Girder Spans
Pile Bent Abutments, Pedestal Type Piers
24'-0" Roadway

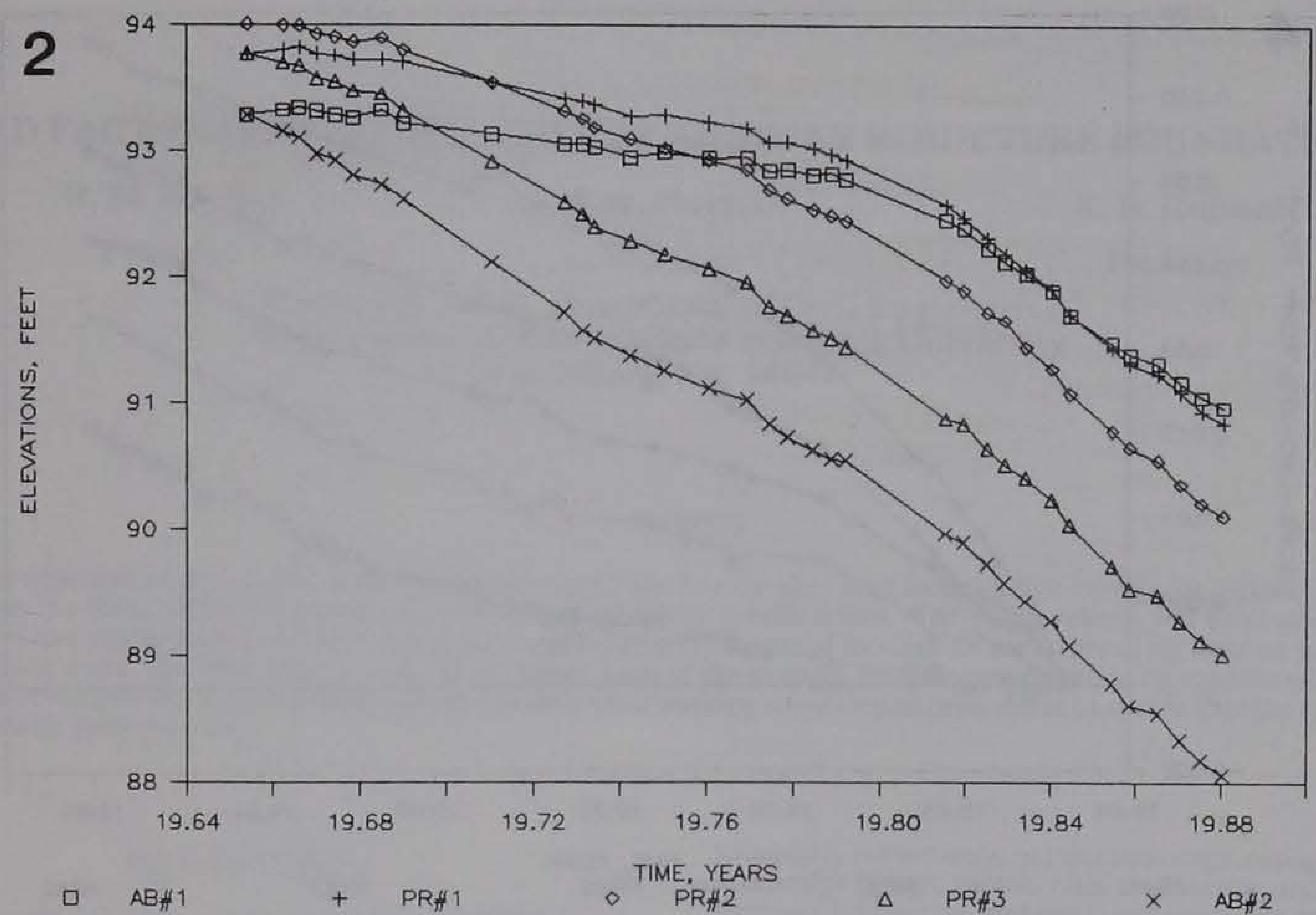




SINK HOLE PROFILE GRADES

PLAN GRADE AT TOP

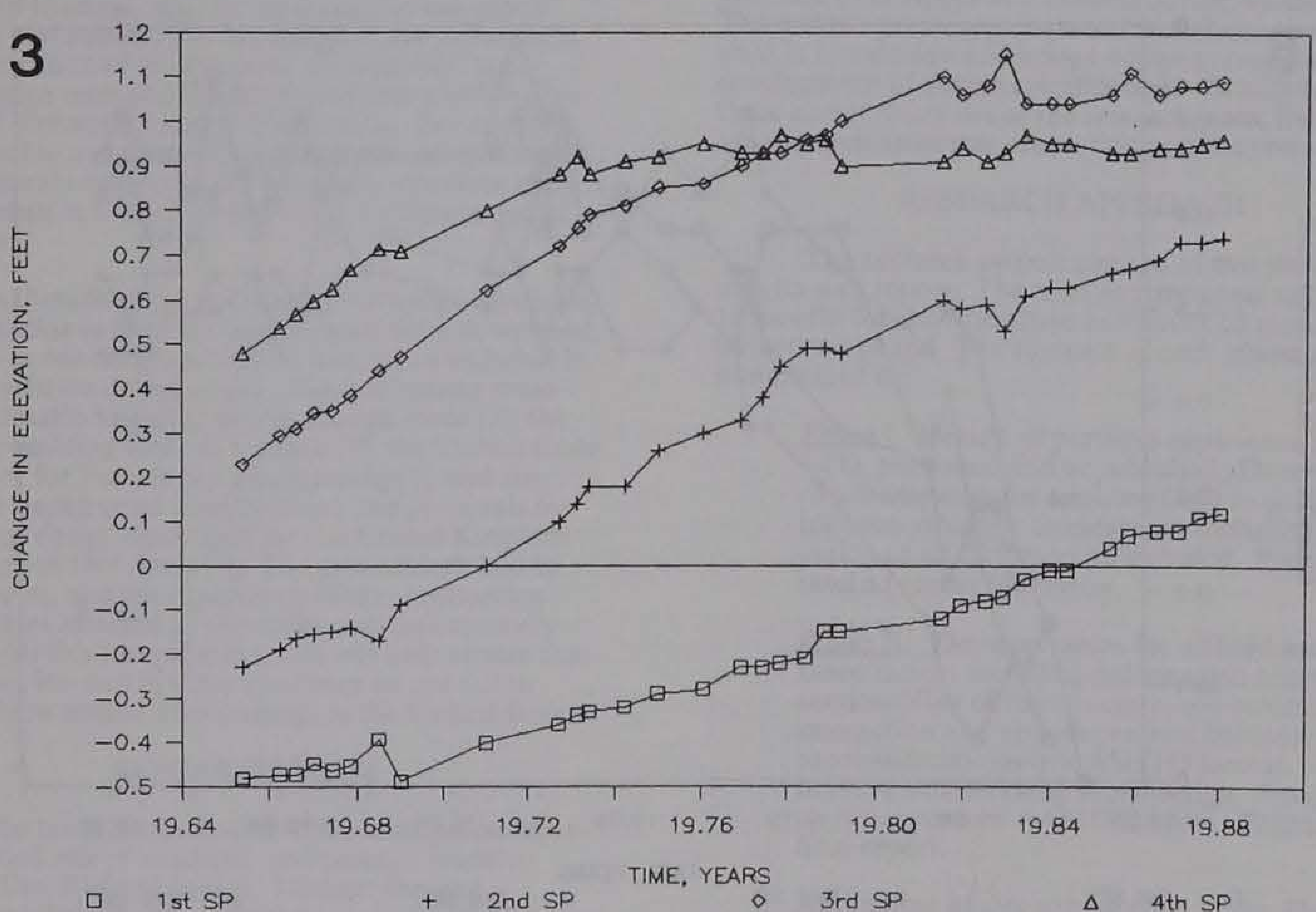
G.# 2



SETTLEMENTS AT CENTERLINE OF BENT

1965 TO 1988

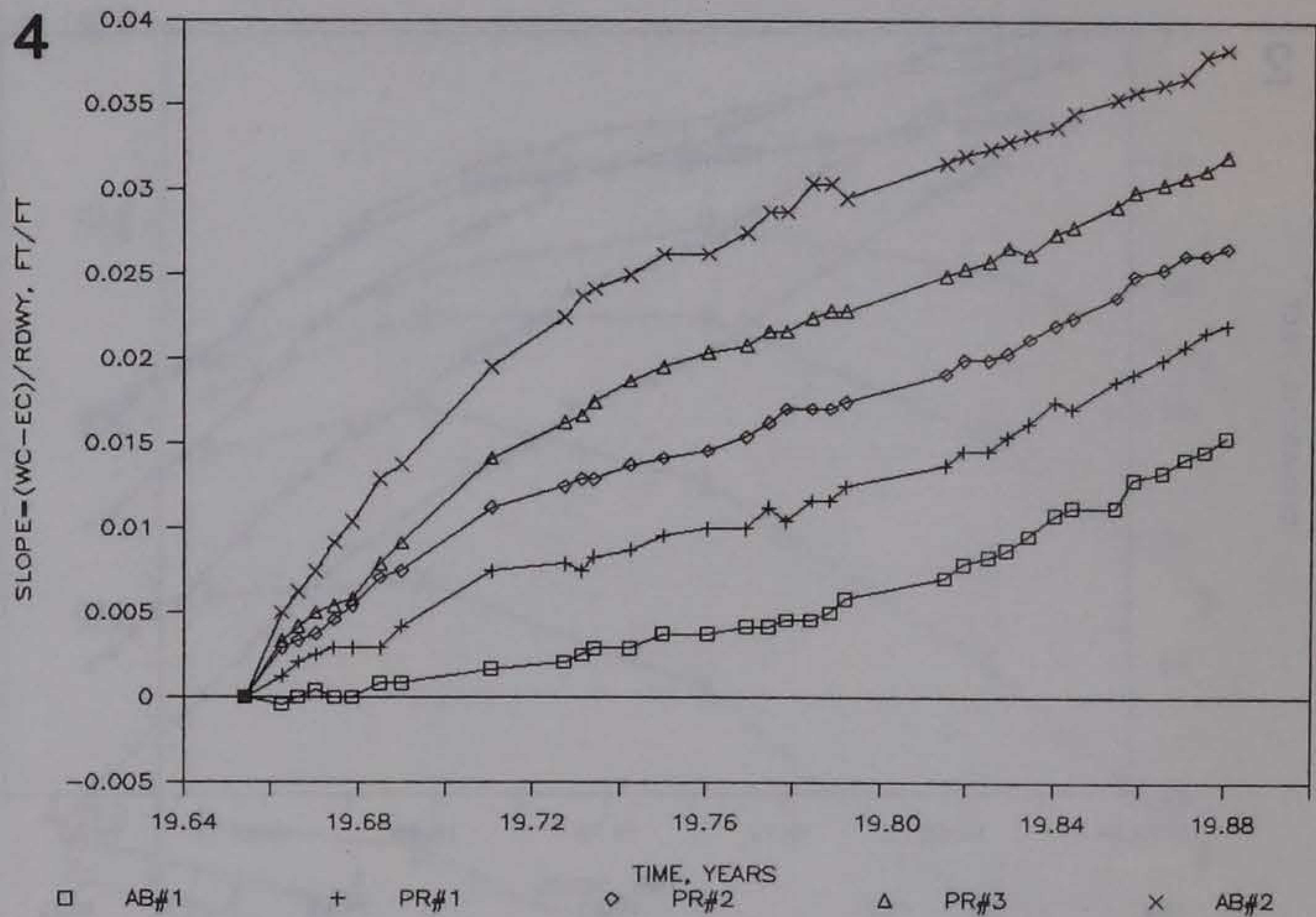
G.# 3



RELATIVE SETTLEMENTS @ CL OF BENT

1965 TO 1988

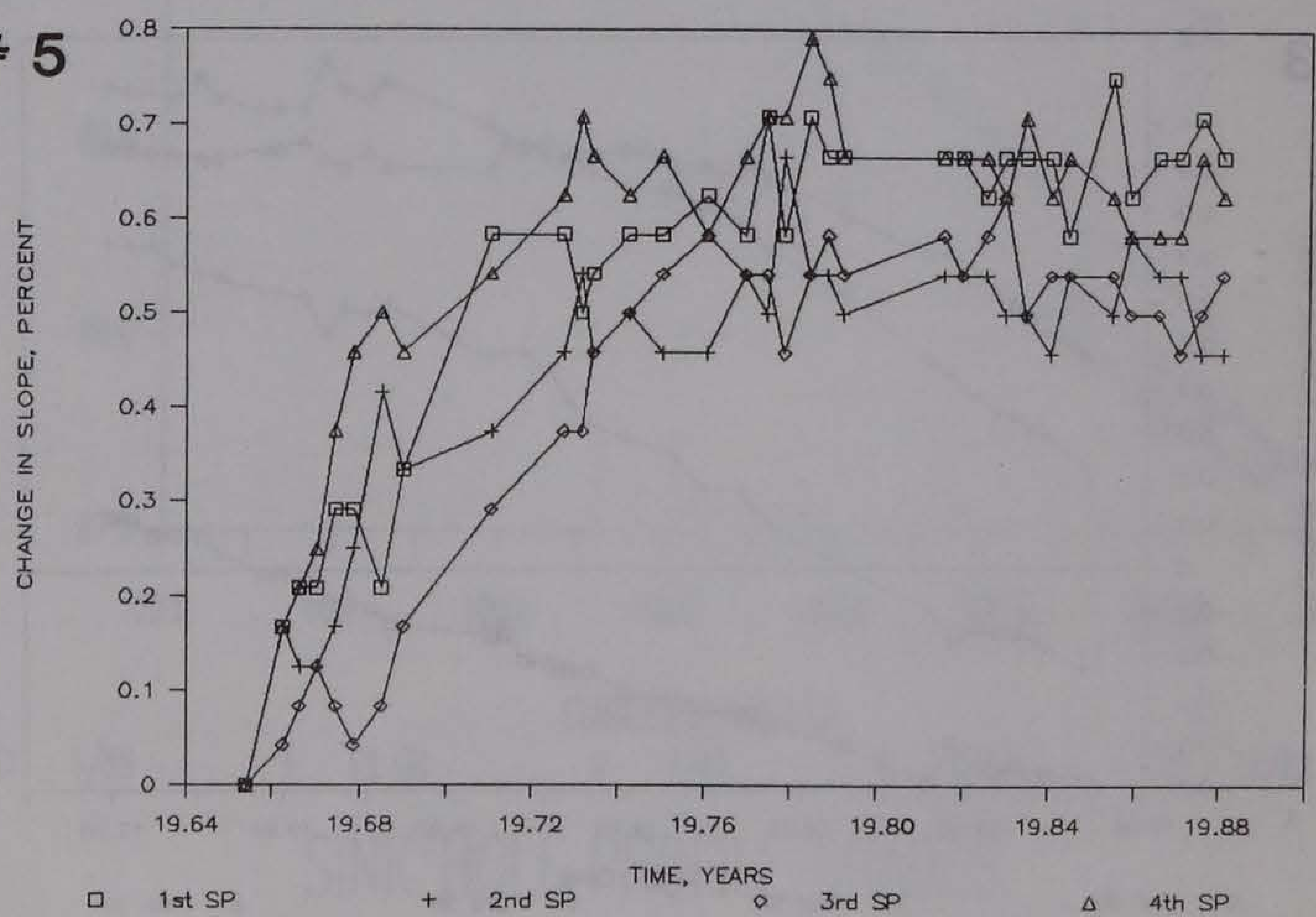
G.# 4



ROTATION AT BENT

1965 TO 1988

G.# 5



RELATIVE ROTATIONS

1965 TO 1988

LOAD FACTOR DESIGN CRITERIA FOR HIGHWAY STRUCTURE FOUNDATIONS

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SYNOPSIS

The objective of the research described in this summary is to develop load factor design criteria for highway structure foundations in a form consistent with the current AASHTO bridge specifications. The design criteria will be developed for drilled piles and shafts, driven friction piles, driven end-bearing piles, spread footings on rock, spread footings on soil, and rigid retaining walls. Both the loading and the resistance sides of the strength design equation are to be considered. The procedure developed will be calibrated against currently used working stress design procedures to ensure that the results are consistent with good practice.

INTRODUCTION

Load factor design was adopted by AASHTO in the mid-1970's as an approved method of superstructure design. Currently there are no provisions for load factor design of bridge foundations. Engineers who use load factor design for bridge superstructures must develop two sets of loadings, one for the design of the superstructure and another for the design of the foundation. This duplication of effort can be eliminated if load factor design methods are developed and approved for design of highway structure foundations. Further, this would lead to a more uniform margin of safety for all the structural components in a highway structure and should result in a more consistent and efficient use of materials.

In Canada and a number of European countries, studies similar to this one have already been completed, and load factor design principles have been included in their foundation design codes. Notable among these are the Ontario Highway Bridge Design Code [3], the National Building Code of Canada [2], the Danish Code of Practice for Foundation Engineering [1], and the extensive background investigations and proposals for load factor design developed for the United Kingdom by the firm of Ove Arup [4]. The groundwork laid by these studies, and the experience of the engineering communities affected by the codes, will be extremely valuable for this investigation, and will help ensure that the lessons learned in other countries do not fail to benefit these similar undertakings in the United States.

RESEARCH TEAM

The team assembled for this investigation represents a good mix of academic and practice oriented people. Drs. Richard Barker, Michael Duncan, and Kamal Rojiani will serve as co-investigators for the project, which is administered through Virginia Polytechnic Institute and State University (Virginia Tech) in Blacksburg, Virginia. The investigators specialize, respectively, in structural engineering and bridge design; geotechnical engineering and soil-structure interaction problems; and structural analysis and structural reliability. Working with the investigators will be Mr. James Langer and Mr. Martin Huntzinger of Gannett Fleming Geotechnical Engineers in Harrisburg, Pennsylvania and Dr. James Withiam of D'Appolonia in Monroeville, Pennsylvania. These three practicing engineers have extensive experience in foundation and bridge design practice and in development of design specifications. In addition to these senior members of the research team, four graduate research assistants are working on the project.

RESEARCH APPROACH

The research project consists of two phases, each with its own report. The time requirements are about 12 months for the first phase and about 18 months for the second phase. The contents of each phase can be summarized as:

Phase I. Review of previous experience with LFD, published and unpublished. Development of a framework for applying LFD to design of highway structure foundations, including identification of all factors to be included. Prepare an interim report for review.

Phase II. Develop values for all load and resistance factors including deformation criteria, serviceability of the structure, soil-structure interaction and time-dependent behavior. Put recommendations in AASHTO format. Calibrate against working stress design. Study affects on other parts of AASHTO specs. Prepare a final report.

At the time of this writing (July 1988), Phase I is nearly complete and work has begun on the interim report which is expected to be submitted in September 1988. In the paragraphs that follow, progress on the tasks in the first phase are reviewed and comments made on how the results of the second phase are to be integrated with other ongoing research.

Review of Previous Experience with LFD

A thorough search of the literature has been conducted and a questionnaire has been sent to the state DOT's to solicit their opinions and experience with LFD. Responses from the questionnaire have been received from 44 of the 53 organizations included in the first mailing. The replies to two questions that required only a yes or no answer were (1) Does your office use LFD methods for design of highway structures? and (2) When LFD methods have been developed for highway structure foundations, do you think they will be used by your office? The "yes" response was 80% for the first question and 70% for the second. Perhaps there isn't as much resistance to LFD concepts as was originally thought. If the education and information process continues, it appears that a majority of the people are willing to change.

Discussions were held at VPI with other LFD researchers who visited the campus. Al Dimillio of FHWA and Felix Yokel of NBS shared results of their work on shallow bridge foundations. Malcolm Bolton of Cambridge University discussed a code for retaining wall abutments on spread footings he is writing for the UK Department of Transportation. Members of the research team also interacted with a number of presenters of papers at the ASCE Joint Specialty Conference on Probabilistic Methods hosted by VPI.

Selection of a Framework for LFD Concept

Dr. Rojiani has written an internal position paper entitled, "Probabilistic Framework for the Determination of Load and Resistance Factors for the New LFD Code for Highway Structure Foundations." In the paper, Dr. Rojiani presents some typical values for target reliability indices and a method for determining load and resistance factors consistent with the implied safety level. Taking the position that the current AASHTO load factors will also be used for foundations, this leaves only the resistance factor to be determined. The issue then becomes whether to use a single resistance factor for a particular limit state or to use partial resistance factors applied to the individual soil strength properties such as cohesion and angle of internal friction.

A procedure has been formulated for a reliability analysis of spread footings and driven piles. This has resulted in a computer program that can determine the reliability index and corresponding resistance factors when given the mean and coefficient of variation of the soil properties and applied load. At the present time this is being done for a specific soil bearing capacity equation and a specific pile load capacity equation. A companion study involves the evaluation of the reliability of different equations to predict reasonable capacities. For example, five different pile capacity equations are being evaluated by comparing their predictions with 98 different pile load tests to determine which equations are the most reliable. With this procedure formulated, the effect of changes in the reliability index on the resistance factors can be studied. The general trend is known: increase the reliability index and the resistance factors decrease. To determine how sensitive they are to change, a parametric study is being conducted.

A similar study on estimating settlements of shallow foundations on sands and gravels is being conducted by Dr. Duncan and a graduate student. They have records from 120 cases where field measurements of settlement were made and are conducting a statistical analysis of the relationship between predicted and measured settlement for different methods. From this study, the two "best" methods for predicting settlements will be chosen.

Development of an Outline of LFD Criteria

Dr. Duncan has prepared an internal report that presents the objectives and guiding philosophy of developing an LFD code. It includes an article by article analysis of the current AASHTO code and identifies those articles that can remain as is and those that need changes, additions, or deletions. The annotated code provides an organizational outline for development of the new LFD code. The outline of the new LFD code is now set up on a word processor with headings and text indicating the topics to be included. As material is developed, differences in corresponding sections of the new and old codes can be clearly shown and reproduced for further discussion.

INTEGRATION WITH LRFD RESEARCH EFFORTS

The results of Phase II of this project can be adapted to a LRFD format without too much difficulty. The main change will be in the calibration of load and resistance factors to a safety index based on theories of probability and statistics rather than to a global factor of safety based on current working stress design procedures. The data base for conducting a statistical analysis of variables such as soil strengths and pile capacities may not be as large as desired, but there appears to be in the literature a number of field investigations with suitable data for a statistical analysis. The confidence level of this data, however, remains to be evaluated.

ACKNOWLEDGEMENT

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PLASTIC HINGE DETAILS FOR THE BASES OF ARCHITECTURALLY OVERSIZED BRIDGE COLUMNS

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SYNOPSIS

An experimental investigation is being conducted to examine the seismic behavior of bridge columns with moment-reducing hinge details. Column specimens incorporating modified hinge details proposed by designers in Washington and California are being subjected to cyclic lateral loading and then evaluated with respect to energy absorbing characteristics. Preliminary results indicate that columns with the moment-reducing details undergo less strength degradation and exhibit greater ductility than do similar unmodified columns.

INTRODUCTION

Bridge foundations in seismic regions are designed to withstand the plastic hinge moments that develop at the bases of the bridge columns. In columns that are oversized for architectural reasons, this approach results in excessively large foundations. Various hinge details have been proposed, principally by bridge engineers in Washington and California, to reduce the plastic moments in the columns and, thereby, reduce the sizes of the foundations. This paper presents an overview and some preliminary results of an investigation into the plastic hinging behavior of columns into which these moment-reducing details have been placed.

BACKGROUND

Current AASHTO [1] design requirements for bridges in seismic regions are based upon the guidelines contained in the Advanced Technology Council Report ATC-6 [2]. A key feature of the design approach recommended in ATC-6 is that damage caused by seismic loads should occur in the columns of bridges, not in the foundations. A consequence of the current design approach is that the foundation must be designed to be at least as strong as the columns that it supports. Hence, a column that is oversized for architectural reasons also results in a foundation that is oversized. As the cost of the foundation can represent a significant portion of the total cost of a bridge, designers have suggested procedures for reducing the plastic column moments and, thus, also for reducing the sizes of the foundations.

The basic concept inherent in the modified details is to provide a reduced moment capacity in the plastic hinging region at the bases of the columns. Two of the modified hinge details studied in this project are shown in Fig. 1 (identified in the figure as detail A and detail B). A layer of easily compressed material is located at the base of the column providing partial discontinuity between the column and the footing. The discontinuity results in a smaller effective cross-section at the column base and, therefore, a reduced hinge capacity in the column.

To a great extent, the modifications that have been suggested are based on engineering judgement, and the behavior and safety of the modified details have not been fully established. Sufficient lateral reinforcement must be provided to confine the concrete core within the hinge, to prevent lateral buckling of the longitudinal reinforcement and to provide adequate shear reinforcement. However, the amount and distribution of lateral reinforcement necessary to insure satisfactory hinge formation and the way in which the modifications affect the hinge behavior are still matters of uncertainty. A particular question with regard to these details involves the vertical length of detail necessary to develop an acceptable plastic hinge.

EXPERIMENTAL PROGRAM

Reinforced concrete scale models of bridge columns are being tested with hinge details which have been used in both California and Washington. The column specimens are subjected to cycled static lateral loads which result from displacements of up to 14 times the yield displacement. The test setup for the specimens is shown in Fig. 2. The study includes an evaluation of the hinge details for adequate hinge behavior and a comparison of the ultimate moment capacities, displacement ductilities, plastic hinge lengths, and failure modes associated with the specific hinge details. Experimental tests are being performed on two different sizes of specimens: small-scale specimens of approximately 1/20-scale and moderate-scale specimens of approximately 1/6-scale.

The small-scale tests provide a cost efficient parametric study and will also aid in the selection of the parameters for the larger-scale tests. While size effects in the small-scale study preclude direct application of the numerical results obtained from the small-scale tests, the study does provide valuable information about the relative performance and failure modes associated with the various details. Parameters being evaluated in the small-scale study include:

- specific hinge detail behavior;
- vertical detail length;
- column aspect ratio;
- column shape;
- magnitude of axial load;
- amount of both longitudinal and spiral reinforcement;
- thickness of the discontinuity joint;
- effects of number of load cycles at each displacement level; and
- fatigue characteristics.

Approximately 40 small-scale specimens are being tested.

The larger 1/6-scale tests will result in a more realistic representation of the hinge behavior in actual bridge columns, and size effects are reduced when compared to the small-scale tests. However, the cost and effort required to conduct these tests is much greater, and, therefore, only six larger-scale specimens are being tested.

PRELIMINARY FINDINGS

The parametric study performed on the small-scale specimens is nearly completed. Selected preliminary findings of the small-scale study are discussed below.

To serve as a basis for evaluating the hinge performance of columns with the modified details, a circular column with the same dimensions and reinforcement as the inner hinge of the modified details was tested (detail C of Fig. 1). A plot of the load-displacement history for the test is shown in Fig. 3. During the cycle bringing the column to a displacement of 12 times the yield displacement, the spiral reinforcement fractured (see Fig. 3) leading to a rapid decrease in strength and energy absorbing capability in the column. This behavior is typical of that of other tests on bridge columns with spiral reinforcement which are reported in the literature [3].

Tests were performed on columns incorporating the details A and B. Even at displacement levels of 14 times the yield displacement, no evidence of the spiral reinforcement breaking was observed. The columns exhibited little degradation in strength, and the plastic hinges continued to absorb energy. The greater ductility and the absence of fracture of the spiral reinforcement in the columns with the modified details may be due to additional confining affects provided around the hinge regions by the architectural columns.

To evaluate the relative performance of details A and B, specimens incorporating the two modified details were cycled to a displacement level of 10 times the yield displacement and then subjected to 10 load cycles at this displacement level. The load-displacement history for the specimen incorporating detail B is shown in Fig. 4. After the second load cycle at 10 times the yield displacement, very little degradation in the load-displacement loops occurred. The hinge continued to exhibit nearly plastic behavior through the remaining cycles. The load-displacement history for the specimen incorporating detail A is shown in Fig. 5. In contrast to the behavior observed in the specimen with detail B, degradation of the loops occurred and the energy absorbed dropped with every cycle at a displacement level of 10 times the yield displacement.

After testing, each of the small-scale specimens was sectioned with a diamond lapidary saw. Cross-sections of the specimens incorporating details A and B are given in Fig. 6 showing the internal crack patterns associated with each detail. For the specimen with detail A, there is evidence of longitudinal bar buckling and of significant deterioration of the concrete within the hinge. For the specimen with detail B, no evidence of buckling of the longitudinal bars was observed and, with the exception of the region at the base of the hinge, no deterioration of the concrete occurred. The differences in behavior observed in the columns incorporating the two details may be because detail A allows for some lateral movement in the hinge region. Thus, less confining effects from the architectural column is provided with detail A than with detail B.

SUMMARY

Preliminary observations from the small-scale tests indicate that the energy absorbing characteristics of columns incorporating the modified hinge details appear to be better than those of unmodified columns. This improved hinge behavior may be due to the confinement provided around the hinge detail by the outer portion of the architectural column. Work is continuing on the larger-scale specimens to verify the findings which were obtained from the small-scale tests.

ACKNOWLEDGEMENTS

The project reported herein is being conducted under Agreement No. GC8287 by the Washington State Transportation Center (TRAC) at Washington State University under sponsorship of the Washington State Department of Transportation.

REFERENCES

1. Standard Specifications for Highway Bridges, adopted by the American Association of State Highway and Transportation Officials, 13 th. ed., 1983.

2. "ATC-6, Seismic Design Guidelines for Highway Bridges", Applied Technology Council, Washington, D.C., 1981.

3. Stone, W.C. and Cheok, G.S., "Inelastic Behavior of Full-Scale Bridge Columns Subjected to Cyclic Loading", NBSIR 88-3788, National Bureau of Standards, Gaithersburg, Maryland, May 1988, 252 pp.

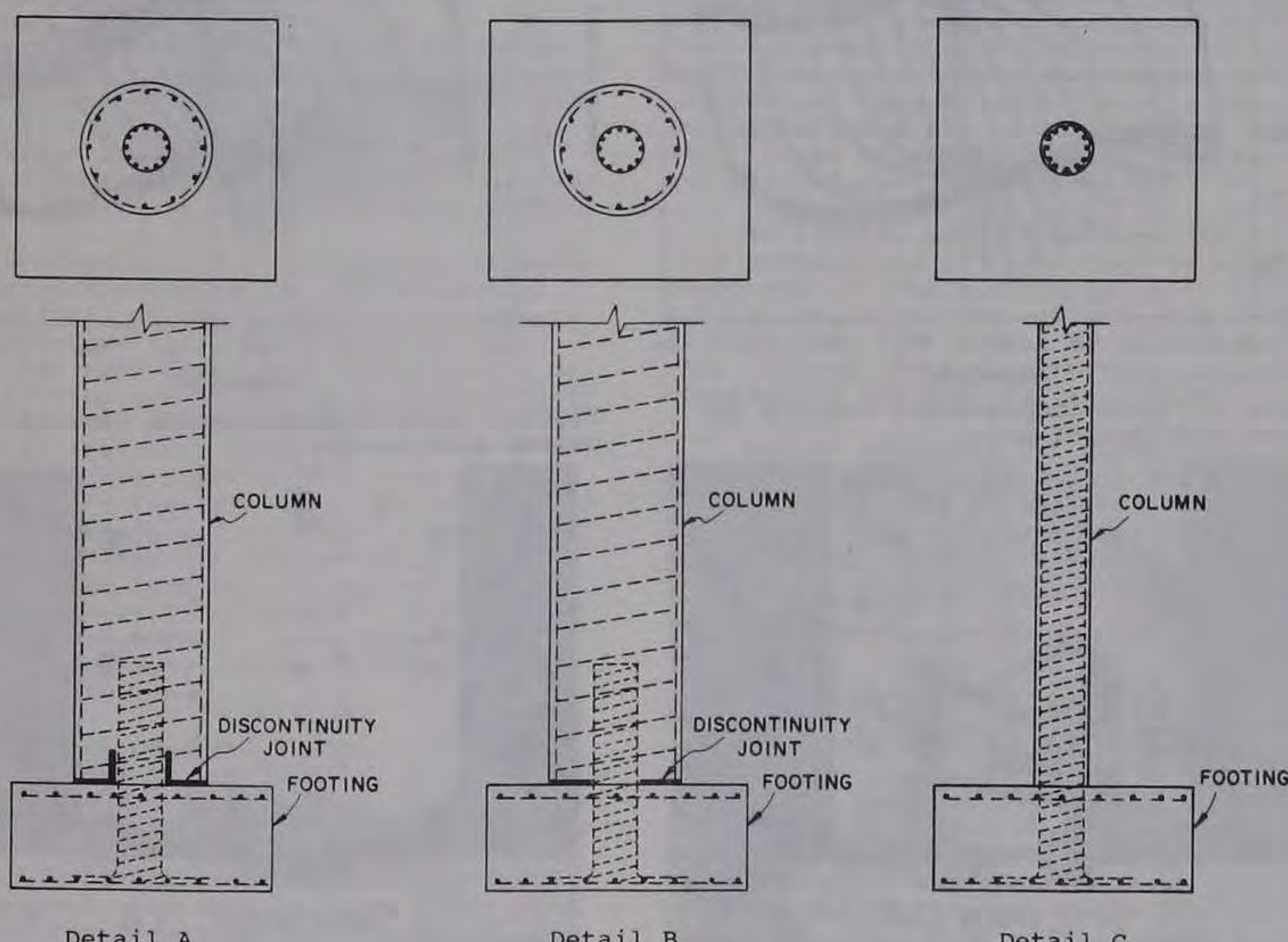


Figure 1 - Hinge details studied in this project.

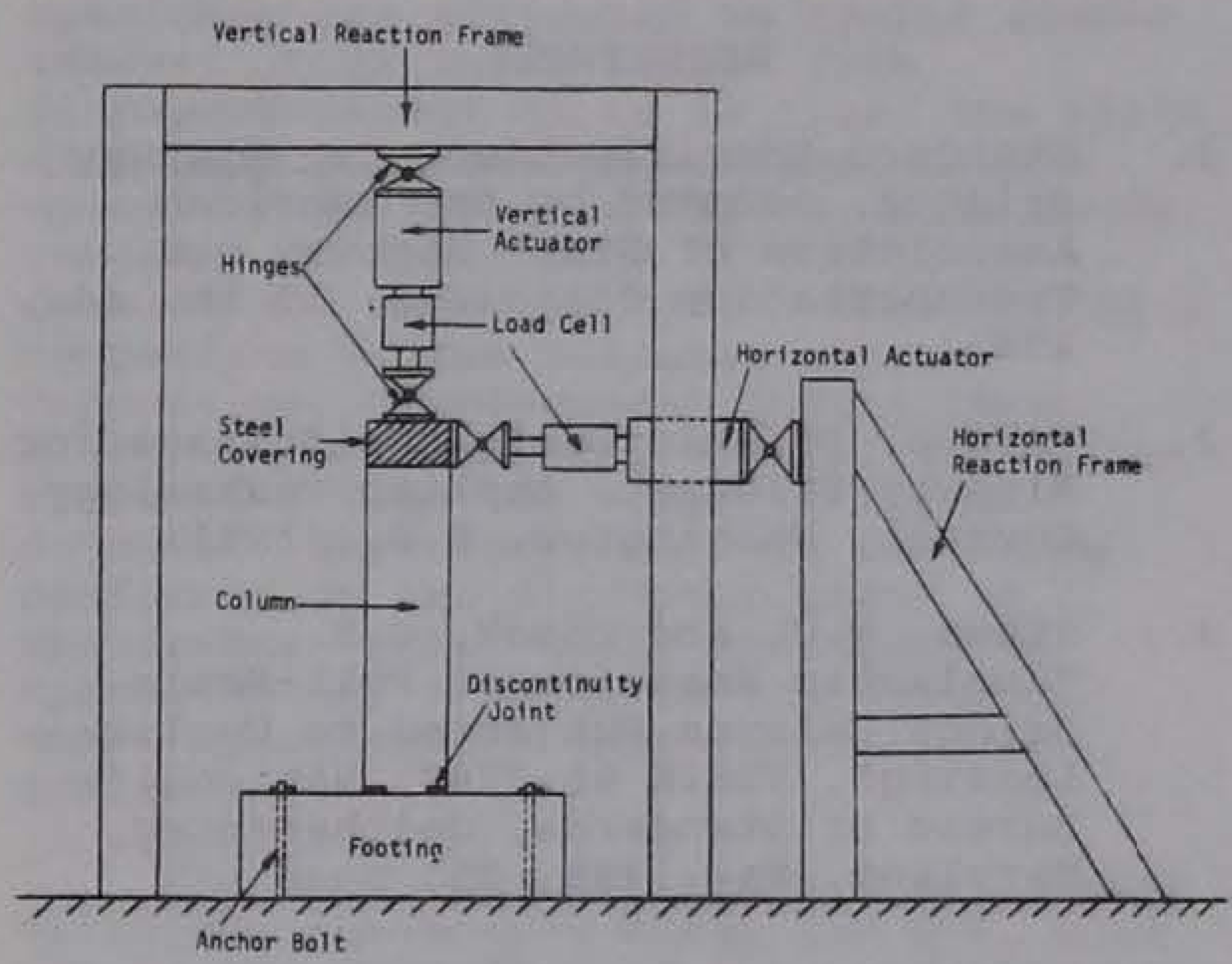


Figure 2 - Experimental test setup.

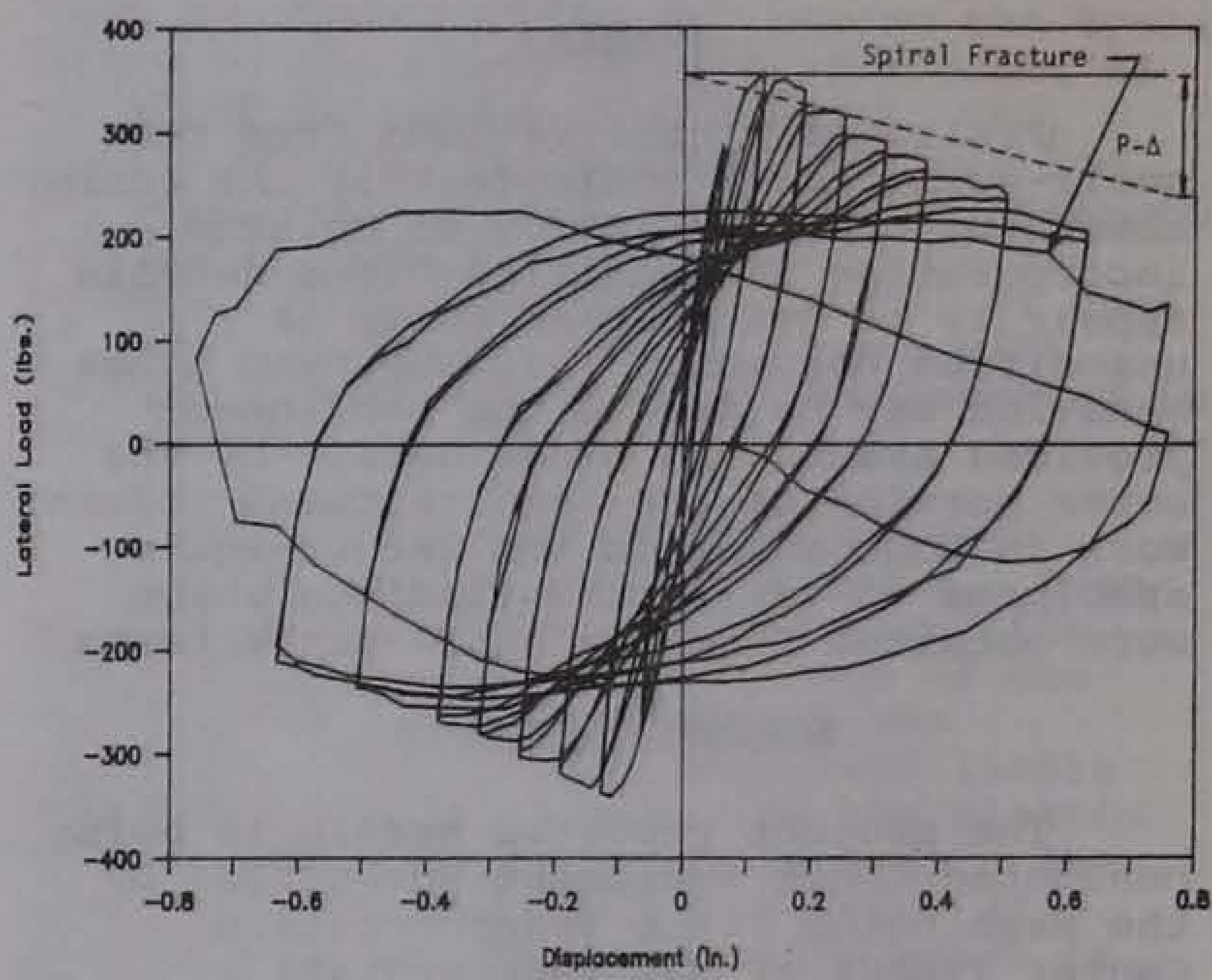


Figure 3 - Load-displacement history for circular column with no modified detail.

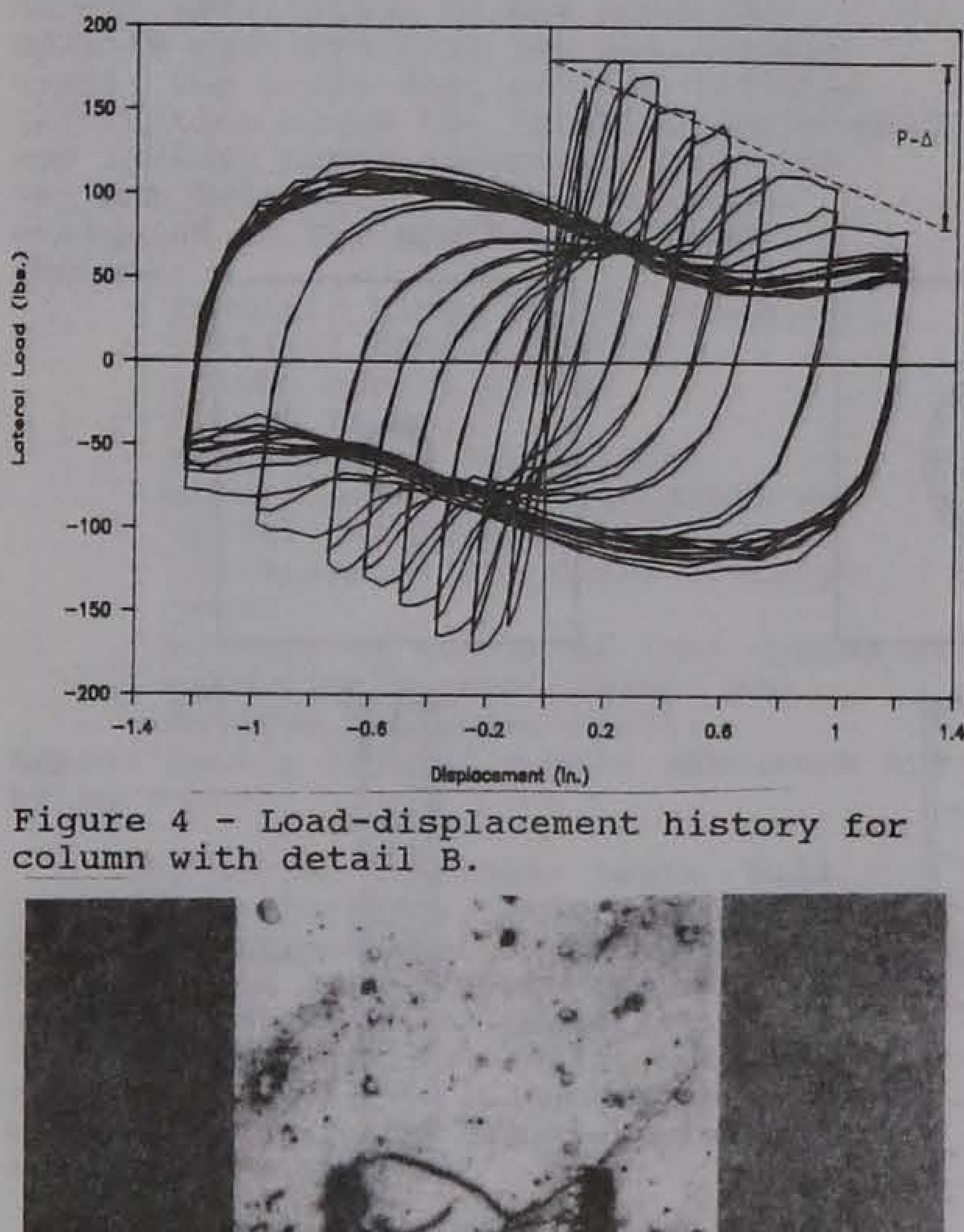


Figure 4 - Load-displacement history for column with detail B.

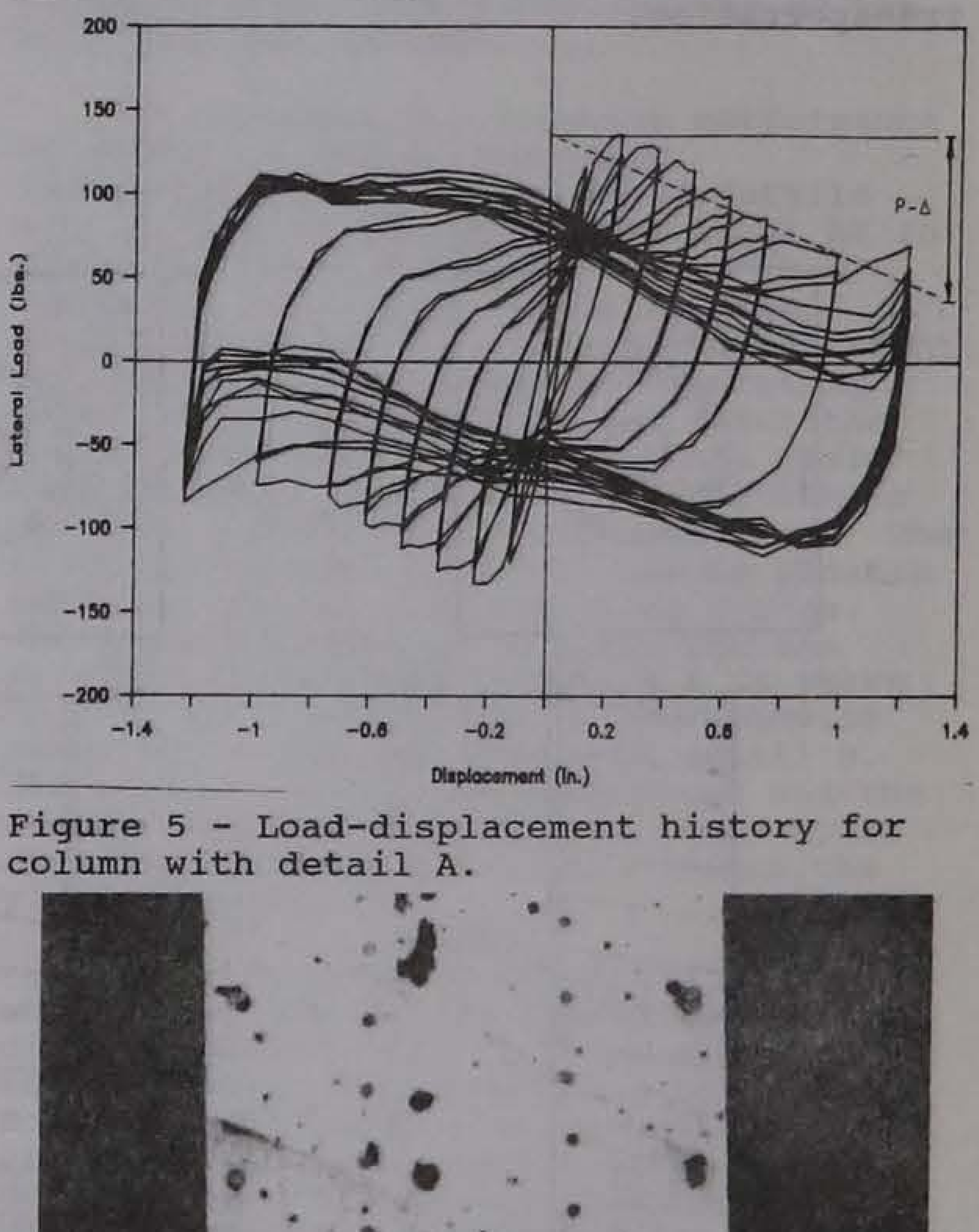
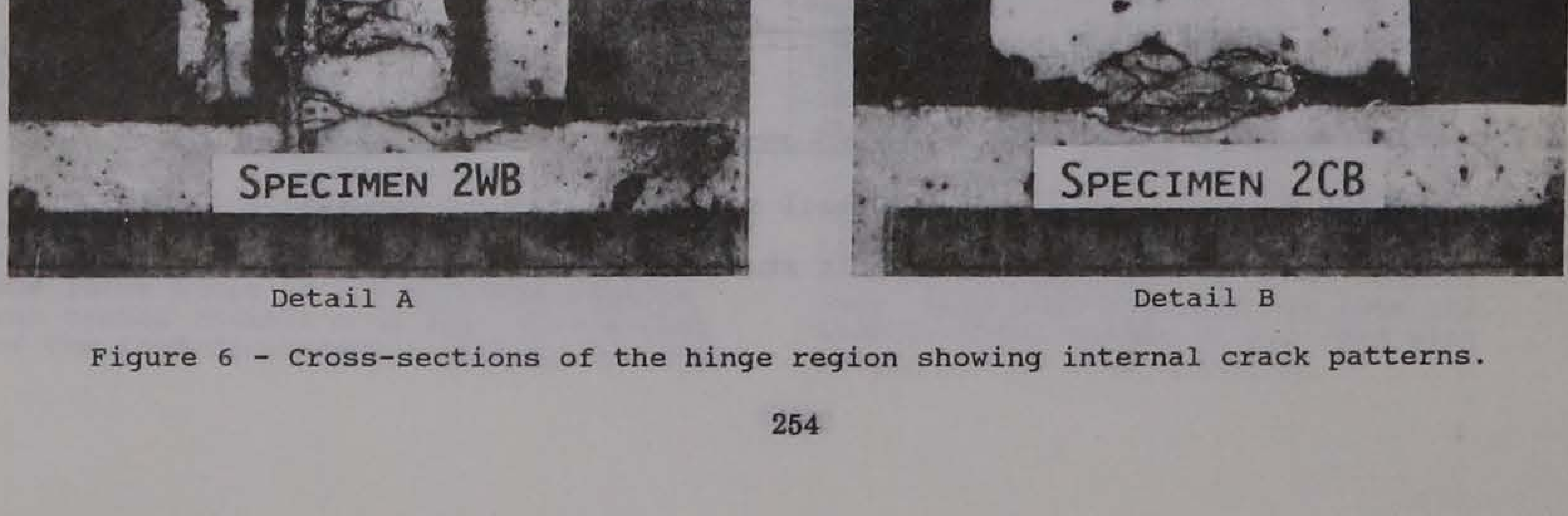


Figure 5 - Load-displacement history for column with detail A.



Detail A

Detail B

Figure 6 - Cross-sections of the hinge region showing internal crack patterns.

PREVENTING SUPPORT UPLIFT ON SKEW STEEL BRIDGES

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SYNOPSIS

Simply supported skew bridges with pronounced obliqueness have the tendency of uplifting their outermost supports. For concrete bridges, this phenomenon can be prevented by intentionally increasing the tributary dead load over the threatened supports. However, a similar procedure is not recommendable for skew steel bridges since their maintenance could be jeopardized. Usually, when pronounced obliquenesses are required for steel bridges, the design leads to wider piers and abutments, so that to facilitate a reduction of the steel structure obliqueness. Nevertheless, all these contrivances can be avoided by creating a flexibility of some of the cross-frames or floor beams of the stucture. A such procedure is herein described, analyzed, and illustrated by a numerical example.

INTRODUCTION

Skew bridges present numerous and,at the same time, costly inconveniences: unequal loading of the main girders or trusses; difficult corner details; uncertain assumptions for the structural analysis; longer substructures; and, mostly, the tendency of uplifting their outermost supports.

Numerous are also the methods of reducing or even avoiding the obliqueness of skew bridges. As for example: enlarging of substructures; using normal cantilevered seats over skew piers [1]; interrupting or cantilevering one of the main girders over the abutments; et al. However, when the technical ingenuity does not succeed in avoiding the obliqueness of a skew steel bridge superstructure, then the tendency of uplifting its outermost supports remains a governing factor for the design of such bridges. This tendency is aggravated when the obliqueness is pronounced, i.e. less than 60°; and when the dead load of the superstructure is too small in order to exceed the uplift forces.

For skew concrete bridges, the uplift phenomenon can be prevented by intentionally increasing the tributary dead load over the threatened supports. A similar procedure is not recommendable for skew steel bridges since their maintenance could be jeopardized. However, for such bridges, their dead load can contribute to the prevention of support uplifting if special precautions are performed while the erection of the superstructure. As for example: the tightening of the cross-frames or of the floor beams after the completed main girders or trusses are burdening only on their final supports; or leveling the superstructure so that the reactions on its outermost supports be deliberately increased. But the uplift tendency of skew steel bridges is mostly aggravated when the cross-frames or the floor beams of the bridge, as well as its end floor beams, are too stiff.

As for the end floor beams, by following some precautions [2], they can be eliminated, improving in this way the behavior of the bridge. While for the cross-frames or current floor beams, they cannot be eliminated, but only relaxed. Dealing with this relaxation constitutes the goal of this paper.

Before analyzing the effectiveness of the cross-frame relaxation on skew steel bridges,there will be analyzed the behavior of such bridges when they have very stiff cross-frames, as well as when they have very supple cross-frames.

ASSUMPTIONS AND NOTATIONS

In all the aforementioned analyses there are considered simply supported antisymmetrical steel skew bridges having two main girders or trusses, each of span L, spaced at a distance of b apart, and each containing skew parts of length a. Each main girder is considered as loaded by a concentrated load P representing a half of the total live load subjecting the bridge, and located on its center line (Fig.1).

The outermost supports of the superstructure are marked with 0; the inner supports and the firsts cross-frames as well, with 1; the subsequent cross-frames, with

2, 3, etc..; the middle cross-frame, with p; the front main girder, with f; and the rear main girder, with r. The numerical dimensions shown in Fig.1 refer to the numerical example developed at the end of this paper.

The distance between the supports 1, projected on the bridge length, is:

$$\ell = L - a \tag{1}$$

and it constitutes an important parameter in the subsequent analyses.

Assuming the entire live load located on the centerline of the bridge is a conservative assumption which simplifies the analysis and gives an easy way for the preliminary design of skew steel bridges.

VERY STIFF CROSS-FRAMES

Let the superstructure represented in Fig.1 be equipped with very stiff cross-frames. Assume that the reactions on the two outermost supports 0 due to the dead load of this superstructure does not exceed the corresponding uplift forces due to the live load. In this case, the deflections of the two portions 0-P of the main girders will have the tendency to follow the deflections of the two parallel and more rigid portions 1 - P of these girders, leading to the uplift of the two supports 0.

This phenomenon is aggravated when the cross-frames are too stiff and when the deflection of the main girders are too pronounced.

Each of the two main girders will behave as being loaded, besides the live load P, by two groups of forces $X_1 + X_2 + ... = \Sigma X_i$, of opposite directions, representing the actions and the reactions due to the cross-frames which assist the two main girders of reciprocally leaning when they lose their supports 0. Fig.2 represents the front main girder, having to carry on its left side the actions of the cross-frames, and, on its right side, the reactions of the cross-frames.

A mirror image of Fig.2 would represent the loading of the rear girder, since as said before, the bridge is antisymmetric. Each of the two main girders should satisfy the conditions of equilibrium and, consequently, if the phenomenon of uplift occurs, then the reaction on the support 1 should have the same magnitude as P; and the couple $P\ell/2$ should be equal to that of the groups of forces ΣX_i:

$$P \ell / 2 = c \, \Sigma X_i \tag{2}$$

where c is the lever arm of the couple X_i.

It follows that, when the cross-frames are able to carry on each side of the bridge centerline, the forces:

$$\Sigma X_i = P \ell / 2 c \tag{3}$$

then the uplift phenomenon will occur.

In this case, the analysis will be made by considering the two main girders as a whole and the forces ΣX_i as internal forces in a special structural system. This system will behave as a virtual beam of span ℓ overhanging its supports 1 by cantilevers of span a, and being loaded by a concentrated load of magnitude 2P located in its centerline. It is considered that the virtual beam has the strength of two main girders on its center; of only one, in line with the two supports 1, and a transition from one to two, between these cross sections (Fig.3).

Since the described variation should be continuous in the vicinity of the centerline, as well as in that of the supports 1, the system can be considered as having a variable moment of inertia of:

$$I_v = I \, (1.5 - 0.5 \cos 2 \pi x / \ell) \tag{4}$$

where I is the moment of inertia of a single main girder; and x, the distance from the left support to any cross-section of the virtual beam (Fig.4).

However, for reasons of simplification, the variation of I_v will be considered as linear in accordance with the dashed line in Fig.4, and responding to the expression:

$$I_v = I \, (1 + 2 x / \ell) \tag{5}$$

The slope of the deflected shape of the virtual beam is determined by the equation:

$$\theta_x = \int_0^x \frac{M_x \, dx}{E \, I_v} + C_\theta \tag{6}$$

where C is an integration constant; E, the elasticity modulus for steel; and:

$$M_x = 2 P x \tag{7}$$

the bending moment along the virtual beam.

Implementing Eqs.5 and 7 in Eq.6, and determining the integration constant by writing $\theta_x = 0$, for $x = \ell / 2$, it follows:

$$\theta_x = - P \ell^2 [1 - \ln 2 \ell / (\ell + 2 x) - 2 x / \ell] / 2 E I \tag{8}$$

where ln is the symbol for the natural logarithms.

And the slope at the end 1 of the

virtual beam is determined by implementing $x=0$ in Eq.8:

$$\theta_0 = - P \ell^2 (1 - \ln 2) / 2 E I \quad \text{or}$$

$$\theta_0 = - P \ell^2 / 6{,}52 E I \qquad (9)$$

The deflection of the virtual beam is determined by the equation:

$$\Delta_x = \int_0^x \theta_x \, dx + C_\Delta \qquad (10)$$

where C_Δ is an integration constant.

By implementing Eq.8 in Eq.10, and determining the integration constant by writing $\Delta_x = 0$, for $x = 0$, it follows:

$$\Delta_x = - P\ell[x^2+\ell \; x \ln 2\ell-0.5\ell(\ell+2x)$$

$$\ln(\ell+2x)+0.5\ell^2\ln\ell]/2 E I \qquad (11)$$

where ln is the symbol for natural logarithms.

And the deflection at the centerline of the virtual beam is determined by implementing $x = \ell / 2$, in Eq.11:

$$\Delta_p = - P \ell^3 (0.25+0.5 \ln 0.5)/2 E I$$

or:

$$\Delta_p = - P \ell^3/20.71 E I \qquad (12)$$

VERY SUPPLE CROSS-FRAMES

Let the structure represented in Fig.1 be equipped with very supple frames, permitting therefore to each main girder to deflect independently when they are subjected to the corresponding concentrated load of magnitude P. In Fig.5 are represented the deflected shapes of the front main girder (with thick line) and that of the rear main girder (with thin line).

The equation of the deflected shape of the front main girder is:

$$\Delta_{xf} = P \, x(0.5\ell+a)[L^2-x^2-(0.5\ell+a)^2]/$$

$$/6 E I L \qquad (13)$$

And that of the rear main girder:

$$\Delta_{xr} = P(x+a)\ell[L^2-(x+a)^2-0.25\ell^2]/$$

$$/12 E I L \qquad (14)$$

The deflection in line with the center of the structure is:

$$\Delta_p = P(L+a)^2 \ell^2/ 48 E I L \qquad (15)$$

The differences between the deflections of the two main girders is:

$$\Delta_{xr}-\Delta_{xf} = P \, a[(0.25 \ell^2)(1.5\ell -x)+$$

$$+2 a \ell(0.5 \ell -x)]/6 E I L \qquad (16)$$

which is zero for 0.5ℓ; and $\ell^2(0.375 \ell+a)$, for $x=0$.

RELAXED CROSS-FRAMES

A skew bridge structure with an uplift tendency can be managed to lean on all its four supports, by increasing the flexibility of its cross-frames. As it results from Fig.5, the flexibility of the cross-frames should be increased with respect to their distance from the centerline, so that to match with the differences between the deflections expressed in Eq.16. When these differences are too large in line with some intended cross-frames, then these ones should be replaced by pin-connected floor beams as shown in Fig.6. For construction reasons, it is recommendable to size the remaining cross-frames so that to generate equal forces X_i. This will lead to patterned connections of the cross-frames to the main girders.

In such conditions, and considering that precautions were made for avoiding uplift, the magnitude of the reaction on the support 0, as it results from Fig.7, is:

$$R = (0.5 \ell P-c \, n \, X)/L \qquad (17)$$

where n is the numer of active cross-frames on each half span of the bridge.

The condition to prevent uplift is:

$$R_{OL}+R_{OD} = (0.5 \ell P-c \, n \, X)/L +R_{OD}>0$$

$$\text{or: } 0.5 \ell P-k \, c \, n \, X+L \, R_{OD} > 0$$

$$\text{or: } X < (0.5 \ell P+L \, R_{OD})/k \, c \, n \qquad (18)$$

where R_{OD} is the reaction on the support 0 from dead load only; and k, a coefficient of safety against uplift.

The deflection of each cross-frame should be equal to the difference of the deflections of the main girders in line with that cross-frame, when the bridge is loaded as in Fig.7. Thus:

$$X b^3/12 E I_i = \Delta_{ir}-\Delta_{if} \qquad (19)$$

where I_i is the moment of inertia of the cross-frame i; Δ_{xr} and Δ_{xf}, the deflections of the rear and the front main girder, respectively. It follows that the required moment of inertia of the cross-frame is:

$$\text{Req. } I_i = X b^3/12 E(\Delta_{xr}-\Delta_{xf}) \qquad (20)$$

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NUMERICAL EXAMPLE

Let each main girder W 36×300 of the skew ASTM A 36 steel bridge shown in Fig.1, be subjected to a concentrated load of 60 kip. Consider that the reactions on the outermost supports, due to the dead load, are 3.9 kip; and that the section modulus of the floor beams should be at least 70 in^3, required for resisting bending due to the stringer loading. Determine the deflections of both main girders in the assumptions that the cross-frames are: A.Very stiff; B.Very supple; and C.Relaxed. For the last assumption, size also the floor beams.

Data: $I=20300\ in^4$; min.$S=70\ in^3$; $E=29000$ ksi; $P=60$ kip; $R_{0D}=3.9$ kip; $k=1.25$; $L=100$ ft; $a=22$ ft; $b=13$ ft; $x_2=13$ ft; $x_3=26$ ft; $x_p=39$ ft. From Eq.1:$\ell=100-22=28$ ft. Consider other dimensional data as they are resulting from Figs.1; 3; 5; and 7.

Determine the deflections for the assumption A in accordance with Eq.11; for B, with Eqs.13 and 14; and for C, with usual formulas. All these deflections, and their differences as well, are contained in the Table 1.

Since the difference of deflections for the cross-section 1 is relative large, it is suggested to provide there a pin-connected floor beam. While in the cross-section P, there are not limitations for using a cross-frame as strong as it is required by other considerations.

There are remaining only two floor beams; 2 and 3, to be designed with the use of Eqs.18 and 20.Consequently, it follows: $n=2$; $c=39$ ft; $X=28$ kip, leading to:

Req.$I_2=449\ in^4$, for the floor beam in the cross-section 2. Use W 14×43, for which the effective moment of inertia is 428 in^4.

Req.$I_3=1222\ in^4$, for the floor beam in the cross-section 3. Use W 14×109, for which the effective moment of inertia is 1240 in^4.

Table 1. Deflections related to the Numerical Example

Cross-section	x	Deflections for the Assumptions: A		B		C		
	ft	r=f	r	f	r-f	r	f	r-f
1	0	0.00	2.01	0.00	2.01	1.27	0.00	1.27
2	13	2.03	2.91	1.42	1.49	1.92	1.24	0.68
3	26	3.49	3.39	2.61	0.78	2.41	2.16	0.25
P	39	4.04	3.32	3.32	0.00	2.57	2.57	0.00

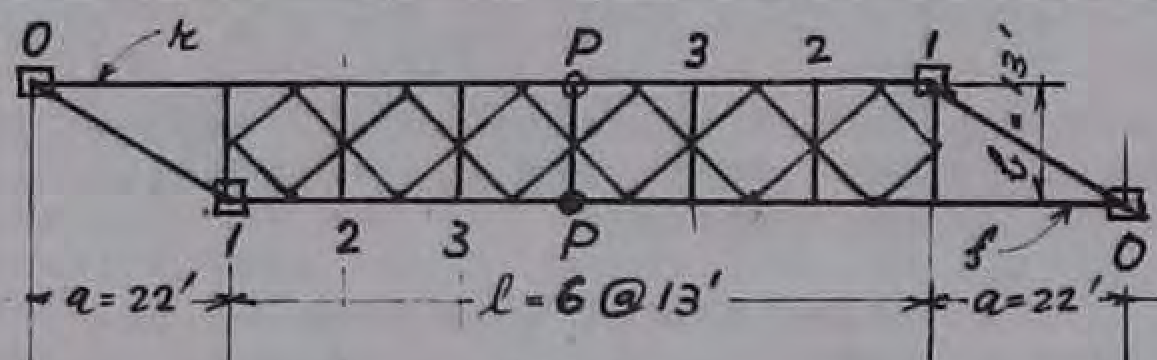

Fig.1

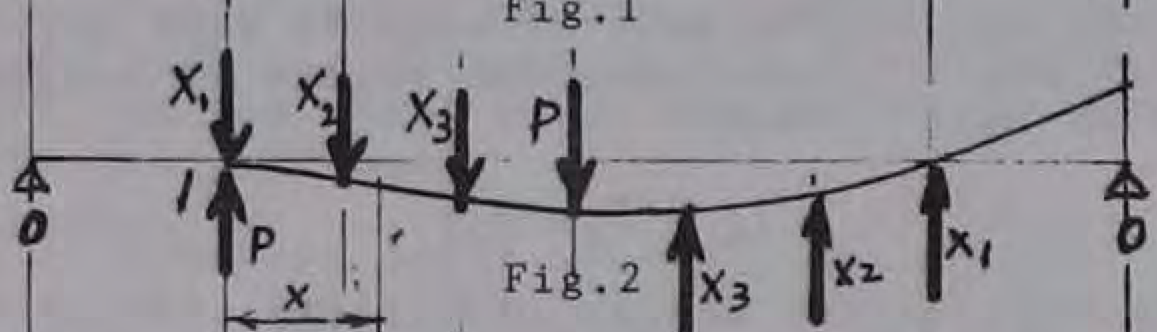
Fig.2

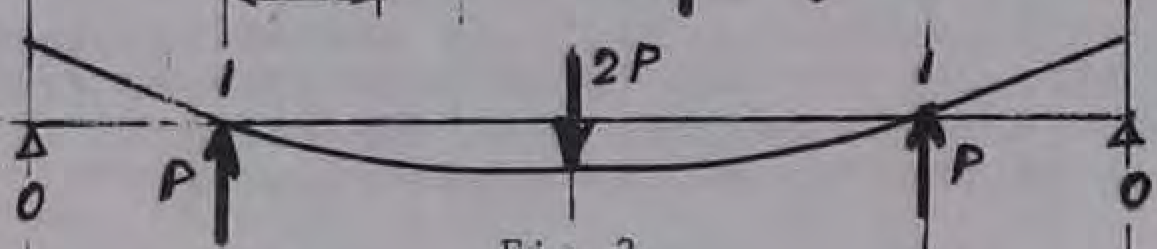

Fig.3

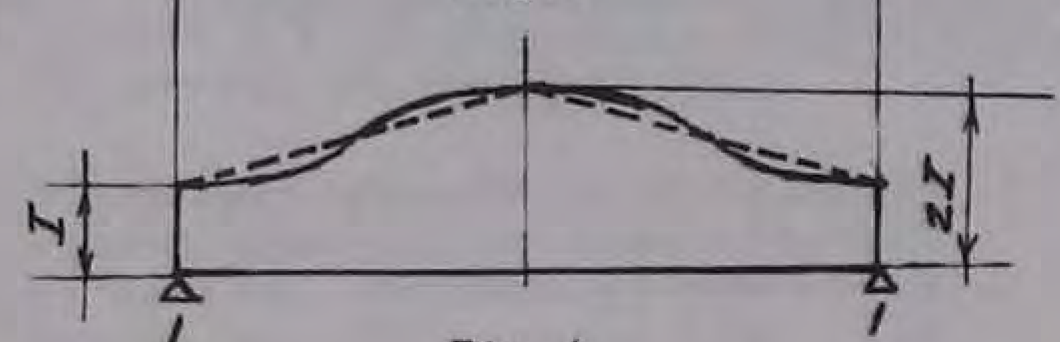
Fig.4

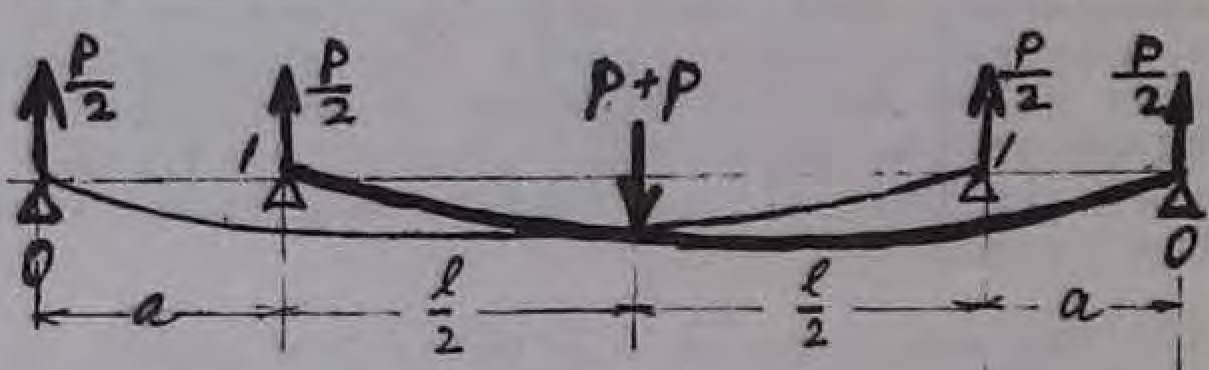

Fig.5

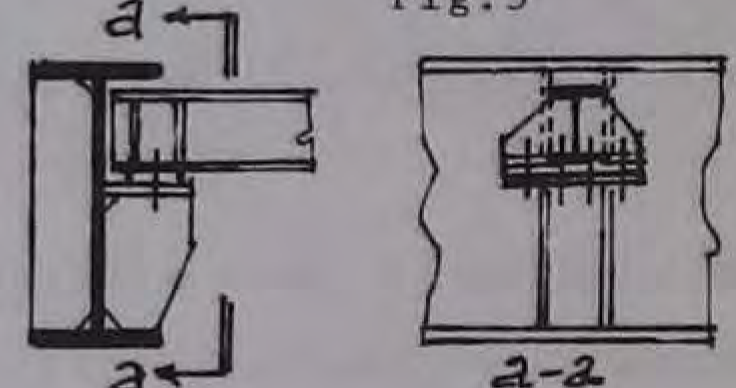

Fig.6

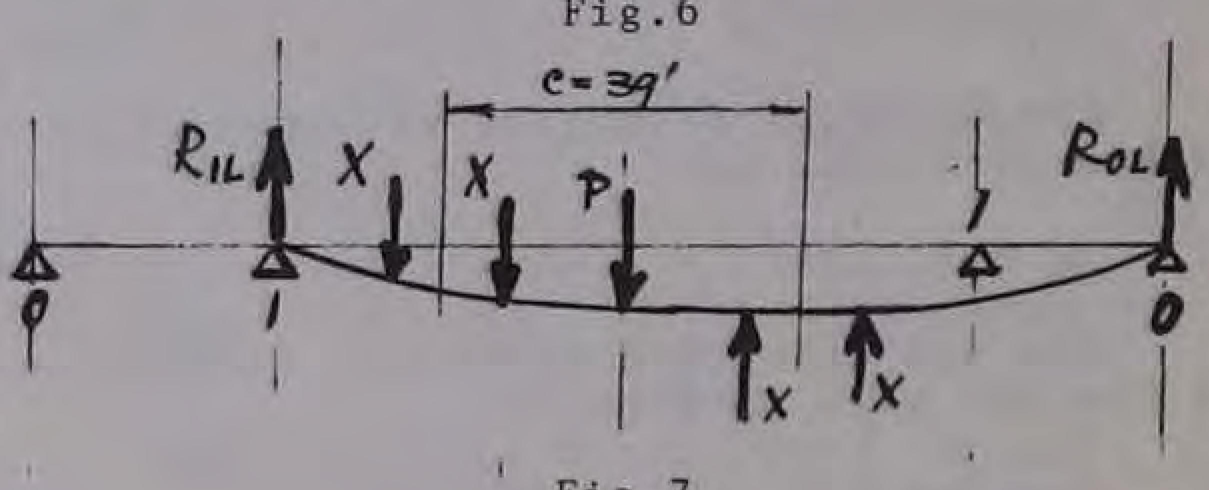

Fig.7

A RATING STUDY OF TIMBER DECK STEEL STRINGER BRIDGES

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SYNOPSIS

Load rating of steel stringer-timber deck bridges has been severely limited due to an AASHTO provision which prohibits the use of a timber deck as a lateral stiffening mechanism for the stringers. The research reported herein demonstrates that, contrary to the AASHTO specifications, the decking with or without connectors is adequate to provide the stability required. Analytical and experimental approaches are employed to establish the influence of lateral stability behavior of steel stringers. Experimentally it is demonstrated that, with the addition of timber deck panels to steel stringers, instability in the stringers has become almost nonexisting. This study suggests that AASHTO revise this provision to allow decking as the stiffening mechanism for the top flange of steel stringers.

INTRODUCTION

The vast majority of bridges in this country are on secondary roads and a large share of these are considered to be low volume bridges.Although their decks may be covered with a topping, many of the low volume bridges are constructed with timber decks on steel stringers.

Steel stringer-timber deck bridges suffer from a serious design limitation which has been addressed by this research program. Specifically, the allowable bending stress in the steel stringers (and consequently the allowable load) is limited by the laterally unbraced length of the stringer. According to the code which governs the design and ratings of these bridges, AASHTO Highway Bridge Specification (1), rolled steel sections may not use a timber floor system as lateral support mechanism unless the flooring and connectors are specifically designed for this purpose.

This AASHTO requirement results in overly conservative ratings of existing bridges and unnecessary expense in the construction of, or renovation of bridge structures. The additional cost of over-designing a new bridge could be easily determined but in some cases could be substantial.

Stated very simply, the goal of this project is to establish that lateral bracing of steel stringers can be achieved fully by the frictional forces which develop between the deck and the stringers, assisted by simple shear connector devices. The extent of lateral support provided by frictional forces may prove to be quite important since the effectiveness of connector devices will probably decrease as the decking deteriorates over time and repeated loading cycles.

SCOPE

All highway bridges on the federal aid system must conform to AASHTO Specifications for Highway Bridges, hence our goal of revising the specifications pertaining to timber deck-steel stringer bridges must begin with a thorough understanding of the existing provisions. The Ontario Highway Bridge Design Code (2) uses much the same procedure as the AASHTO code except for the addition of load factors.

Investigation of the rationale followed in the current AASHTO specifications shows that the most appropriate method of revising the AASHTO code provisions must be some method of reducing the effective braced length of steel stringers from the distance between diaphragms to some fraction of this distance. To do this we must show that lateral instability is not a problem in this type of bridge system, i.e., the load applied to each stringer is well below the critical lateral buckling load.

Two approaches have been followed by the research team to establish the effectiveness of timber decking as lateral support for the stringers. A series of experiments has been performed to test several of the parameters which could affect the stability of the bridge system and a stress analysis approach has been taken to determine the types of forces present in the stringer deck system. The stress analysis technique is combined with a rheological model to complete the analysis.

The experimental program was designed to evaluate the effects of:

1) type of connector and spacing,
2) absence of connectors,
3) type of timber deck,
4) spacing of stringers,
5) repeated loading.

Evaluation of these effects on lateral stability was accomplished by measuring deformations and strains of the stringers.

EXPERIMENTAL SET-UP AND DATA ANALYSIS

A model timber deck-steel stringer bridge was erected in the Major Units Lab at WVU. The initial model was constructed of three W14x22 stringers spaced 48" on center, but the 5-1/2" thick laminated timber deck was inadequate to span this distance for an HS-20 loading. The second model (see Figure 1) consisted of five W 12 x 44 sections (again not compact), each 30' in length, simply supported in the vertical direction and bolted together with end diaphragms. The spacing of the stringers was 22-1/2" center to center. Decking was either nail laminated panels, each

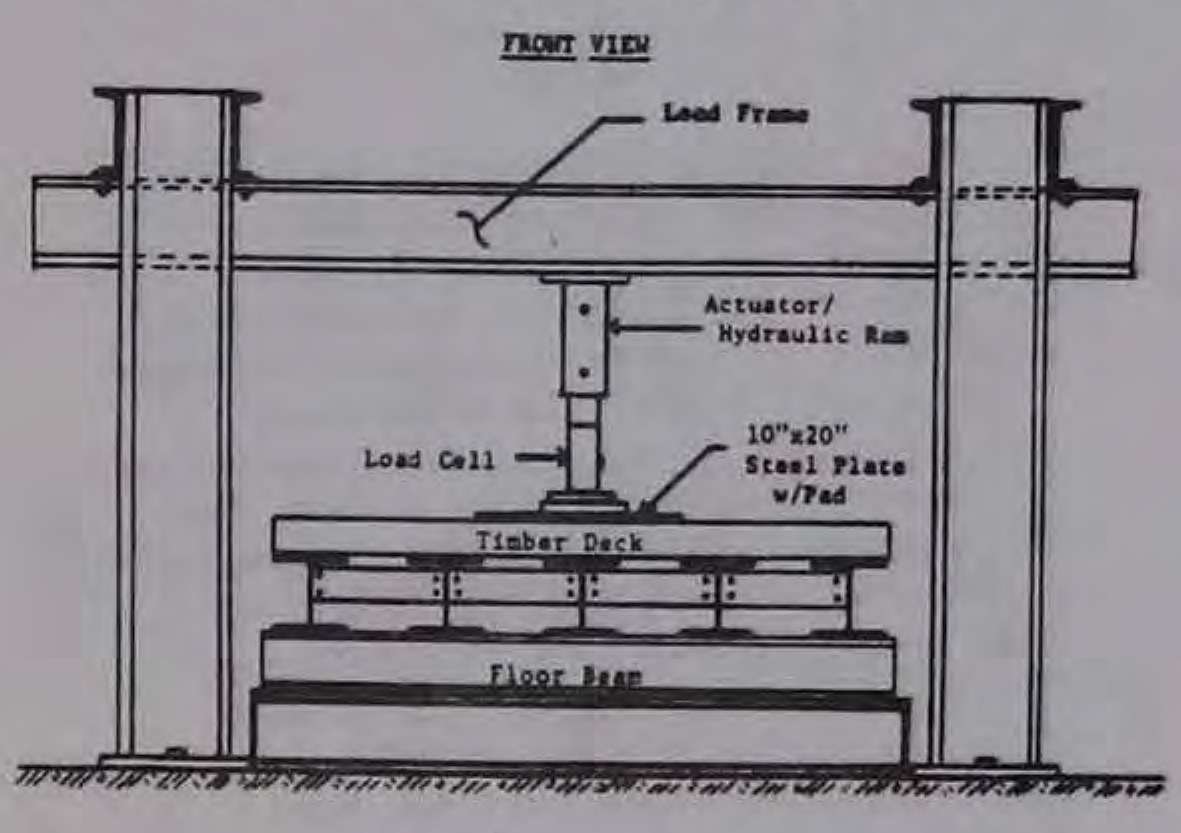


Figure 1, Typical Test Set-Up

consisting of eight southern pine 2"x6"x8' long, or yellow pine glulam panels, each 5-1/4"x12"x3'. Three connector types were used at spacings ranging from 12" to 36" center to center. Static loading was applied at various locations along the midspan centerline over a 10"x20" area to simulate the real wheel dimensions of a typical truck. Dynamic loading was applied by a two way actuator MTS system.

Instrumentation included single and rosette type strain gages to determine stresses and dial gages to measure displacements. The rationale for particular locations and the interpretation of the readings from these devices is covered in depth in the following sections.

Forty-two separate static tests were performed for various types of connectors, spacing of connectors, type of decking and load locations. Dynamic testing to 100,000 cycles was also performed, but only on a limited basis.

The basis for the experimental data analysis is the expectation of linear elastic behavior of the stringer when subjected to gravity loads. In a laterally stable system the relationship between loads and deformations and the relationship between loads and strains should vary linearly for all parameters.

This linear relationship has been found in every test performed for all the parameters tested (stringer spacing, connector type spacing, and deck type); none has shown the rapid rate of change of deformation with small increases of loading typical of buckling behavior. When tested to failure, the modes of failure have either been bending failure of timber or local stringer flange buckling.

The static test data and visual observation, have shown that the timber deck and connectors supply sufficient lateral support to the stringers, but is insufficient to provide composite action. A more thorough understanding of the nature of the stresses involved in the system is required before this conclusion can be confirmed; this is the undertaken in the next section.

THEORETICAL MODELLING

A rheological model was developed to test the validity of the stress analysis technique. In this model several important assumptions are made about the behavior of the model bridge system. Comparison of the deflections and strains generated by the idealized model to those of the constructed model allows verification of the assumptions, which if proven correct, are evidence that the model is valid. Another result of the model is the theoretical load distribution to the stringers which is then compared to the distribution of the experimental loads. These loads are compared to the theoretical critical load for lateral buckling to give another indication of the stability of the system.

The assumptions which make the rheological model (Figure 2) applicable to the constructed model are these:

1) the shear transfer between adjacent panels in the Z direction is quite low,

2) the interactive forces R, is 0 in the Z direction and is very small in the X direction,

3) the interactive moment, M is 0 in X and Y direction and is negligible in the Z direction.

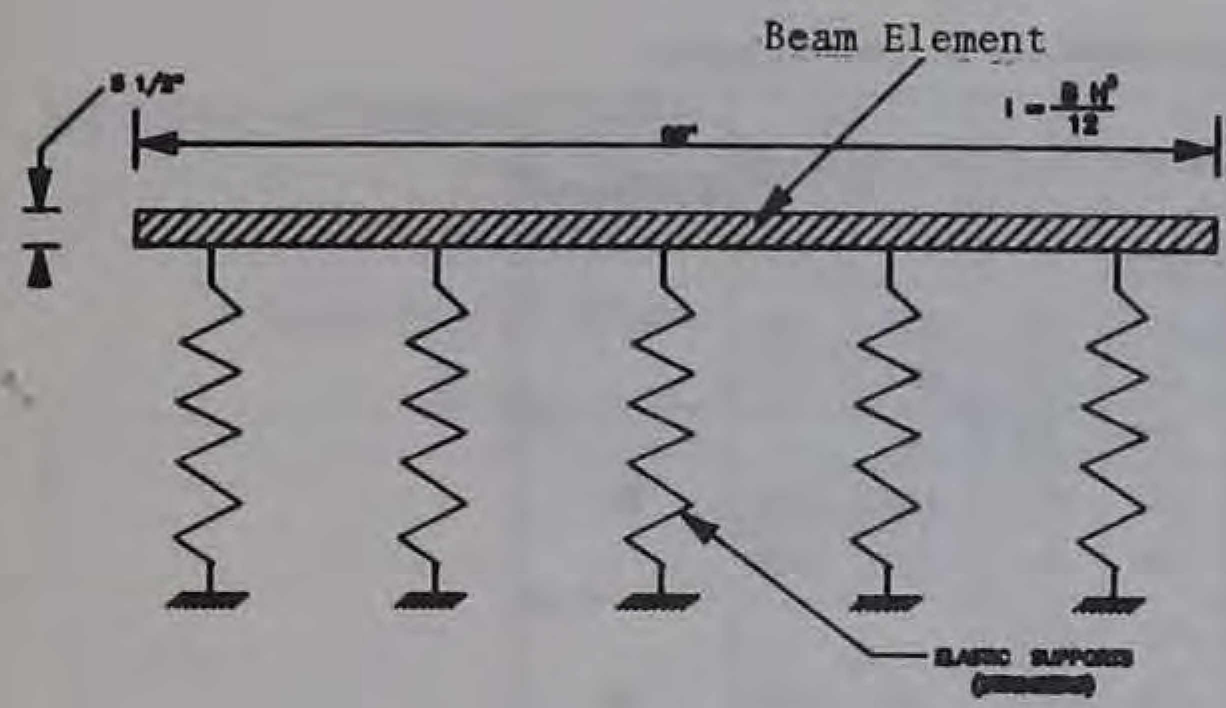


Figure 2. Rheological Model

Composite action has not been found for any of the connector types or spacings as established in the experimental data analysis.

Torsional forces on the stringer (moment about the Z axis) produce both pure torsion (St. Venant's torsion) and warping torsion. Although both pure and warping torsion produce shear stresses, only warping produces normal stresses in the flanges.

The use of the ε_{ADD} additional strain) terminology allows an artificial separation of the total normal strain into strains due to strong axis bending, ε_x, and the strains due to weak axis bending and warping, ε_{ADD}, which are related to lateral-torsional buckling.

Since lateral displacement and rotation are coupled in lateral stability problems, and have opposite signs, the <u>total</u> additional strain $\left(\varepsilon^T_{ADD}{}^B\right)$ can be found from two points B and C located on the edges of the lower flange of a stringer respectively:

$$\varepsilon^T_{ADD} = \varepsilon^B_{ADD} - \varepsilon^C_{ADD}$$

Table 1 shows the theoretical and experimental values for the three tests that that were used for the

It can be seen from Table 1 that in only a few cases do significant differences exist between theoretical and experimental results, and even these do not exceed 15% (on the stringer with the maximum load share). Extensive studies and experimental data and design examples can be found in reference 3.

CONCLUSIONS AND RECOMMENDATIONS

From both the experimental data and the stress analysis it is apparent that sufficient lateral support is provided by the decking with or without connectors. Even when directly loading the edge stringer (with deck and connectors) to

One approach is to revise this article to allow some fraction of the distance between diaphragms as the effective unbraced length. Since the research showed that the type or spacing of connectors had little effect on the stability of the system and since any effect of connectors will probably deteriorate with time and load cycles, it makes little sense to try to relate effective unbraced length to connector spacing.

Our recommendation is that rolled steel stringers be considered as fully braced by timber decks with any adequate connector. At least within the elastic range of the stringers, the behavior of the model bridge had showed no signs of lateral torsional failure. Frictional forces are sufficient to provide the lateral support required, and the use of any approved connector type will maintain the contact required and contribute additional lateral support. Article 10.33.1.2 should be either eliminated entirely, or revised to read:

"The compression flange of rolled beams supporting timber floors shall be considered as fully supported when adequate fastening devices are used."

The adoption of this provision would require a thorough understanding of a multitude of stress and serviceability limits. The contents of this paper have included only stress in the stringers; stress in the connectors and decking as well as serviceability limits on deformation and vibrations must also be considered.

The proposed alteration of the code would have significant impact on both new design of steel stringer-timber deck bridges and the ratings of many bridges of this type currently in use.

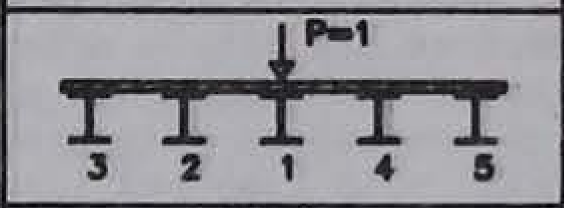
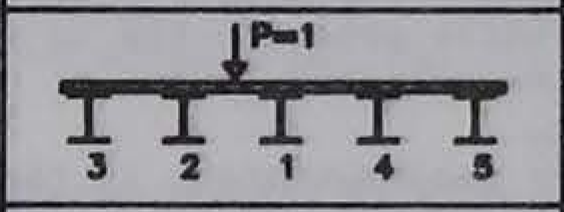
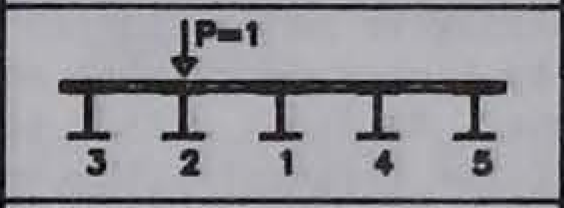
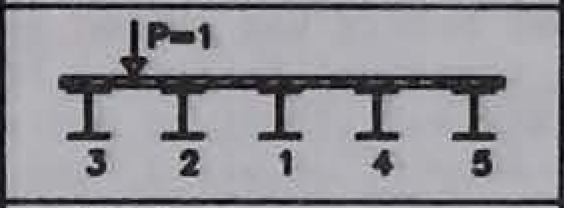
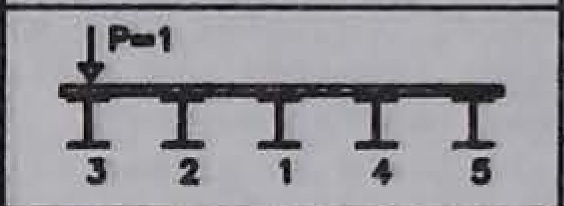
REFERENCES

1) *Standard Specifications for Highway Bridges*, American Association of State Highway and Transportation Officials, Thirteenth Edition, 1983, Washington, DC.

2) *1983 Ontario Highway Bridge Design Code*, Ministry of Transportation and Communications, Highway Engineering Division, Toronto, Ontario.

3) C.C. Spyrakos and H.V.S GangaRao, *Lateral Stability of Steel Structures Stiffened by Laminated Timber Deck*, Final Report to West Virginia Department of Highways, 1988.

Table 5.1 Normalized Interactive Forces Over Stringers

Load Position	Source	Str.#3	Str.#2	Str.#1	Str.#4	Str.#5	% Difference*
	Strain EXPERIMENTAL	.168	.209	.271	–	–	−7.7
	Deflection EXPERIMENTAL	.202	.237	.257	.237	.190	
	computer ANALYSIS	.134	.227	.277	.227	.134	
	Strain EXPERIMENTAL	.262	.360	.252	–	–	20
	Deflection EXPERIMENTAL	.322	.306	.250	.184	.086	
	computer ANALYSIS	.240	.286	.262	.164	.046	
	Strain EXPERIMENTAL	.389	.347	.200	–	–	7.8
	Deflection EXPERIMENTAL	.439	.359	.228	.122	−.004	
	computer ANALYSIS	.367	.331	.227	.100	−.026	
	Strain EXPERIMENTAL	.534	.322	.168	–	–	11.4
	Deflection EXPERIMENTAL	.581	.396	.205	.062	−.100	
	computer ANALYSIS	.516	.355	.183	.0366	−.090	
	Strain EXPERIMENTAL	.720	.317	.096	–	–	10
	Deflection EXPERIMENTAL	.750	.391	.121	−.061	−.311	
	computer ANALYSIS	.674	.367	.134			

* Maximum difference between theoretical and experimental value at the point of maximum load share.

–: Data not recorded

STATIC STRENGTH OF SIMPLY SUPPORTED PRESTRESSED COMPOSITE GIRDERS

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SYNOPSIS

According to the 1986 U.S. Federal Highway Administration statistics, there are 574,729 bridges in the highway systems. About half of these bridges are structurally deficient and/or functionally obsolete. External prestressing can be used for the strengthening of these bridges. The potential benefits of this method of construction are to; enlarge the elastic range of behavior; redistribute internal stresses in a favorable manner; decrease deformation; reduce steel weight; increase fatigue strength; and increase redundancy by providing multiple load paths. This paper examines the behavior of prestressed composite steel concrete girders.

Introduction

Ten prestressed composite concrete-steel girders were tested to their ultimate capacity. Four beams, A, C, D and E were tested under positive bending moment loading. One beam, B, and five plate girders, F, G, H, I and J were tested under negative bending moment loading. The experimental program was designed to study several aspects of prestressed composite girders including tendon type, loading type, dimensions of the cross-section of the girders, concrete slab type, tendon profile, epoxy coating on tendons, and the bond between the tendons and concrete. The experimental program was designed after a thorough literature search and analysis (3, 4, 5, 6, 7, 8).

Beams A and B were tested initially as a preliminary study and the results were reported to the National Science Foundation in an interim report (5).

In this paper, the structural behavior and the test results of specimens, A, B, C, D, E, F, G, H, I, and J are summarized.

Experimental Program

In this study, four specimens were subjected to a positive bending moment loading.

The 4572 mm (15 ft.) long Beam A, has a 915 mm (3 ft.) wide by 75mm (3 in.) thick concrete slab compositely connected to a W 14x30 rolled beam of ASTM A588 high-strength low-alloy steel. The steel beam was prestressed with two 16mm (5/8 in.) diameter Grade 150 Dywidag Thread bars running 57mm (2-1/4 in.) below the bottom (tension) flange along the full beam length. The beam was prestressed before the concrete slab was cast. Specimens C, D, and E are described in Table 1. They were tested as simply supported beams with two concentrated loads symmetrically applied about the mid-span of the beam, 457mm (1.5 ft.) to the right and left of the beam's center. The span of the beams was 4572 mm (15 ft). Beams C and D were tested to compare and study the differences in the structural behavior between bars and strands as prestressing tendons. Beams D and E were tested to study the difference in structural behavior between straight and draped tendon profiles.

Six specimens were subjected to negative bending moment loading. They include one beam and five plate girders. Beam B has a 455mm (18 in.) wide by 75mm (3 in.) thick concrete slab compositely connected to a W 14x30 rolled beam of ASTM A-588 steel. The composite beam was prestressed with two 16mm (5/8 in.) diameter Grade 150 bars running 14mm (9/16 in.) below the top (tension) flange, placing the concrete under compression. The plate girders can be divided according to their failure modes into three general classes. Girders F and I have compact flanges and non-compact webs. Girders G and J have non-compact flanges and non-compact webs. Girder H has non-compact flange and compact web. Girders F and I are identical except for the type of tendons in the precast concrete slabs. Girder F has six uncoated strands, and Girder I has six coated strands in the precast concrete slab. These two girders were tested to study the effect of epoxy coating on the strands on the structural behavior of the girders. Girders G and J are identical except that girder G has precast prestressed concrete slab and girder J has post-tensioned cast-in-place concrete slab. The specimens are described further in Table 2.

The dimensions of the specimens were based on scaling down a real bridge. The bridge was designed according to AASHTO bridge specifications (1). Figure 1 shows the dimensions and loading conditions for specimen C for illustration purposes.

Each specimen was instrumented using strain gages on the steel flanges, web, top and bottom surfaces of the concrete slab, reinforcing steel in the concrete slab and inside the concrete slab. Load cells were used to measure the applied load and the prestressing force of the post-tensioning tendons. Displacement transducers and clinometers were used to measure mid-span deflections and end rotations, respectively.

All beams and girders were incrementally loaded to failure. The deflection, strains in the prestressing bars, the forces in the prestressing tendons, end rotations, the strains in the steel beams and concrete slabs were measured at midspan using a high speed data acquisition system.

For illustration, the load-displacement curve for specimen C is shown in Figure 2 with solid lines. The load end rotation curve for specimen C is shown in Figure 3. The load versus increase in the prestressing force for specimen C is shown in Figure 4. The strain distributions throughout the depth of beam C at selected load levels are shown in Figure 5. Many other curves are available (2); however, cannot be shown herein due to the paper size limitation.

Analytical Study

Analytical models and computer programs were developed to predict the behavior of prestressed, composite girders in the elastic, inelastic and plastic regions. The following main assumptions were made: (1) linear strain distribution through the full depth of the composite steel-concrete girder; (2) small deflections; (3) no creep and shrinkage deformations; (4) no residual stresses in the steel girder; and (5) complete interaction between the concrete slab and steel girder; i.e., no slip. The stress relaxation in the tendons and shear deformations were considered.

The increase in the tendon's force, due to dead and live loads, was calculated using both a compatibility condition for the longitudinal deformations of the tendons and the corresponding girder fiber at the anchorage points and energy methods.

An incremental deformation technique, based on the principles of compatibility of the deformations and equilibrium of the forces, was used to determine the state of the stresses and deformations in the elastic and inelastic regions. The ultimate capacity was predicted by assuming complete plasticization of the section and applying the equations for equilibrium of the forces and moments.

For illustration, the calculated load-deflection curves of beam C is shown in Figure 2 with dashed lines. The predicted behavior agrees well with the measured load-displacement curve.

Based on the results of this study, equations were proposed for the working stress and ultimate strength design of pre-stressed, composite steel-concrete beams and girders subjected to positive and negative bending moments. The initial and final (service) stages of loading are considered in the proposed design methodology.

A detailed description of the tests, results, analytical models and design guidelines are described by Ayyub, Sohn and Saadatmanesh (2).

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8. Tochacek, M. and Mehta, C.L. (1972), "Economical Design of Prestressed Plate Girder," J. Struc. Div., ASCE, 98(6), 1273-1289

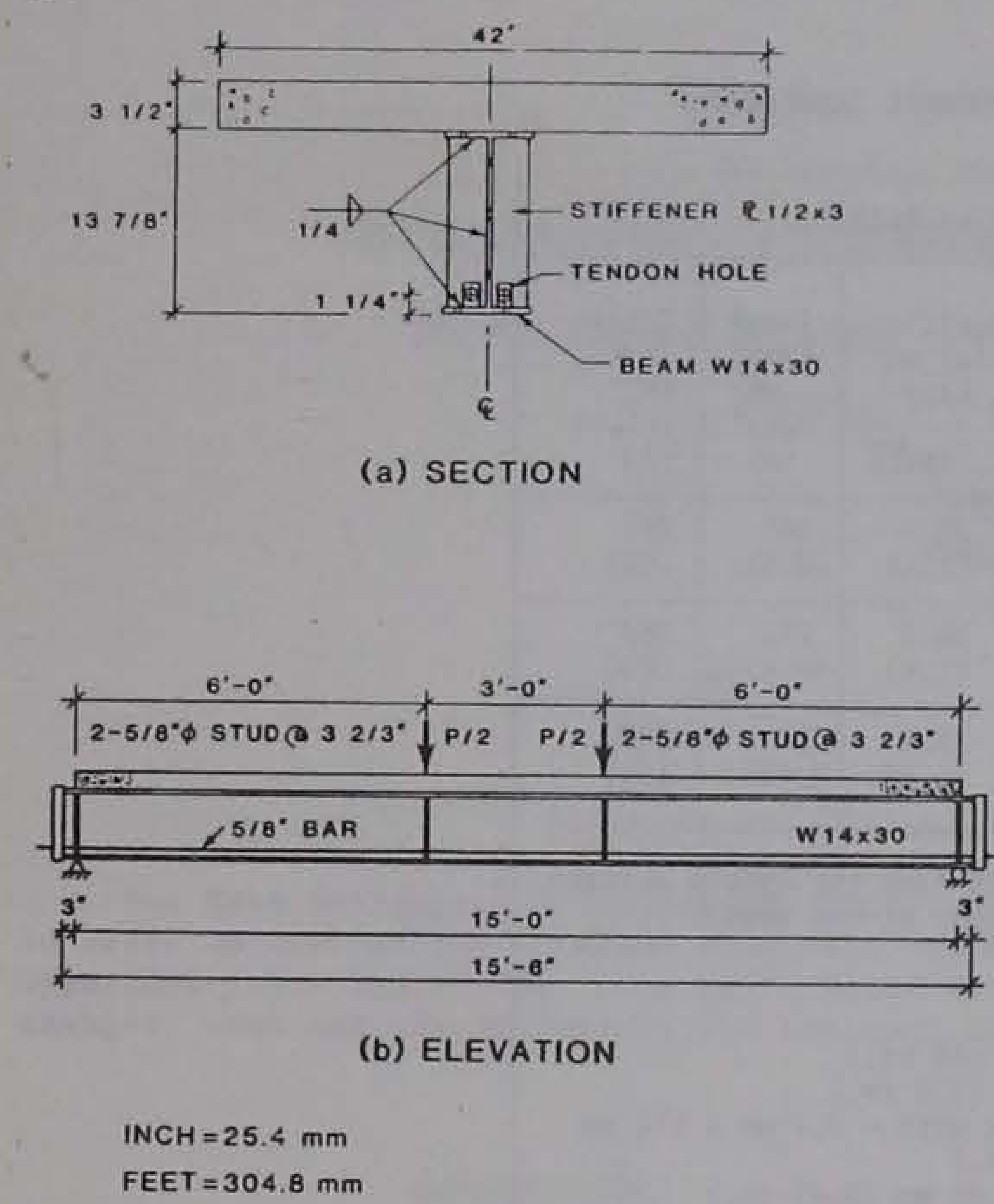

Fig. 1. Beam C

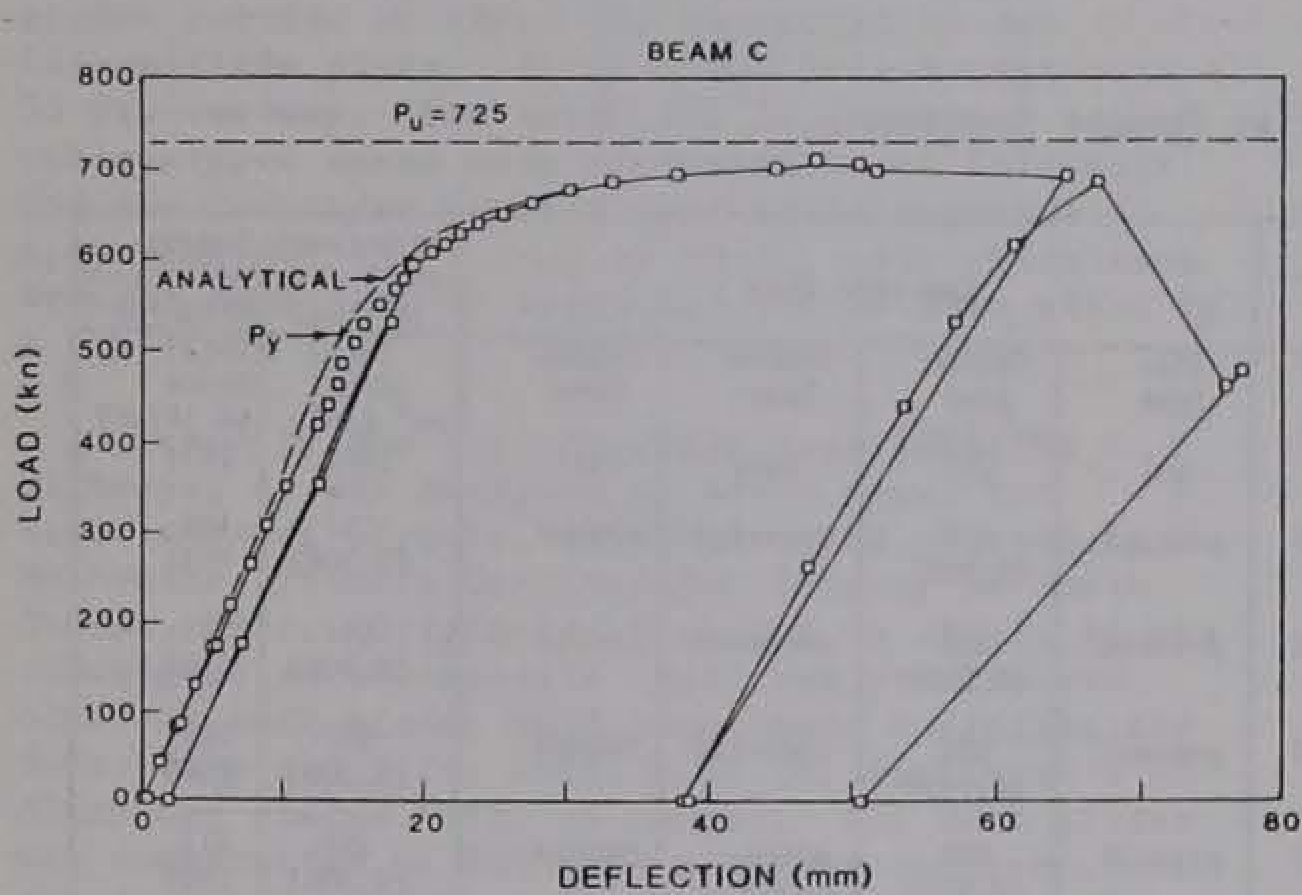

Fig. 2. Load Deflection

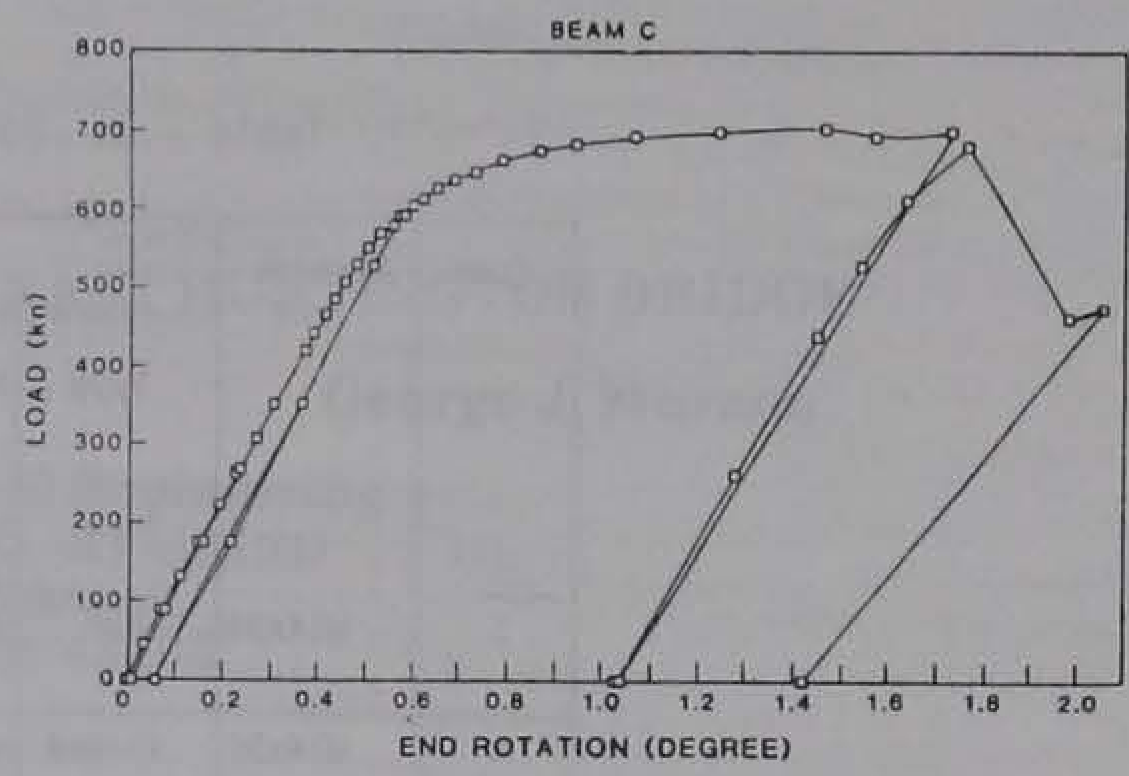

Fig. 3. Load-End Rotation

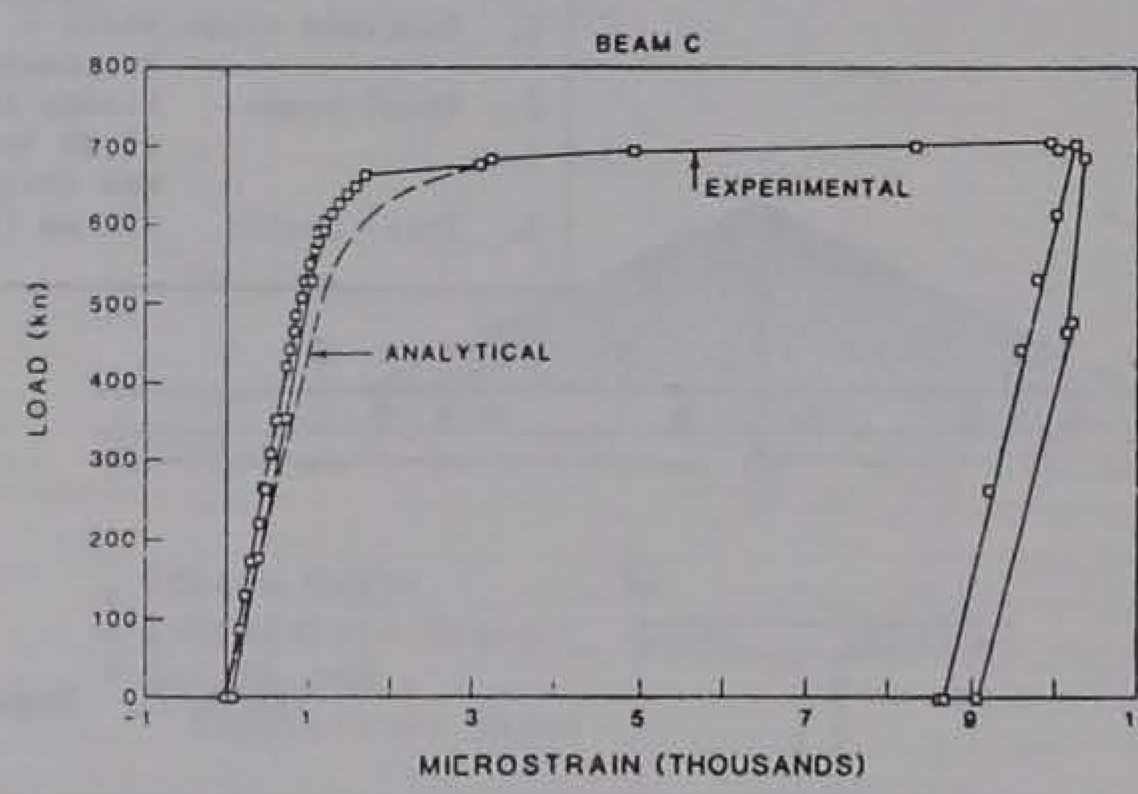

Fig. 4. Prestressing Bar Strain

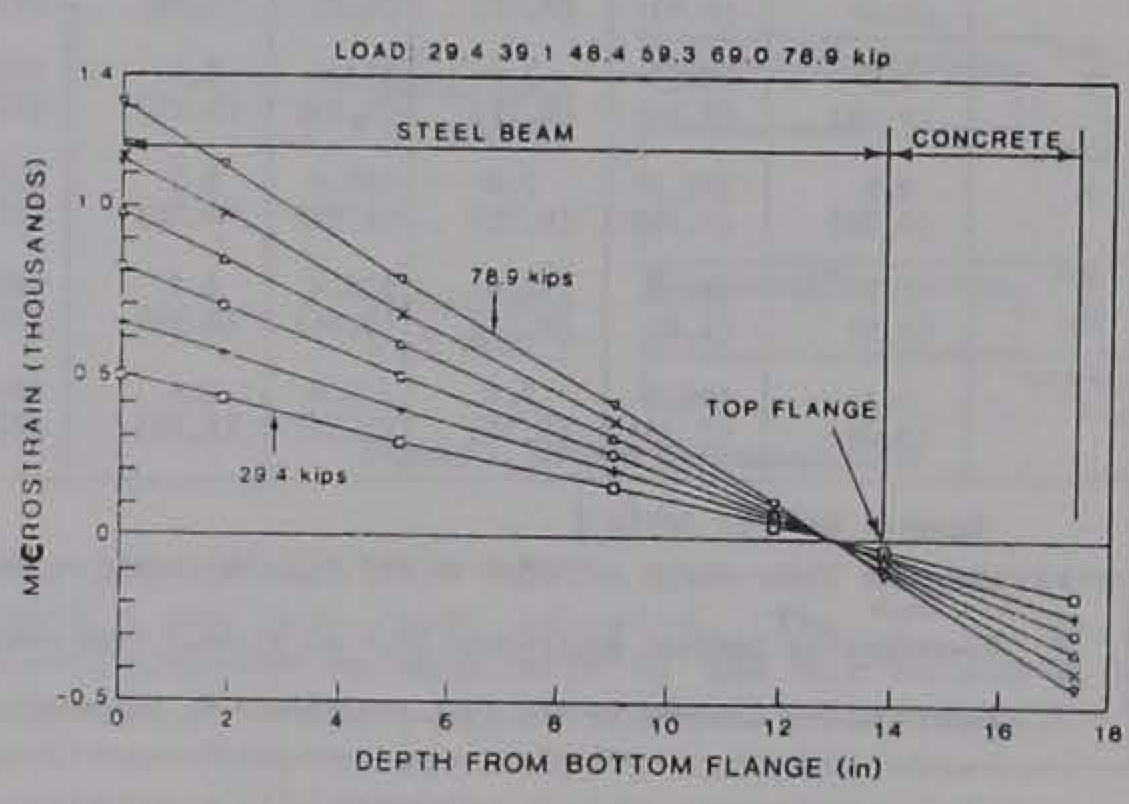

Fig. 5. Strains Through Depth
(Beam C)

Table - 1. Positive Moment Specimens

Beam	Section Type	Prestressing Tendon				
		Type	Profile	Position (a) mm (in)	Total Area mm² (in²)	Total Force KN, (Kips)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
C	W14x30	bar	straight	30.5 (1.2)	361 (0.56)	267 (60)
D	W14x30	strand	straight	30.5 (1.2)	279 (0.432)	289 (65)
E	W14x30	strand	draped	30.5 (1.2)	279 0.432)	267 (60)

(a) Position of tendon is the distance from the bottom surface of the tension flange at the midspan of the beam

General Notes on Table 1

1. Concrete slab: Width = 1066.8 mm (42 in.)
   Thickness = 89 mm (3.5 in.)
2. Steel beam: Flange thickness x with = 9.7 mm x 171 mm (0.38 in x 6.73 in.)
   Web thickness = 6.86 mm (0.27 in.)
3. Total Depth: 352 mm (13.86 in.)

Table 2. Negative Moment Specimens

	Steel Girder						Concrete Slab				Prestress Tendon in Steel Girder	
	Compress Flange		Tension Flange		Web							
Girder	Thickness mm (in)	Width mm (in)	Thickness mm (in)	Width mm (in)	Thickness mm (in)	Depth mm (in)	Slab Type	Tendon Area mm² (in²)	Tendon Type	Tendon Bond	Total Area mm² (in²)	Total Force KN (Kips)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
F	12.7 (0.5)	147.6 (5.8)	7.9 (0.31)	133.4 (5.25)	6.4 (0.25)	685.8 (27)	precast	592 (0.918)	uncoated	bonded	197 (0.306)	196 (44)
G	9.5 (0.38)	196.9 (7.75)	7.9 (0.31)	133.4 (5.25)	6.4 (0.25)	685.8 (27)	precast	592 (0.918)	uncoated	bonded	197 (0.306)	205 (46)
H	9.5 (0.38)	176.9 (7.75)	7.9 (0.31)	133.4 (5.25)	9.5 (0.38)	508.0 (20)	precast	592 (0.918)	uncoated	bonded	197 (0.306)	222 (50)
I	12.7 (0.5)	147.6 (5.8)	7.9 (0.31)	133.4 (5.25)	6.4 (0.25)	685.8 (27)	precast	592 (0.918)	epoxy coated	bonded	197 (0.306)	213 (48)
J	9.5 (0.38)	196.9 (7.75)	7.9 (0.31)	133.4 (5.25)	6.4 (0.25)	685.8 (27)	cast-in-place	836 (1.296)	uncoated	unbonded	0	0

General Note on Table 2

1. Concrete Slab: Width = 1966.8 mm (42 in), thickness = 101.6 mm (4 in.), and prestressing force = 774 KN (167.3 kips).
2. Prestressing tendons positioned 25.4 mm (1 in.) from extreme fiber of tension flange in steel girder.
3. Prestressing tendons in the slab positioned in the middle of the thickness of the slab.

STRAIN MONITORING OF THE EAST HUNTINGTON BRIDGE

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SYNOPSIS

The East Huntington cable-stayed bridge was extensively instrumented with strain and temperature gages in order to monitor the erection stresses. With the completion of construction, the system afforded an opportunity for the bridge to become a field laboratory for the investigation of the effects of temperature changes, wind and time-dependent strains upon the structure.

INTRODUCTION

The East Huntington Bridge is a prestressed, precast cable-stayed bridge across the Ohio River between Huntington, West Virginia and Proctorville, Ohio. The cable-stayed portion is unsymmetrical, supported by a single tower, with 635.5 ft. north of the tower and 715.0 ft. south of the tower. There is also on the south end a cast-in-place box girder portion of 620.0 ft. supported by two intermediate piers. It is a two lane bridge with a 35 ft. roadway. The north 236 ft. of the cable-stayed spans were cast-inplace on falsework and the remainder precast upriver in segments approximately 45 ft long by 45 ft. wide which were brought downriver by barge and lifted into place by a crane.

Owned by the West Virginia Department of Highways, it was designed by Arvid Grant and Associates of Olympia, Washington and erected by Melbourne Brothers Construction Company of North Canton, Ohio. Originally designed in steel, it was redesigned as a composite steel and prestressed concrete deck girder with edge beams approximately 5 ft. deep and 5 ft. wide, a 10 in. deck with W 33 diaphrams placed 4 ft. on center. The deck girder was constructed of 8000 psi zero air concrete. At ten locations selected by the designer along the length of the bridge, 14 to 16 strain gages were placed within the girder and at four of these locations 14 to 16 temperature gages were also located. These locations, shown in Figure 1, are approximately midway between cable supports. Their purpose was to monitor the erection stresses.

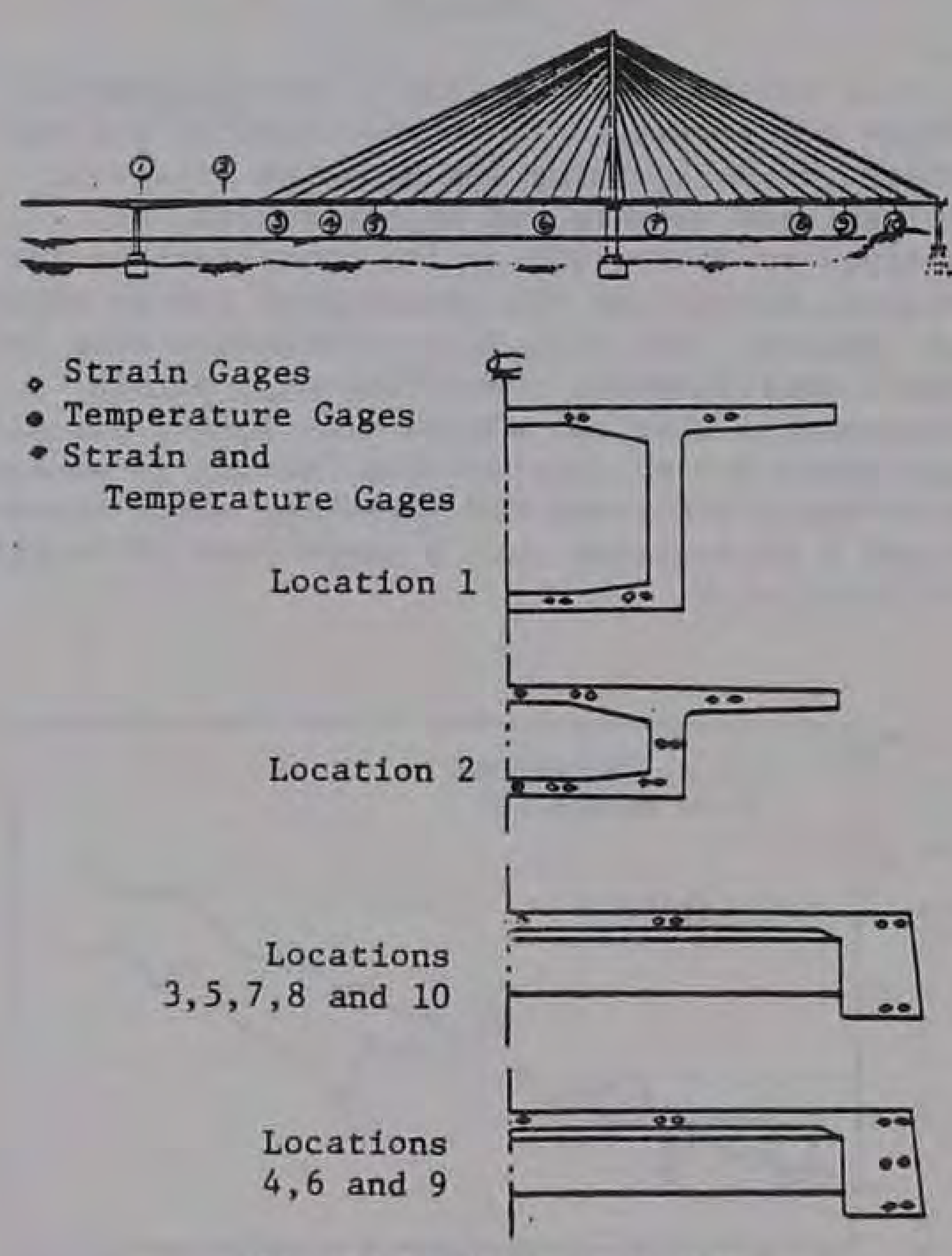

Figure 1. Bridge Instrumentation

INSTRUMENTATION

No. 7 reinforcing bars, 5 ft. - 8 in. long were turned down at the center and used to mount the strain gages. Encapsulated constantan alloy foil 350 ohm gages with a "T" configuration were applied to each side of the prepared bar surface with a two part epoxy adhesive. The "T" configuration on each side of the bar formed a full wheatstone bridge circuit with all four legs active. Temperature sensors were bondable resistance-thermometer gages constructed like strain gages. The sensing grid is of high-purity nickel foil. These were used with a linearizing network which linearizes the resistance-vs-temperature characteristics and forms a balanced 350 ohm half bridge circuit to the strain indicator at 75° F. All gages were protected by the normal waterproofing followed by two layers of a two part polysulfide modified epoxy compound separated by a layer of aluminum foil. Additional protection was provided by encasing the instrumented portion of the bar in a length of thick walled heat shrink tubing with a sealant.

Readings were taken manually with portable strain reading equipment until the installation in the south tower leg of the permanent automatic data acquisition system. This consists of a microprocessor-controlled computing, measurement and control system with an external printer in the tower leg and ten peripheral scanners, one located at each station. The data logger has circuitry to take readings from up to 500 channels (each channel is a full or half bridge). The scanners, located at each station, are medium speed analog switching devices capable of 50 strain channels each. Each scanner is housed in an insulated NEMA type 4 enclosure with a thermostatically controlled heating unit. These enclosures are mounted on the deck except for the first two stations which are located within the box girder section on the south end of the bridge.

RESULTS

As mentioned above, the strain monitoring system was installed to measure strains for the purpose of determining the erection stresses. Modjeski and Masters, of Mechanicsburg, PA, retained by W. VA. DOH as its consultant on the project, determined the theoretical stress values and compared them with those determined from the strain measurements. Unfortunately, strain measurements were not always made before and after each event during construction, making it necessary to estimate the creep and shrinkage which occurred. Figure 2 illustrates such a comparison of strains for Location 6.

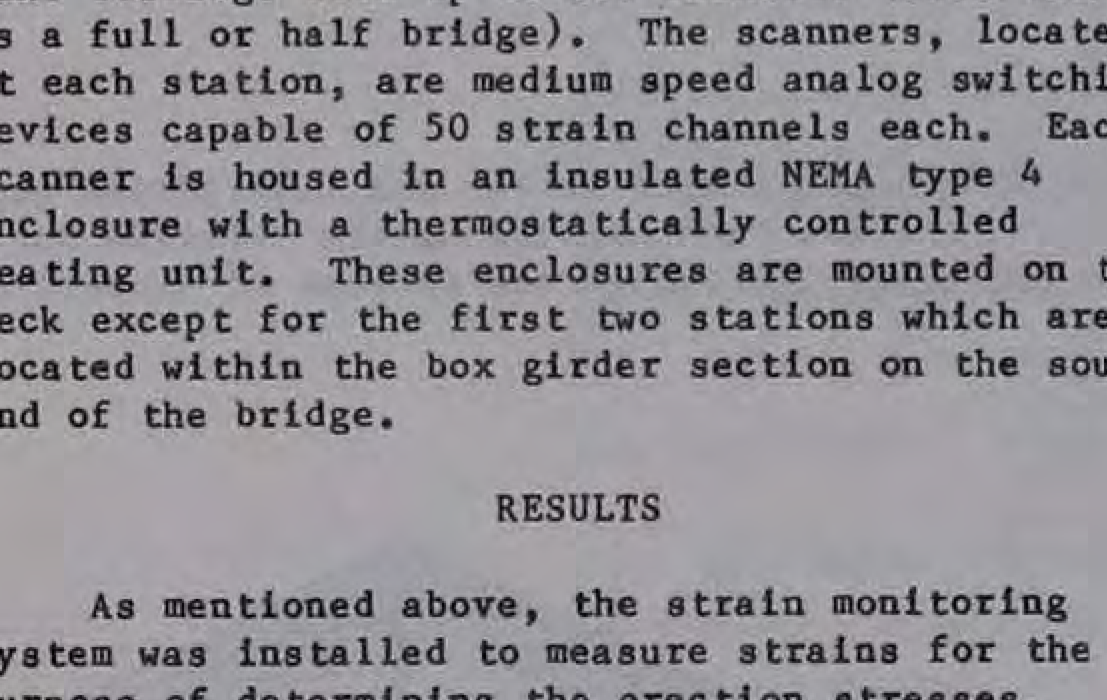

Figure 2. Erection Strains

With completion of the bridge, the strain system, which is still operable, has made the East Huntington Bridge a laboratory. Creep and shrinkage estimates have been developed by L. Hanley (1) from a creep and shrinkage study by the Construction Technology Laboratory (CTL) of Skokie, Illinois, by modifying Branson's Creep Curve which was adoped by ACI Committee 209 (Figure 3). Currently, efforts are

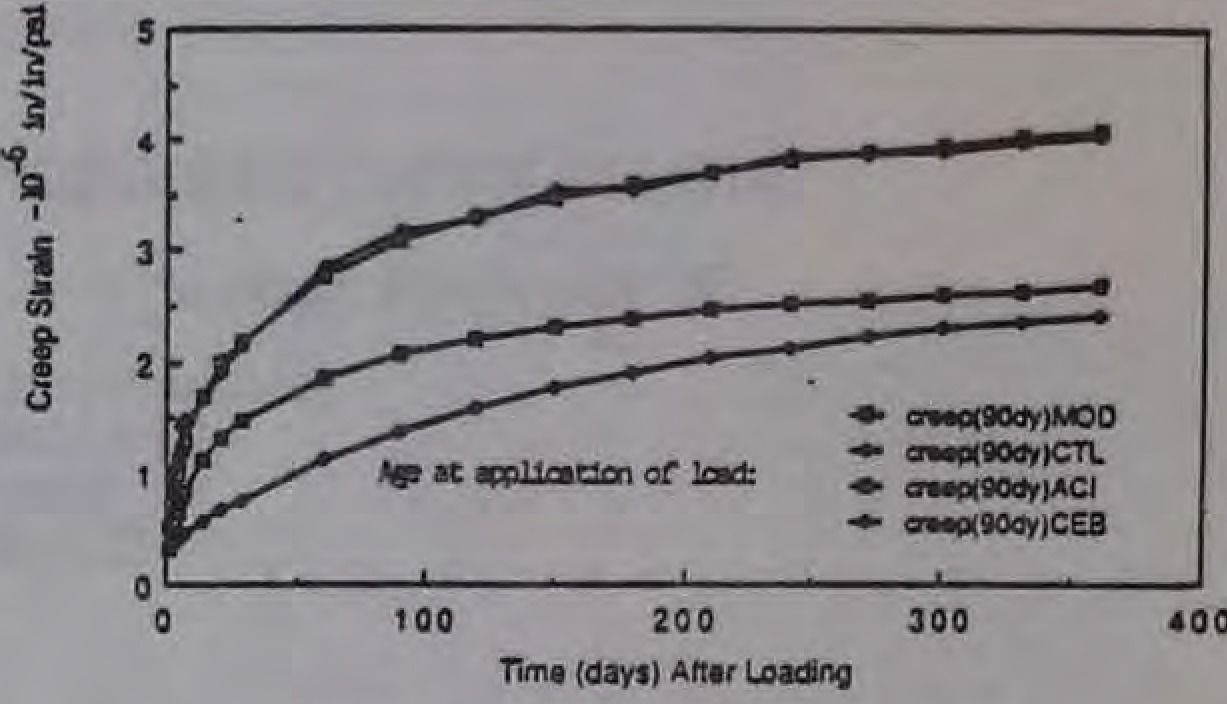

Figure 3. Estimated Creep Strain vs Time

being made to acquire all theoretical stress estimates made during erection and to compare them with those determined from the strain readings, but incorporating the modified creep and shrinkage estimates. A preliminary study has been made, also by Hanley, of the effects of temperature change on the strains along the top and the bottom of the deck, the purpose of which is to determine the change in stresses and in moments along the length of the bridge with changing temperatures. In addition, strain variations vs wind speeds have been recorded by Mr. Claude Skiles of the W. VA DOH (Figure 4). Skiles also compared the anticipated stresses after final cable adjustment with those calculated from average strain readings taken from 2/12/86 to 5/2/86 which are shown in Figure 5. These however are not corrected for creep or shrinkage. Such corrections are in the process of being made and more accurate values should soon be available. A study is underway to verify from the strain readings, or to modify the creep estimates obtained by Hanley from the CTL creep study.

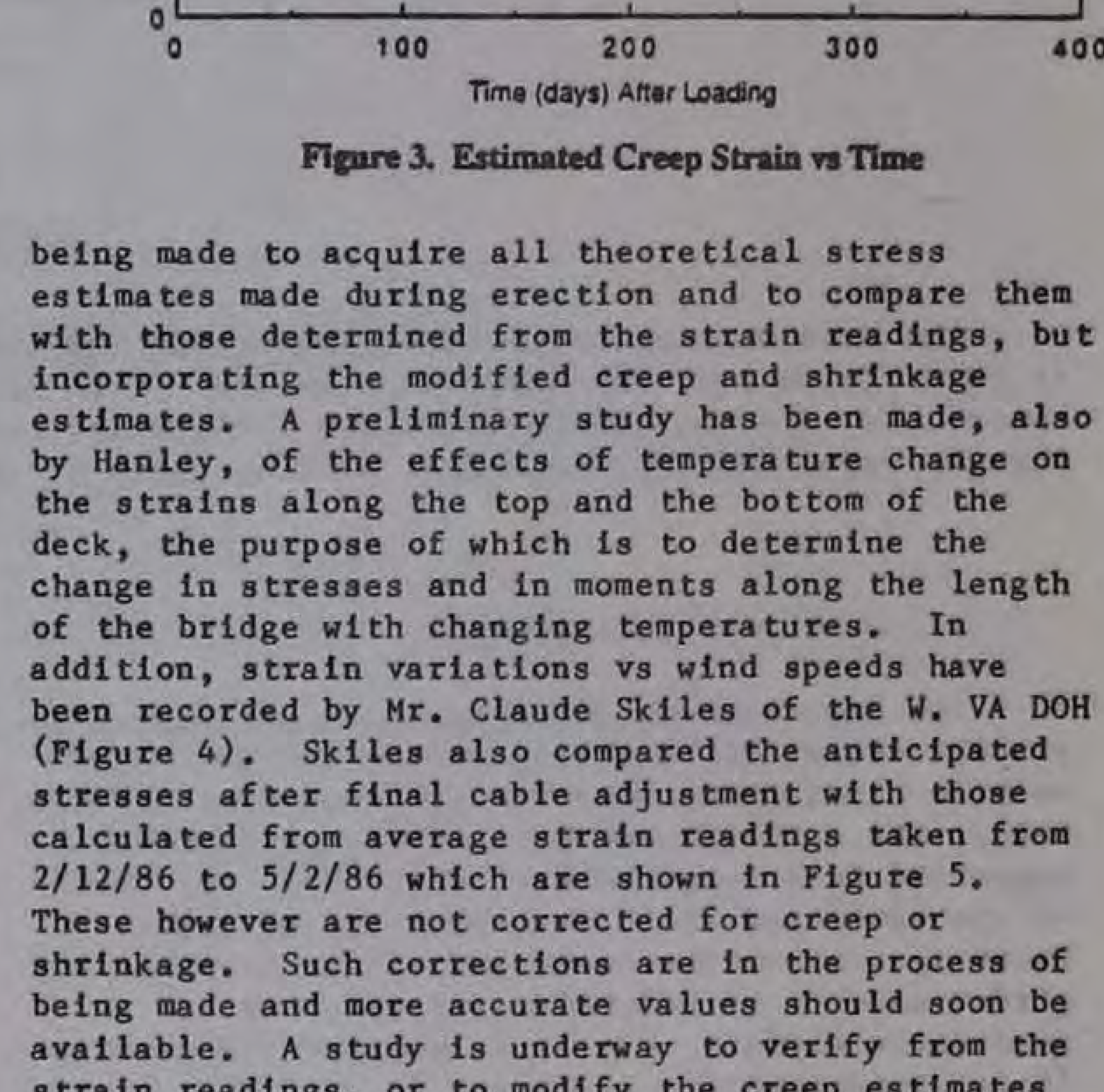

Figure 4. Strain vs Wind Speed

Hanley concluded that an increase in the top, bottom and ambient temperature had little affect on negative average or centroidal strains at the gage locations for segments near the tower. However, for segments near the end supports, the negative centroidal strains increased with increasing temperature. All segments show an increase in the top negative strains as the top and ambient temperatures increase. With the exception of segments near the abutment on the north, all other segments north of the tower showed a decrease in negative strains with increasing bottom temperature.

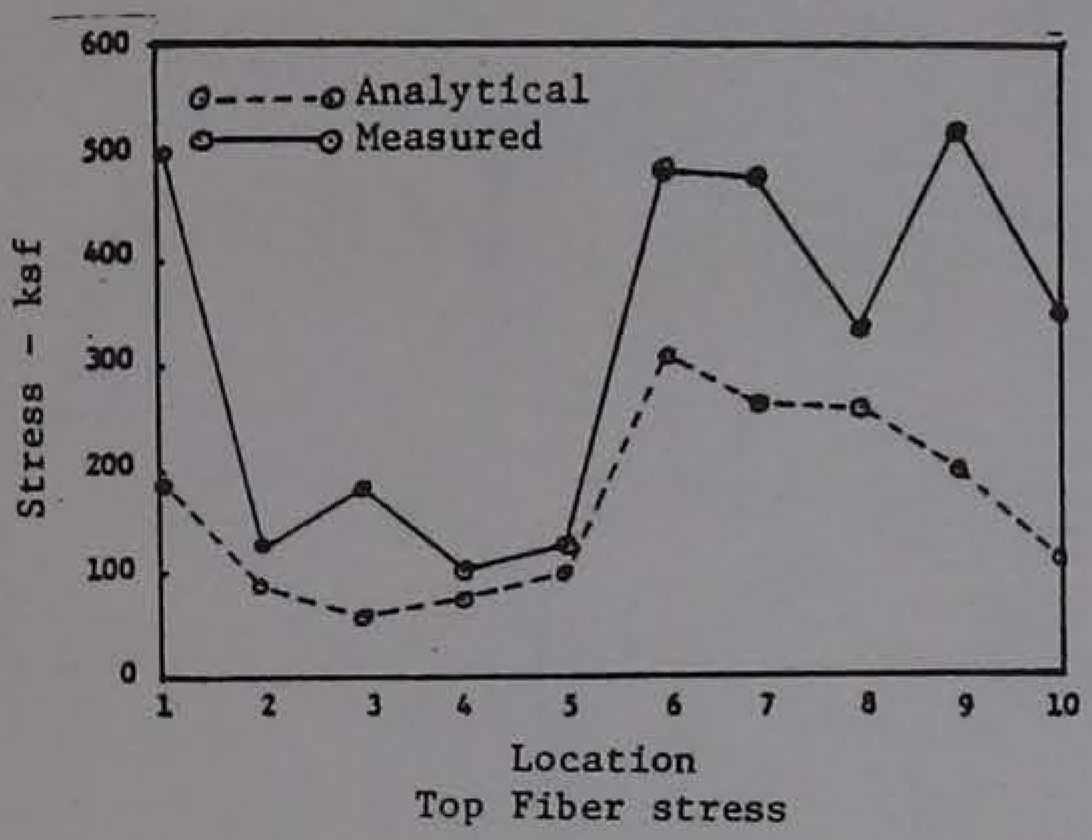

Figure 5. Stress After Final Cable Adjustments

Strain readings were made at random times during the day, often with large variations in the thermal gradients through the depth of the deck girder, resulting in considerable fluctuation in strain readings with temperature increase. With additional data and a more systematic approach to data collection, some of these fluctuations should be eliminated. Wind effects will continue to be a topic for investigation as well as thermal and time-dependent strains.

Presently, several of the locations are not operating properly because of lack of proper maintenance, but a proposal has been submitted to service the entire strain monitoring system and to reactivate it. An inspection has indicated that approximately 85% of the strain gages are still operable.

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SESSION IV

ASPECTS OF BRIDGE RESEARCH AT THE UNIVERSITY OF NEVADA-RENO

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SYNOPSIS

The purpose of this paper is to describe in summary form three bridge research projects currently underway at the University. The first project involves the full scale dynamic testing of the Meloland Road Overpass in El Centro California. The second project involves a study of the impact damage between bridge decks and abutments which is caused by earthquake motions. The two main parameters which influence this behavior are the abutment stiffness and the gap between the abutment and end of deck. Finally a project which involves the classically difficult problem of integrating measured accelerograms for reliable displacements is discussed.

MELOLAND ROAD OVERPASS PROJECT

Recently the University entered into an agreement with the California Department of Transportation (CALTRANS) and the National Science Foundation (NSF) to conduct an assessment of bridge seismic design and instrumentation requirements using full-scale dynamic testing and dynamic analysis of the Meloland Road Overcrossing. This bridge is in a very active earthquake zone in Southern California. It has an extensive permanent strong motion recording system which has recorded much significant earthquake response data. The Dames and Moore Company is a major subcontractor participating with the University on this work. The Dames and Moore effort is under the direction of Stewart Werner and C.B. Crouse.

The Meloland Road Overpass is a two span 208 ft. long reinforced concrete box girder bridge having monolithic abutments. The single central pier and the two abutments are supported with pile foundations. This bridge type represents a major fraction of the total bridge inventory in seismic regions of the west. In the 1979 Imperial Valley earthquake, the peak transverse acceleration of the bridge deck was 50% of gravity while the peak transverse acceleration at the nearby free field station was 30% of gravity.

Significantly, the bridge suffered no observable damage during this very strong shaking.

The dynamic field tests were conducted for this project over the period May 23-26, 1988. A single hydraulic ram at the central pier was used to induce quick-release motions simulating earthquake response. The bridge was completely instrumented in a sequential way so as to measure the transverse and rotational response of the bridge deck, the foundations, and abutments. In addition, measurements were made to estimate the extent to which the soils in the approach fills and central pier foundation participated in the response of the structure.

Shaking amplitudes achieved during the experiments were representative of earthquake caused motions. At the highest load levels, the peak transverse deck acceleration was about 200 cm/sec^2 or 20% of gravity. The peak vertical accelerations at the edges of the bridge deck during these experiments were about 30% of gravity. The next phase of the project will be to develop complete experimental dynamic mode shapes, natural frequencies, and modal damping ratios. Further, the release displacements of the structure caused by the applied loads will be calculated using methods described later this paper.

THE IMPACT BETWEEN THE BRIDGE DECK AND THE ABUTMENTS OF SHORT BRIDGES

To investigate the impact between the deck and the abutments of bridges a theoretical model was developed. Since the primary focus of the model was to apply to short bridges, a simple bridge structure was chosen for analysis. The system which was considered along with the model are shown in Fig. 1.

It consists of the bridge deck, seat type abutments, and a single row of piers. The bridge deck is represented by a rigid block and a rotational spring k_d. The rigid block has mass and mass moment of inertia properties which can be estimated from the geometric properties of the real bridge deck. The rotational spring, k_d, represents the resistance offered at the top of the pier against the rotation of the deck.

Each abutment is separated from the deck by a gap in the longitudinal direction. The abutment is represented by a longitudinal spring K_{ab}, which is allowed to yield at high displacement levels. The impact between the bridge deck and the abutment occurs when the longitudinal displacement of the deck exceeds the corresponding abutment gap. Since seat type abutments are considered in this analysis, the deck rests on the abutments on elastomeric pads. These pads are represented by springs. More details about the abutment and the elastomeric pad springs are provided in recent relevant studies [1].

The single row of piers is represented by a continuous bending beam. This beam can be either elastic or hysteretic. To model the hysteretic behavior of the beam the Q-HYST model was used [2]. At the bottom of the beam there are translational and rotational springs and dampers representing the flexibility and the radiation damping respectively at the foundation level. The foundation springs can be nonlinear accounting in this way for the material damping of the foundation.

The model is excited by an earthquake excitation in the longitudinal direction applied at the base. To determine the response of the model to an applied excitation, the finite element method is employed.

The model, which was briefly described above, was used to perform several parametric studies in order to examine the effects that the values of certain parameters have on the impact between the deck and the abutments. Two different records were used for the parametric studies: the first ten seconds of the 1940 El Centro earthquake (N-S component) and sixteen seconds of the 1952 Pasadena earthquake (S90W component) [1]. Some samples of the parametric studies are shown in Figs. 2-4. In Fig. 2, the effects that the abutment stiffness has on the response of the model for various values of the abutment gap can be seen. In this figure, the variable KR expresses the ratio between the abutment stiffness and the column stiffness of the bridge. One can see that as the abutment stiffness increases, the magnitude of each of the maximum responses decreases. Increases in the gap diminish the influence of the abutment stiffness. In Figs. 3 and 4, selected displacement time histories are presented showing the effects of the elastomeric pads and the hysteretic behavior of the columns respectively. One can see that at low displacement levels, when impact does not occur both the pads and the hysteretic behavior of the columns influence the response of the bridge deck. However, when impact occurs at higher displacement levels, the response of the model is controlled by the impact; and the effects of the pads and the hysteretic behavior of the columns are insignificant. More details about the results of the analysis are provided in recent relevant studies by Maragakis and Thornton [1].

INTEGRATION OF QUICK-RELEASE ACCELEROGRAMS

The calculation of displacement time histories by integrating measured acceleration records is a difficult problem which has been addressed by a number of researchers. Recent work in this field has been done by Iwan et al. [3].

The character of general accelerograms which makes the successful integration of these accelerograms for accurate displacement time histories so difficult, is that the transducer accelerometer produces a large signal for the high frequency content and a very small signal for the long period content. It is this low frequency information in the signal which principally influences the final baseline of the displacement time history. While the signal to noise ratio for the high frequency information may be excellent, the simultaneous signal to noise ratio for the long period part of the accelerogram will be much lower. This low signal to noise ratio can cause serious problems when the accelerograms are doubly integrated.

To illustrate the problem with a concrete example, the University of Nevada field digital data acquisition system is a 66db dynamic range Kinemetrics Dataseis. It has a 64 channel recording capability with 24 FBA11 force balance accelerometers. The full scale peak to peak signal is digitized with a resolution of 4096. Specifically, the least count for measuring the full scale signal is 1/4096 of full scale which is more than sufficient accuracy for most purposes. However, when attempting to integrate accelerograms to obtain displacements, an error of just one bit on the digitizer leads to significantly large errors in the integrated displacements. This is particularly true for long records. This is a problem that is especially acute with most of the older accelerographs. Obviously, modern digital recorders having a much wider dynamic range will do a much better job of measuring the long period content in the accelerogram and will thus be more integrable to displacements. It goes without saying, that the force balance accelerometer must also produce an equivalently precise measurement or a high accuracy digital recorder will simply record accelerometer noise, which when integrated will also produce huge errors.

For those cases where it is known that the displacement baseline contains no long period content, as for example many structural vibration applications, accelerograms can successfully be integrated for their high frequency displacement content by using low pass filter techniques. This low pass filter strategy is however not applicable to those cases where the final displacement record can have a baseline offset which takes place over time. Obviously, a low pass filter would strip off the long period baseline shift.

Structural vibration accelerograms produced by the quick release method of excitation are a special case where the final displacement baseline is offset by the amount induced by the quick-release load and is otherwise flat. In this special case the displacement baseline is a step function. In this situation the development of an integration algorithm was possible for the Dataseis system briefly described above. The method used was based on a generalization and an extension of earlier work by Brady [4], and produces surprisingly accurate results.

An example of this calculation is shown in Fig. 5. The upper quick release accelerogram was produced on a three story, three degree of freedom laboratory model in which both the time history of displacement and acceleration were measured. The bottom solid trace shows the displacement record integrated from the accelerogram. The dashed trace shows the displacement trace measured with an LVDT and recorded simultaneously on the Dataseis. As can be seen the agreement is excellent. A series of measurements obtained by varying all the relevant parameters indicates that the offset displacements can be predicted reliably to within an accuracy of 2 to 7 percent.

The significance of this work for dynamic testing of bridges by the quick release method is obvious. Reliable estimates of the bridge structural displacements caused by the quick-release loads can now be obtained from accelerometers which make their measurement using an inertial reference.

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ACKNOWLEDGEMENTS

Support for these projects provided by the NSF and CALTRANS is gratefully acknowledged. Further we especially want to thank our colleague Dr. Saiidi for his support, and our graduate students Mr. B. Nath and Mr. G. Thornton for their conscientious efforts.

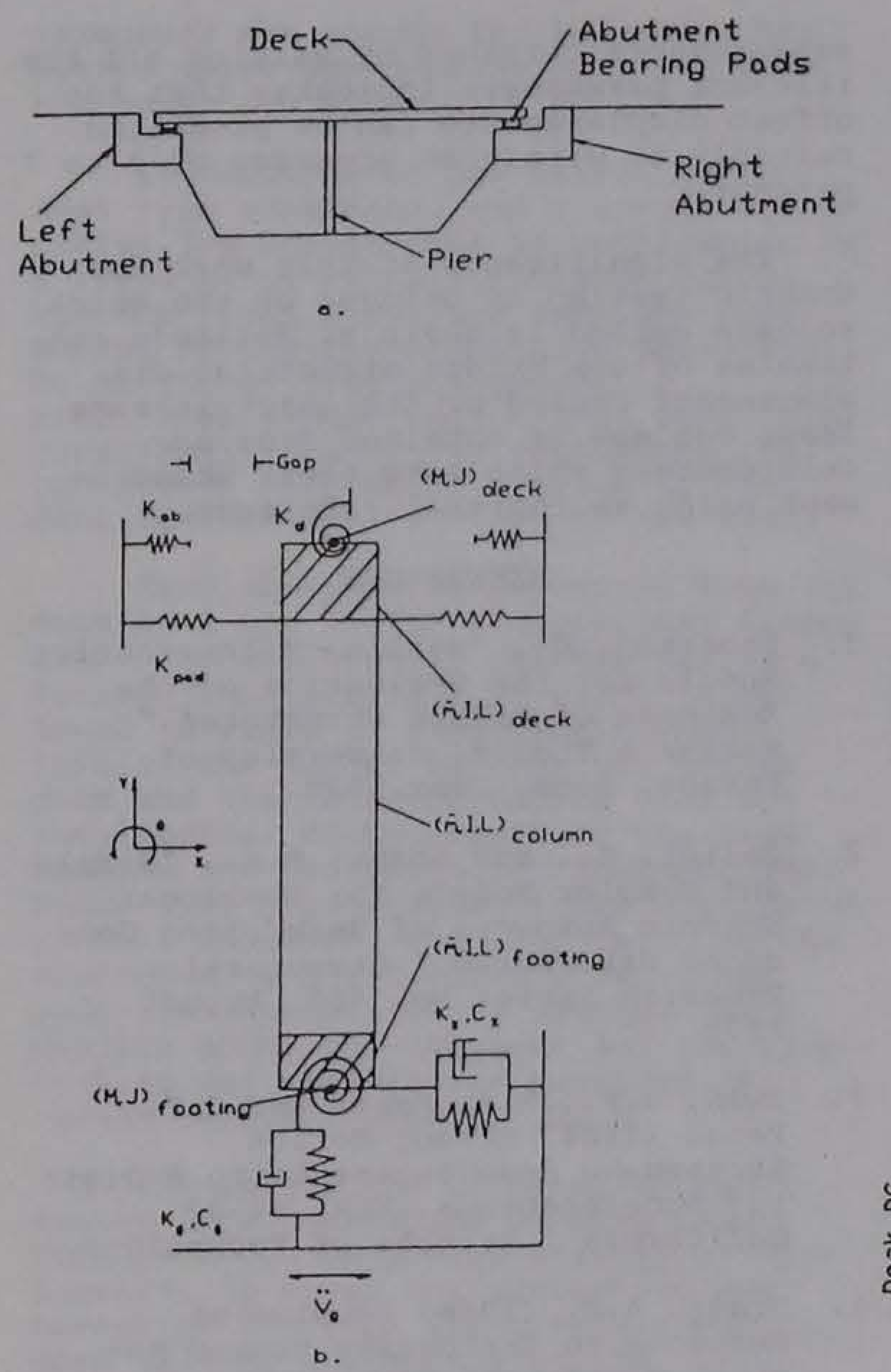

Fig. 1 Simplified Model of a Short Bridge Structure
a. Bridge Structure b. Model

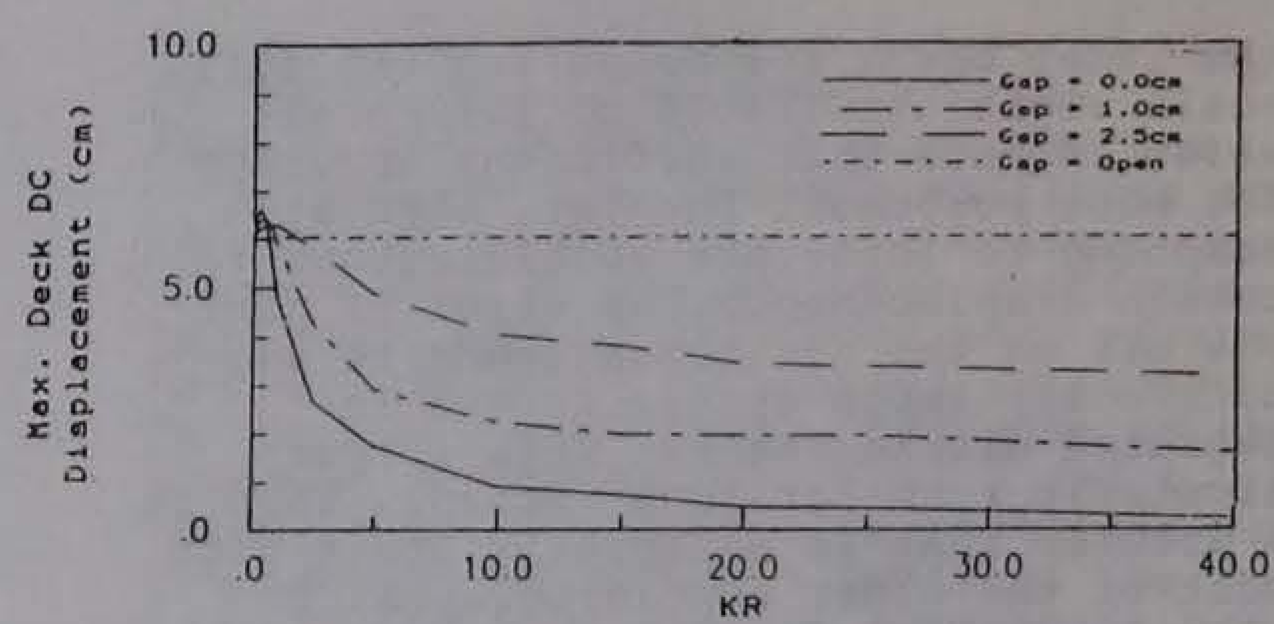

Fig. 2 Effects of the Abutment Stiffness on the Response of the Model

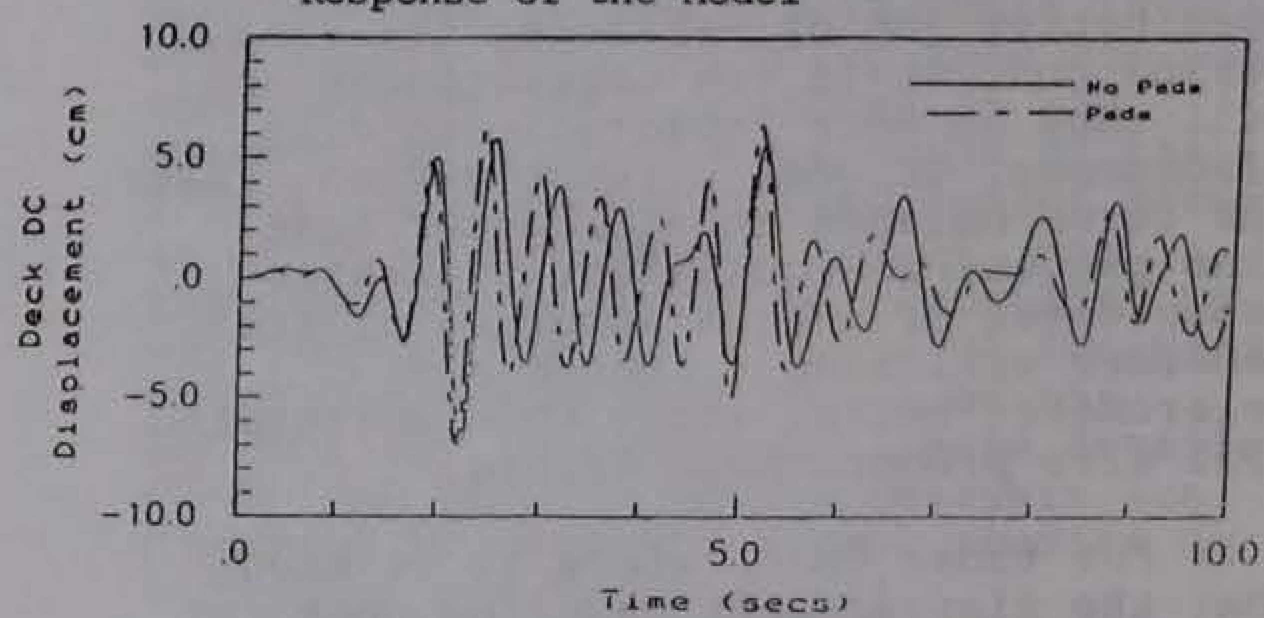

Fig. 3 Effects of the Abutment Bearing Pads on the Time History Displacement Response of the Deck

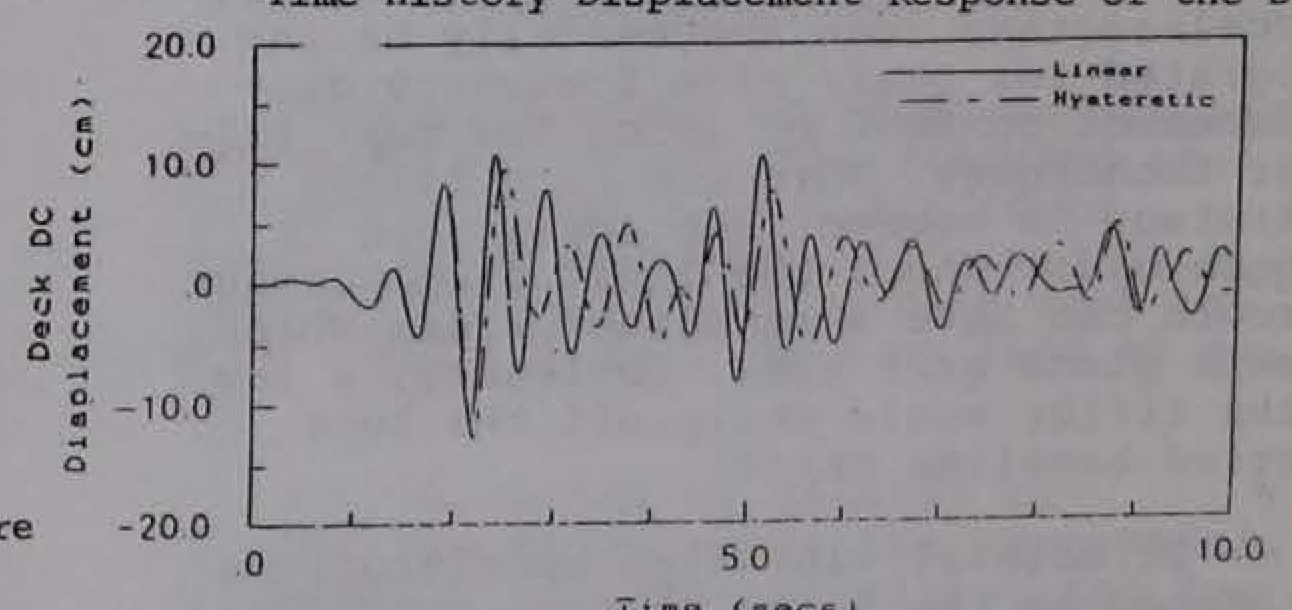

Fig. 4 Effect of the Hysteretic Behavior of the Piers on the Time History Response of the Deck

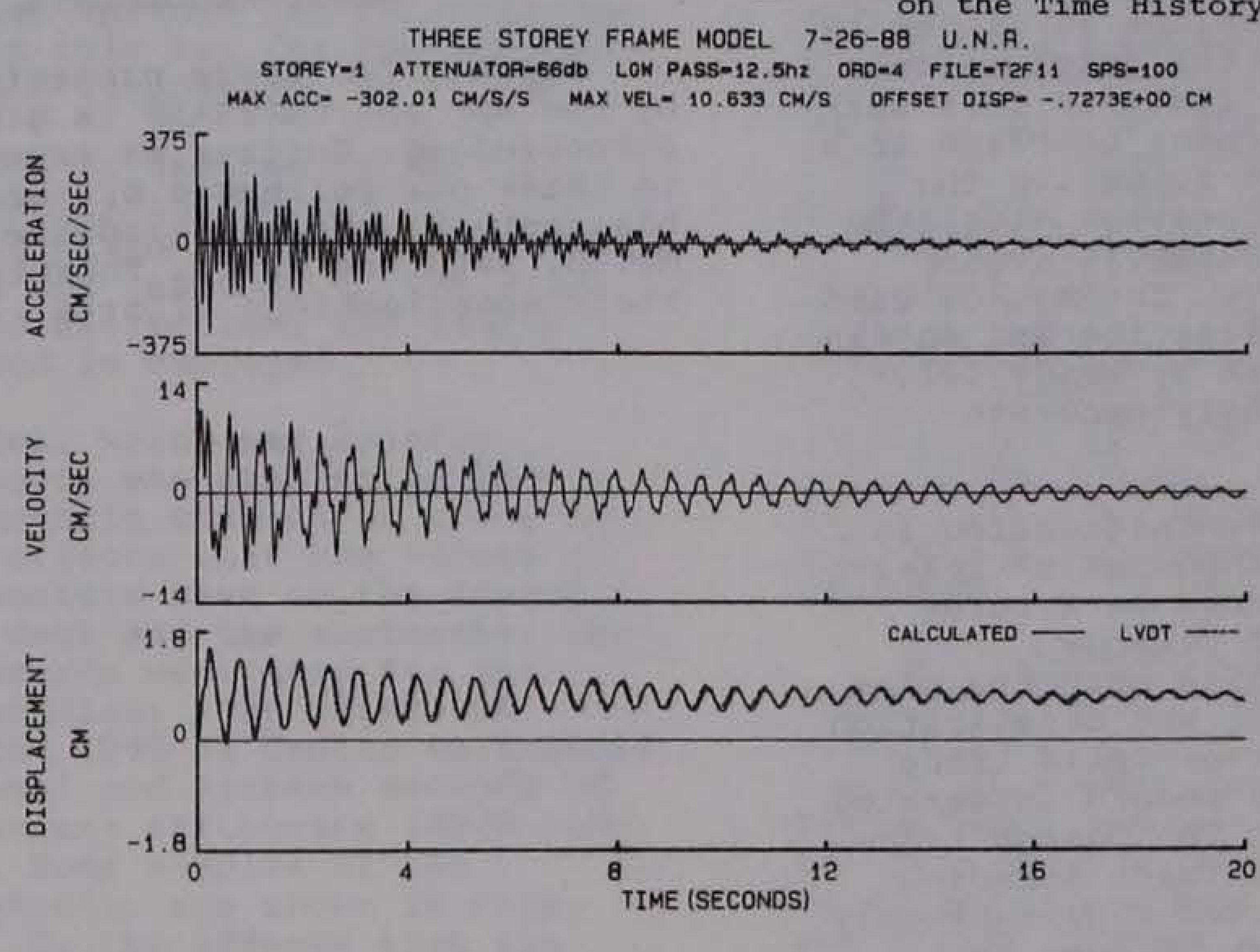

Fig. 5 Integrated Velocity and Displacement from Accelerogram

BRIDGE PERFORMANCE AND CONDITION PREDICTION MODEL FOR A BRIDGE MANAGEMENT SYSTEM

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SYNOPSIS

As part of a study to develop a comprehensive bridge management system for the Indiana Department of Highways (IDOH), a bridge performance analysis was performed. Performance of deck, superstructure and substructure of bridges were analyzed using the techniques of regression, the Markov chain, non-linear programming and combination of these techniques. The results exhibited the power of these techniques, particularly of the Markov chain approach in prediction or estimation of future bridge conditions. The approach, although simple, was found to provide a high level of accuracy in predicting bridge conditions

INTRODUCTION

As part of a study to develop a comprehensive bridge management system for the Indiana Department of Highways (IDOH), a bridge performance analysis was performed. Performance curves for deck, superstructure and substructure of bridges were developed using a regression technique. Bridge performance prediction models were also developed using the Markov chain. This paper briefly describes the development of performance curves and performance prediction models.

DATA BASE AND CLASSIFICATION FACTORS

Performance curves and transition matrices were eventually developed for combinations of the factors shown in Table 1. There were 12 factor combinations in total.

Table 1. Classification Factors

A. Highway System 1. Interstates 2. Other State Highways
B. Bridge Type 1. Concrete 2. Steel
C. Bridge Component 1. Deck 2. Superstructure 3. Substructure

Two other factors considered for analysis, traffic volume and climatic region, were not statistically significant. Each factor combination had 100 bridges for a regression analysis.

DEVELOPMENT OF PERFORMANCE CURVES

A third order polynomial model was chosen to perform the regression analysis, as shown below:

$$y_i = \beta_0 + \beta_1 x_i + \beta_2 x_i^2 + \beta_3 x_i^3 + \varepsilon_i \quad (1)$$

where, y_i is the condition rating, x_i is bridge age, and ε_i is the error term. For a new bridge (age 0), the condition rating is always 9, therefore, β_0 was specified to be 9 in order to make the intercept of a regression model an integer value and meaningful in practice. Regression functions for some of the factor combinations met the two aptness requirements, the constancy of variance and normality of residual distribution. For those that did not meet the aptness requirements, a transformation of the raw data was made to make the regression model appropriate. It was found that the transformation of $y = \sqrt{y}$ was most appropriate to meet the two requirements.

In total, twelve performance curves were developed. Figure 1 presents an example of these curves.

MARKOV CHAIN APPROACH

The Markov chain, as applied to bridge performance prediction, was based on the concept of defining states in terms of bridge condition ratings and obtaining the probabilities of the bridge condition transiting from one state to another. Ten bridge condition ratings defined by the FHWA [1] were used as ten states, with each condition rating corresponding to one of the states. For example, condition rating 9 is defined as state 1, rating 8 as state 2, and so on. Without repair or rehabilitation, the bridge condition rating either remains the same or decreases as the bridge age increases. Therefore, it was considered that a probability of condition transiting from one state, say i, to another state, j, during a given period of time would exist and such probability was denoted by $P_{i,j}$.

Let the transition probability matrix of the Markov chain be P, given by

$$P = \begin{vmatrix} P_{1,1} & P_{1,2} & \cdots & P_{1,10} \\ P_{2,1} & P_{2,2} & \cdots & P_{2,10} \\ \cdot & \cdot & \cdots & \cdot \\ \cdot & \cdot & \cdots & \cdot \\ \cdot & \cdot & \cdots & \cdot \\ P_{10,1} & P_{10,2} & \cdots & P_{10,10} \end{vmatrix} \quad (2)$$

Then the state vector for any time T, $Q_{(T)}$, can be obtained by the multiplication of initial state vector $Q_{(0)}$ and the Tth power of the transition probability matrix P:

$$Q_{(T)} = Q_{(0)}*P*P*\ldots*P = Q_{(0)}*P^T \quad (3)$$

TRANSITION PROBABILITY MATRIX

In developing transition matrices, it was assumed that condition rating would either remain the same or change to a lower number in the two year rating period. That is, a bridge condition rating would monotonically decrease as the bridge age increases. Therefore, the probability $P_{i,j}$ is null for $i > j$, where i and j represent the states in the Markov chain.

Prediction of the Percentages of Bridges in Certain Condition Ratings

From the data base, the number of transitions of bridge conditions from one state to another state was obtained. Let $n_{i,j}$ denote the number of transitions from state i to state j within one time period, then the number of bridges in state i before the transition can be defined as n_i:

$$n_i = \sum_j n_{i,j} \quad (4)$$

It was proved [2] that the estimated transition probability would be

$$P_{i,j} = \frac{n_{i,j}}{n_i} \quad (5)$$

Using this relationship, a transition matrix was determined and the prediction was performed by using Equation 3. Since bridges are inspected every two years, a two year transition period was used. That is, $p_{i,j}$ was the probability of condition transiting from state i to state j in a two year period. Figure 2 shows a comparison of the predicted values and the actual percentages for deck conditions recorded in 1986. The prediction was made by using the actual percentages for deck conditions recorded in 1978 as the initial state vector. As can be seen from Figure 2, the predicted percentages of bridges at each condition rating level were very close to the percentage splits obtained by actual recorded values.

Prediction of Bridge Conditions

The transition matrix for percentage prediction can not be used to predict bridge conditions at a given age, because this matrix is independent of bridge age. In other words, this matrix is not homogeneous with respect to bridge age. A Markov process, however, requires the assumption of homogeneity [2]. To avoid overestimating or underestimating the bridge condition, a different approach, named a zoning technique, was used to estimate transition matrices at different periods of a bridge service life. This approach was used for the development of pavement performance curves in a recent study [3]. Different from percentage prediction, one year transition period was used in developing performance curve. Bridge age was divided into groups and within each age group the Markov chain was assumed to be homogeneous. A six year group was used, and each group had its own transition matrix that was different from those of remaining groups. To make the computations simple, an assumption was made that the bridge condition rating would not drop by more than one state in a single year. The transition matrix of condition ratings had, therefore, the following form:

$$P = \begin{vmatrix} p(1) & q(1) & 0 & 0 & 0 & 0 & 0 \\ 0 & p(2) & q(2) & 0 & 0 & 0 & 0 \\ 0 & 0 & p(3) & q(3) & 0 & 0 & 0 \\ 0 & 0 & 0 & p(4) & q(4) & 0 & 0 \\ 0 & 0 & 0 & 0 & p(5) & q(5) & 0 \\ 0 & 0 & 0 & 0 & 0 & p(6) & q(6) \\ 0 & 0 & 0 & 0 & 0 & 0 & 1 \end{vmatrix} \quad (6)$$

where $q(i) = 1 - p(i)$. It should be noted that the lowest recorded rating number in the data base was 3, indicating that the bridges are usually repaired or replaced at rating not less than 3. Consequently, the corresponding state 7 has the transition probability $p(7) = 1$.

To estimate the transition matrix probabilities, for each age group the following non-linear programming objective function was formulated:

$$\min \sum_{t=1}^{N} |Y(t) - E(t,P)| \qquad (7)$$

subject to: $0 \leq p(i) \leq 1, \quad i=1, 2, \ldots I$

where, N = the number of years in one age group,

I = the number of unknown probabilities,

$P = [p(1), p(2), \ldots, p(I)]$, a vector of length I,

$Y(t)$ = the average of condition ratings at time t,

$E(t,P)$ = Estimated value of condition rating by Markov chain at time t.

The solution to this function was obtained by the Quasi-Newton method [4]. The maximum rating of bridge condition was 9 and it represented a near-perfect condition of a bridge component. It is almost always true that a new bridge has a condition rating 9 for all of its deck, superstructure and substructure. In other words, a bridge at age 0 has condition rating 9 for its components with unit probability. Thus, the initial state vector $Q_{(0)}$ for deck, superstructure or substructure of a new bridge is always [1, 0, 0,...,0], where the numbers are the probabilities of having condition rating of 9, 8, 7, ..., and 0 at age 0, respectively. That is, the initial vector of the first group (age 6 or less) for developing the bridge performance curve is known. Group 2 (age 7 to 12) takes the last state vector of group 1 as its starting state vector. In general, group n takes the last state vector of group n-1 as its starting state vector. Figure 3 compares the predicted deck conditions obtained by the Markov process and the average deck conditions by the regression method for concrete bridges.

CONCLUSIONS

The performance curves obtained in this study provided the measures of effectiveness of bridge improvement alternatives for an optimization model. The Markov chain was used to develop a procedure to estimate future bridge conditions. The procedure was simple and gave highly accurate condition predictions [5]. Results obtained by the Markov chain model can be used for a dynamic programming to achieve an optimal programming of bridge improvement projects.

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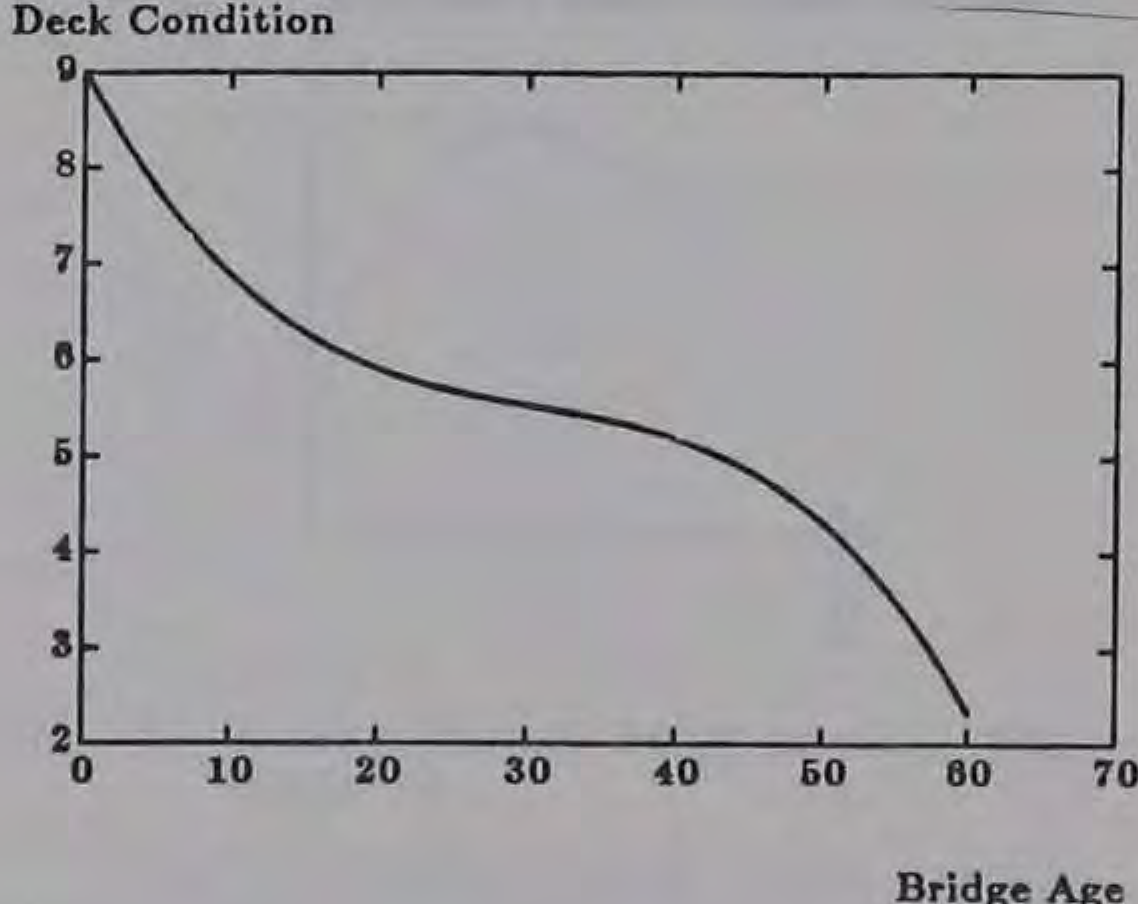

Figure 1. An Example Performance Curve

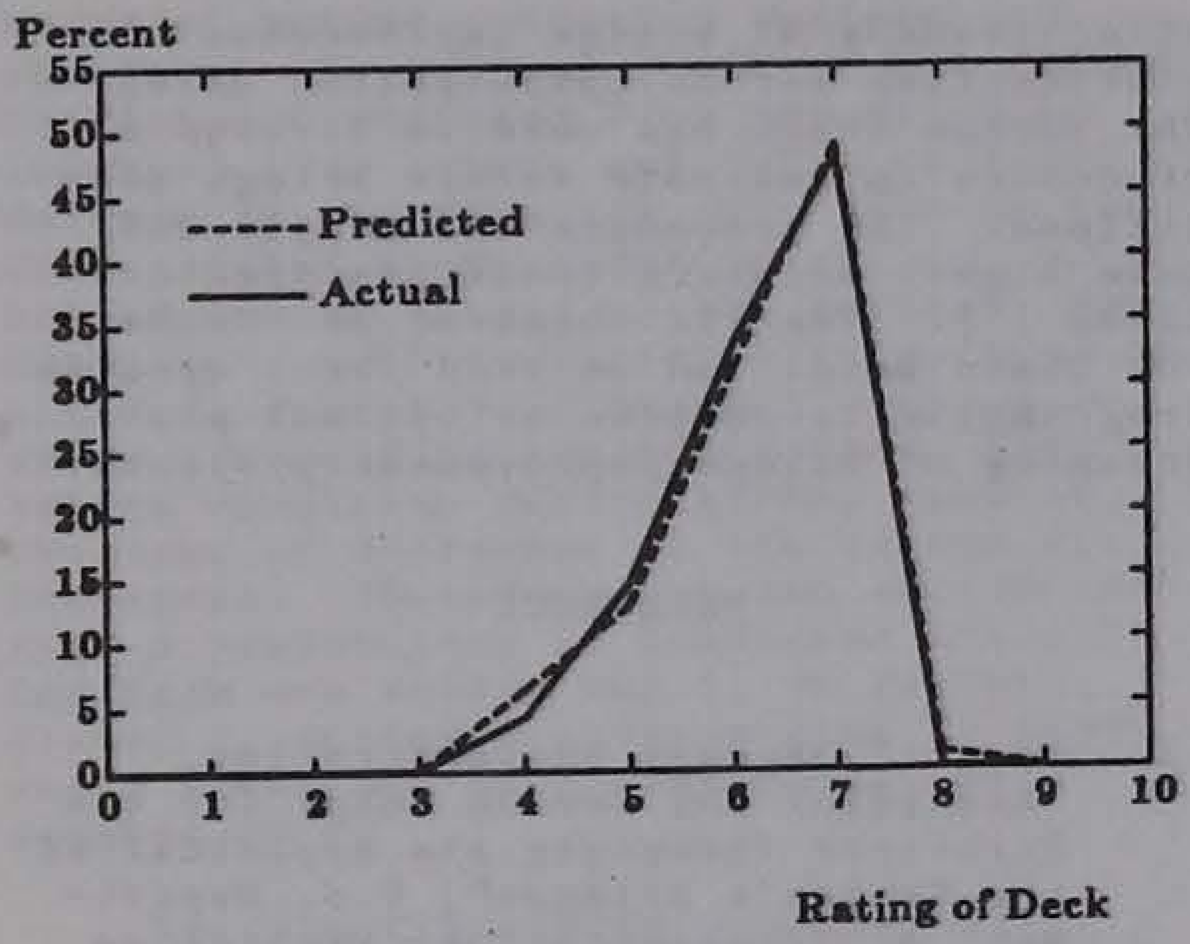

Figure 2. Percentage Prediction

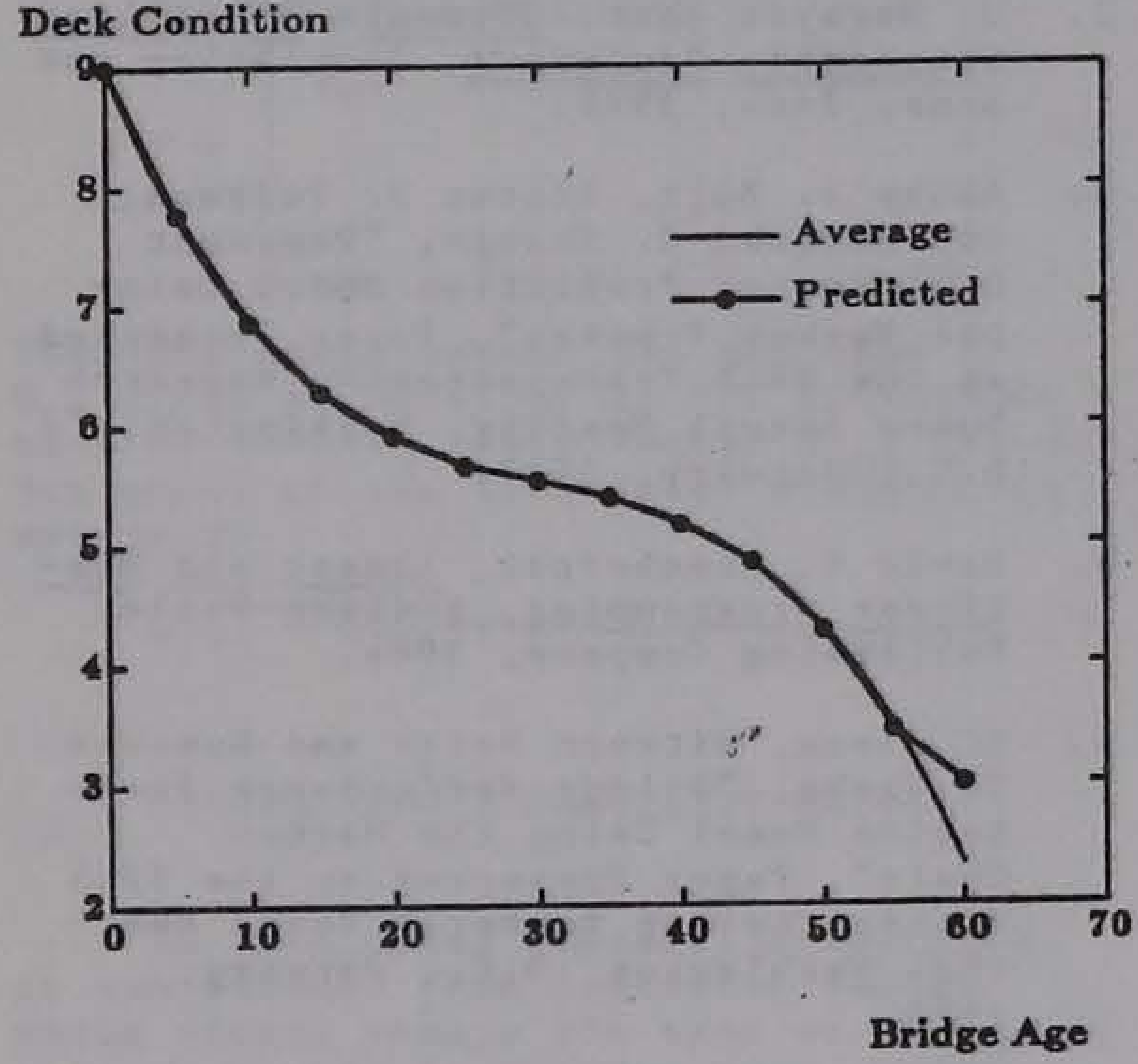

Figure 3. Markov Chain Predictions

BRIDGE RESISTANCE MODELS

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SYNOPSIS

The study deals with quantification of uncertainties in bridge resistance. Three factors are considered: material strength, fabrication (dimensions) and professional or analysis. The available statistical data is used to simulate the moment-curvature relationships for typical girder bridges (composite steel, reinforced concrete and prestressed concrete). The basic parameters (bias factor and coefficient of variation) are calculated for the considered bridge types and typical spans. The results can be used for the development of probability-based design and evaluation criteria for bridges.

INTRODUCTION

The bridge design and evaluation criteria are developed using load and resistance models [1,2]. The parameters of load and resistance involve a considerable degree of uncertainty. The objective of this study is to develop rational structural resistance models for the probability-based design (LRFD) and evaluation criteria.

In conventional reliability studies the structural resistance, R, is related to three factors:

$$R = (MFP)R_n \tag{1}$$

where R_n is the nominal resistance, M is material strength, F is fabrication factor (dimensions and geometry) and P is professional or analysis factor. The latter varies depending on structural system models and method of analysis.

The resistance of the bridge is a function of resistances of components and connections. The parameters of resistance are different for newly designed members and existing members. Since highway bridge loading is a repeated load spectrum - a low cycle damage model is needed to determine progressive damage to resistance and stiffnesses.

The parameters determining structural resistance (material properties, dimensions, geometry, analytical model) are random variables. The statistical data (distribution functions, means, coefficients of variation, coefficients of correlation, time variation, deterioration rate and so on) can be derived from tests, measurements, surveys, observations or analysis.

Extensive tests of material properties and member behavior were performed for building structures. However, less data is available for bridges. Therefore, special analytical procedures are developed to obtain resistance models for bridges.

The basic material test data is available. Examples of idealized stress-strain curves for concrete and steel, used in simulations, are given in Fig. 1 and 2, respectively.

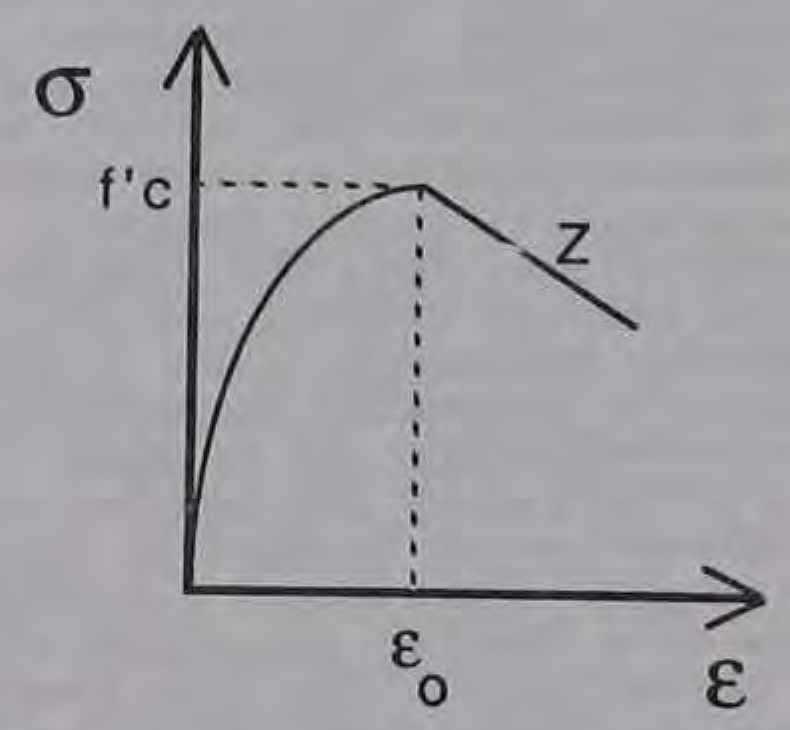

Fig. 1 Stress-Strain Relationship for Concrete.

The statistical data (mean values, bias factors and coefficients of variation, V) for material strength (compressive strength for concrete and yield stress for steel) are shown in Table 1.

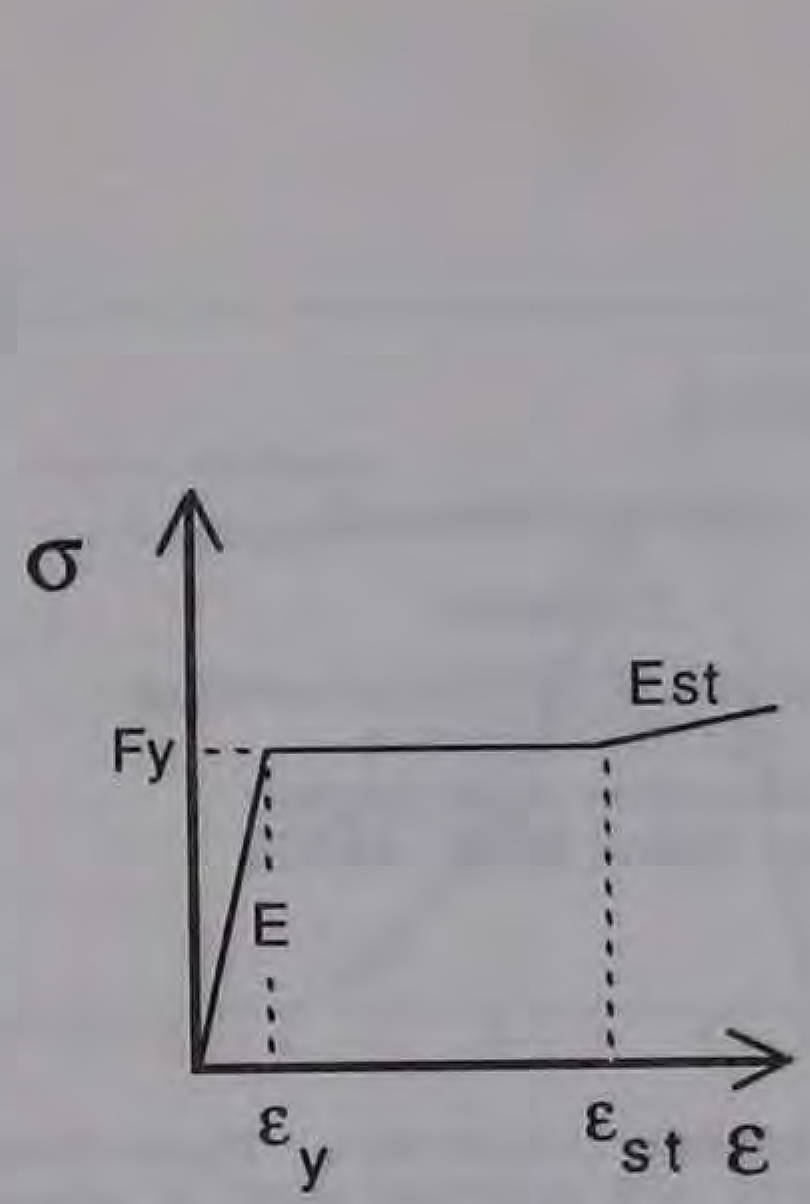

Fig. 2 Stress-Strain Relationship for Steel.

Table 1 Parameters of Steel and Concrete

Item	Mean value	Bias factor	V
Concrete:			
f_c'= 3000psi	2760psi	0.85	0.18
f_c'= 4000psi	3390psi	0.85	0.18
f_c'= 5000psi	4028psi	0.80	0.15
Steel:			
grade 40	45.3ksi	1.13	0.116
grade 60	67.5ksi	1.125	0.098
grade 270 (prestr.)	281.0ksi	1.04	0.025

Little data is available about the variation of dimensions in bridge structures. Therefore, the variation model is developed by combining the available data with other observations and judgement. Dimension tolerances in bridges are similar to those in buildings. In the result, the absolute values of departures from nominal dimensions are similar to the values observed in buildings. Yet, the size of bridge girders usually exceeds the size of building members. Therefore, even if standard deviations of dimension parameters were similar in buildings and bridges, the coefficients of variation are smaller for bridges. Construction of bridges is usually carried out by more experienced companies, which results in a further reduction of the coefficient of variation. The bias factor for dimensions can be assumed equal to 1.0, and the coeffiecient of variation from 0.015 to 0.025.

Resistance models were developed for composite steel girders, reinforced concrete girders and prestressed concrete girders. The resulting statistical parameters are presented in figures and tables.

FAILURE MECHANISMS IN GIRDER BRIDGES

A bridge fails when it cannot perform its function (carry traffic) any longer. Failure may take various forms, from problems resulting in restricted serviceability to the overall collapse. Indications of failure may include cracking, excessive deformations, kinks, corrosion, buckling (local or overall), rupture, and so on. A failure of a single element (or component) of a structural system may not necessarily result in total damage to the structure. As the loading applied to a structure is increased, damage will initiate at some load level and the structure will go through a spectrum of increasing damage states until total damage occurs at the ultimate load. The ratio of the total damage load to the damage initiation load is a measure of the system ductility. Structural ductility results from element ductility and system redundancy. Bridges are subject to cyclic loads (heavy trucks). Each occurrence of a heavy load may contribute to permanent deformations and it may affect load-deformation relationship in the proceeding load cycles. Accumulated damage may reduce the load carrying capacity of the bridge.

To evaluate the bending behavior of a bridge girder, the section can be treated as a collection of composite segments, as shown in Fig. 3. Each segment can represent

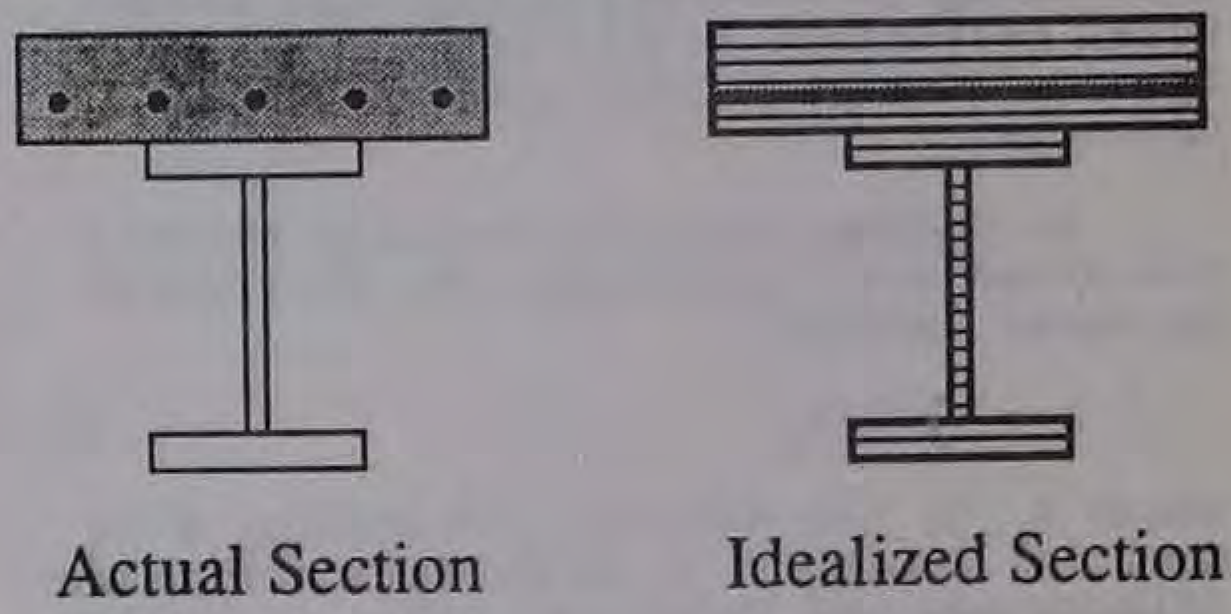

Fig. 3 Analytical Model of a Girder.

a different material and is made up of several layers. Using stress-strain relationship for each material, the stress level can be calculated at each layer, as shown in Fig. 4. By iteratively finding the correct stress distribution for each level of moment, the moment-curvature relationship can be established. The moment-curvature relationship varies because of random variation of section parameters. A numerical procedure was developed based on Monte Carlo technique which simulates the behavior of a grider [3].

The ultimate limit state (bending, shear, torsion, buckling) is determined by the load carrying capacity. However, the bridge performance is often governed by serviceability limit states, such as crack-

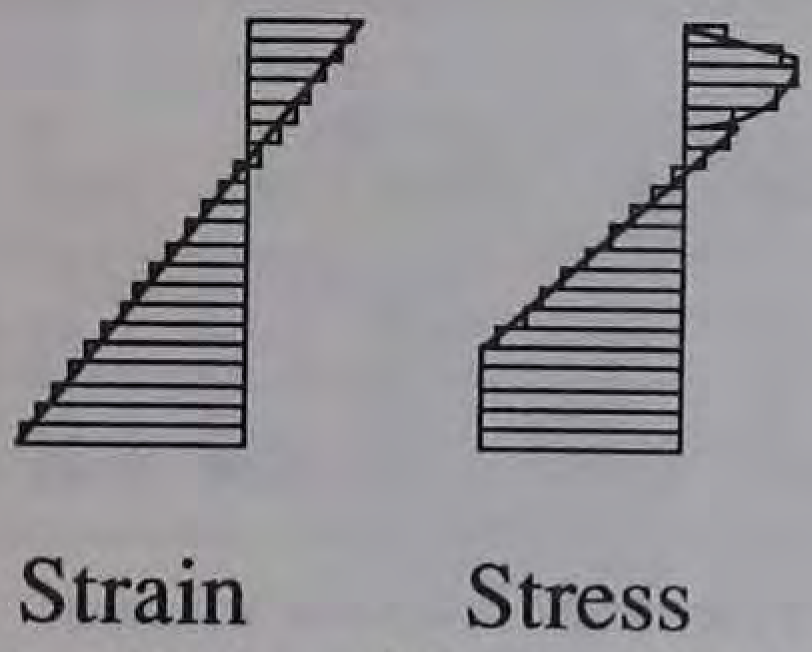

Fig. 4 Stress and Strain Distribution in the Idealized Section.

ing, deflection, vibration or by fatigue. The main concern is accumulation of the damage caused by repeated applications of load (truck axles). Therefore, it is important to consider not only the load magnitude but also frequency of occurrence.

PRESTRESSED CONCRETE GIRDERS

The major parameters determing the structural behavior include: moduli of elasticity of steel and concrete, strength of steel (prestressing and reinforcing), stength of concrete, and dimensions. In the analysis the following assumptions were made: perfect bond between steel and concrete, full composite action and crack does not propagate into slab under service loading.

For girder shown in Fig. 5 the generated family of moment-curvature curves is presented in Fig. 6.

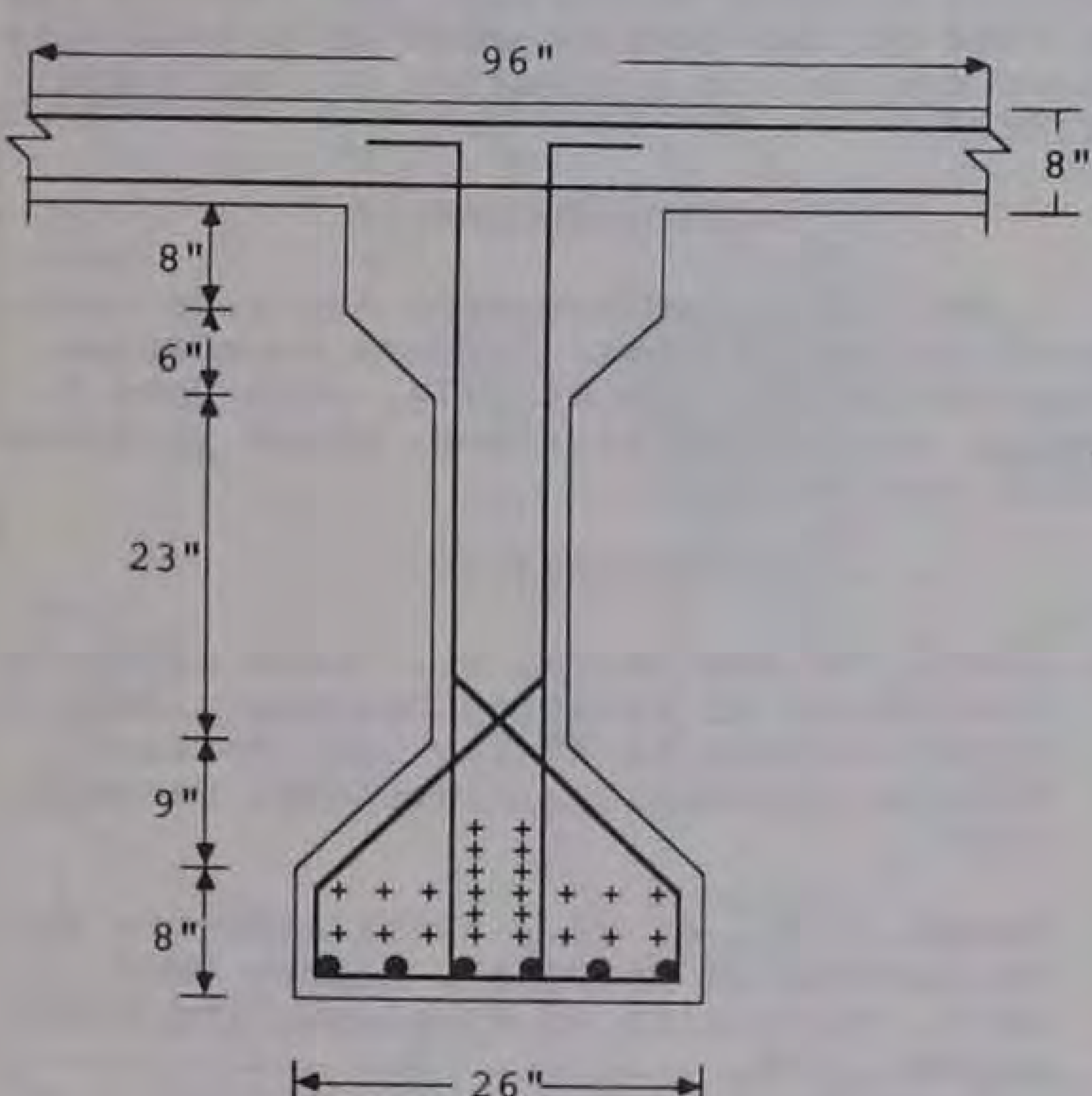

Fig. 5 Cross-Section of a Prestressed Concrete Girder.

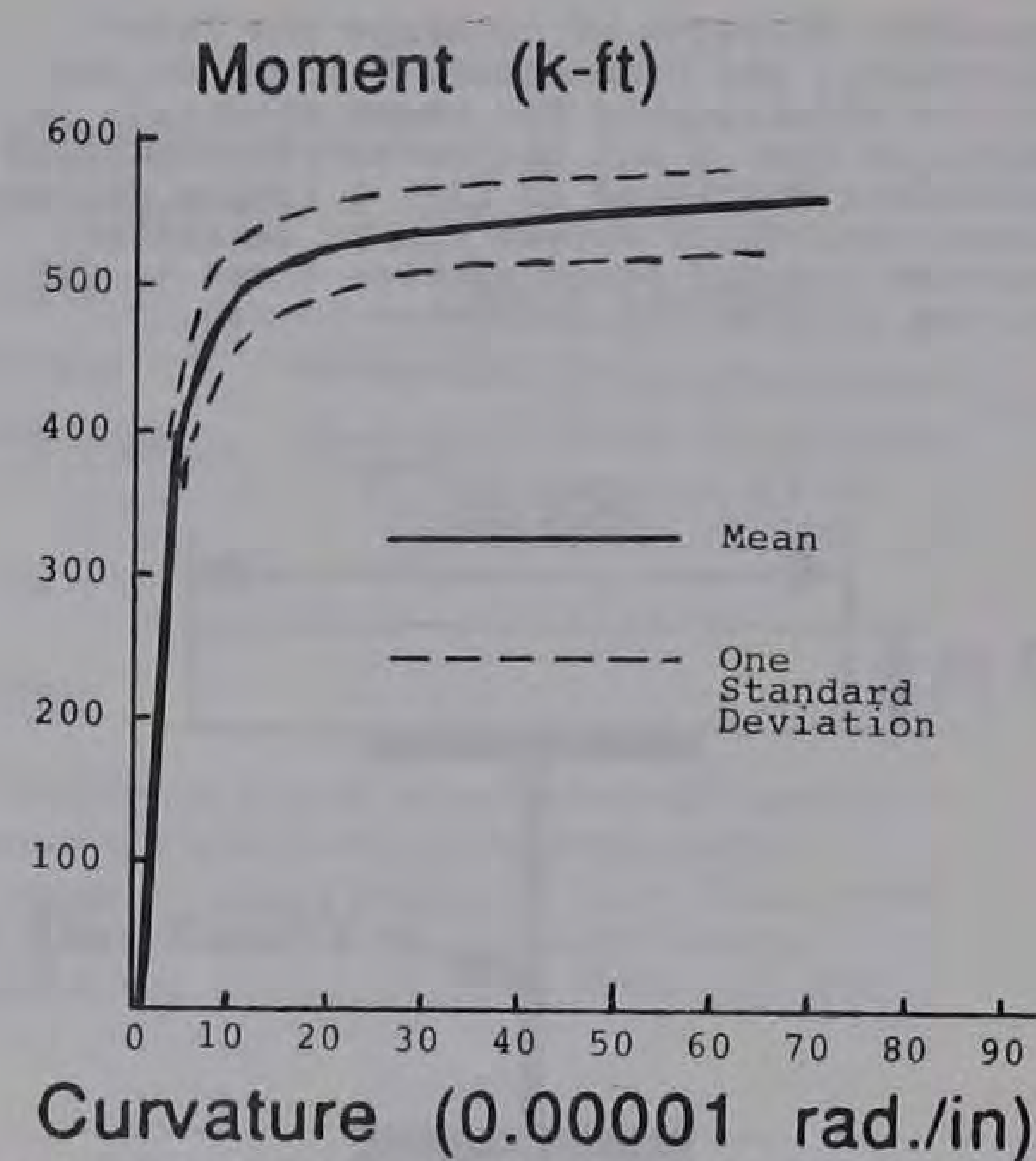

Fig. 6 Moment-Curvature Relationships for Prestressed Concrete Girders.

The resistance parameters (bias factors and coefficients of variation) for prestressed concrete girders are presented in Table 2. In Tables 3, 4 and 5, FM represents two parameters, M and F.

Table 3 Resistance Parameters for P/C

Span	FM bias	FM V	P bias	P V	R bias	R V
40ft	1.05	0.037	1.00	0.046	1.05	0.06
60ft	1.04	0.036	1.00	0.046	1.04	0.06
80ft	1.05	0.037	1.00	0.046	1.05	0.06
100ft	1.05	0.040	1.00	0.046	1.05	0.06

In the analysis of the whole bridge, the behavior of each girder is characterized by a curve (Fig. 4). The moment-curvature relationship can be approximated by three straight line components, corresponding to three stiffnesses: uncracked section, cracked section and ultimate load section. The three stiffnesses correspond to the stages of behavior under increasing loads: cracking moment, decompression moment and ultimate moment. For a typical bridge girder the section cracks for the first time under about 1.15 of the decompression moment, and the ultimate moment is about twice the decompression moment.

COMPOSITE STEEL GIRDERS

The behavior of composite steel girders depends on strength of steel (yield stress), strength of concrete and reinforcement, and dimensions. Typical stress-strain relationship for these materials is shown in Fig. 1 and 2. Using the numerical procedure described in [3], a family of moment-curvature curves can be generated. For the typical cross section shown in Fig. 7, the results are presented in Fig. 8.

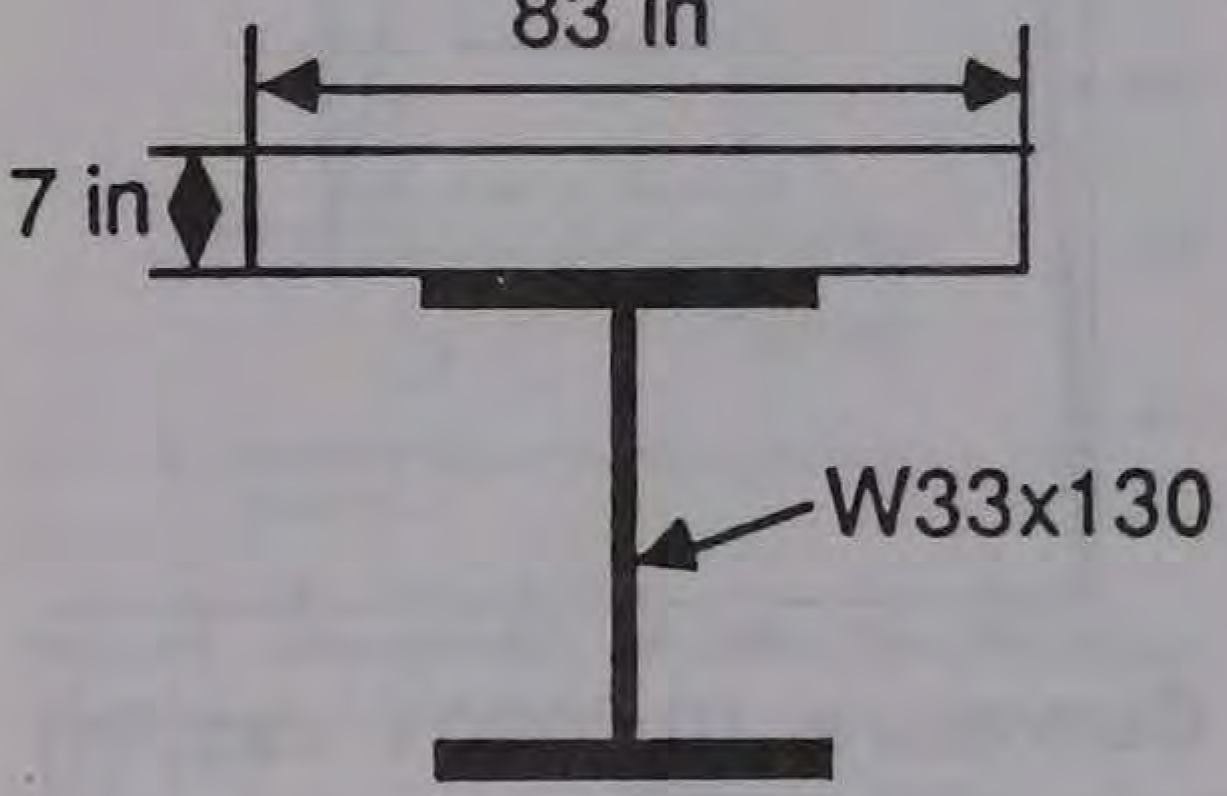

Fig. 7 Cross-Section of a Composite Steel Girder.

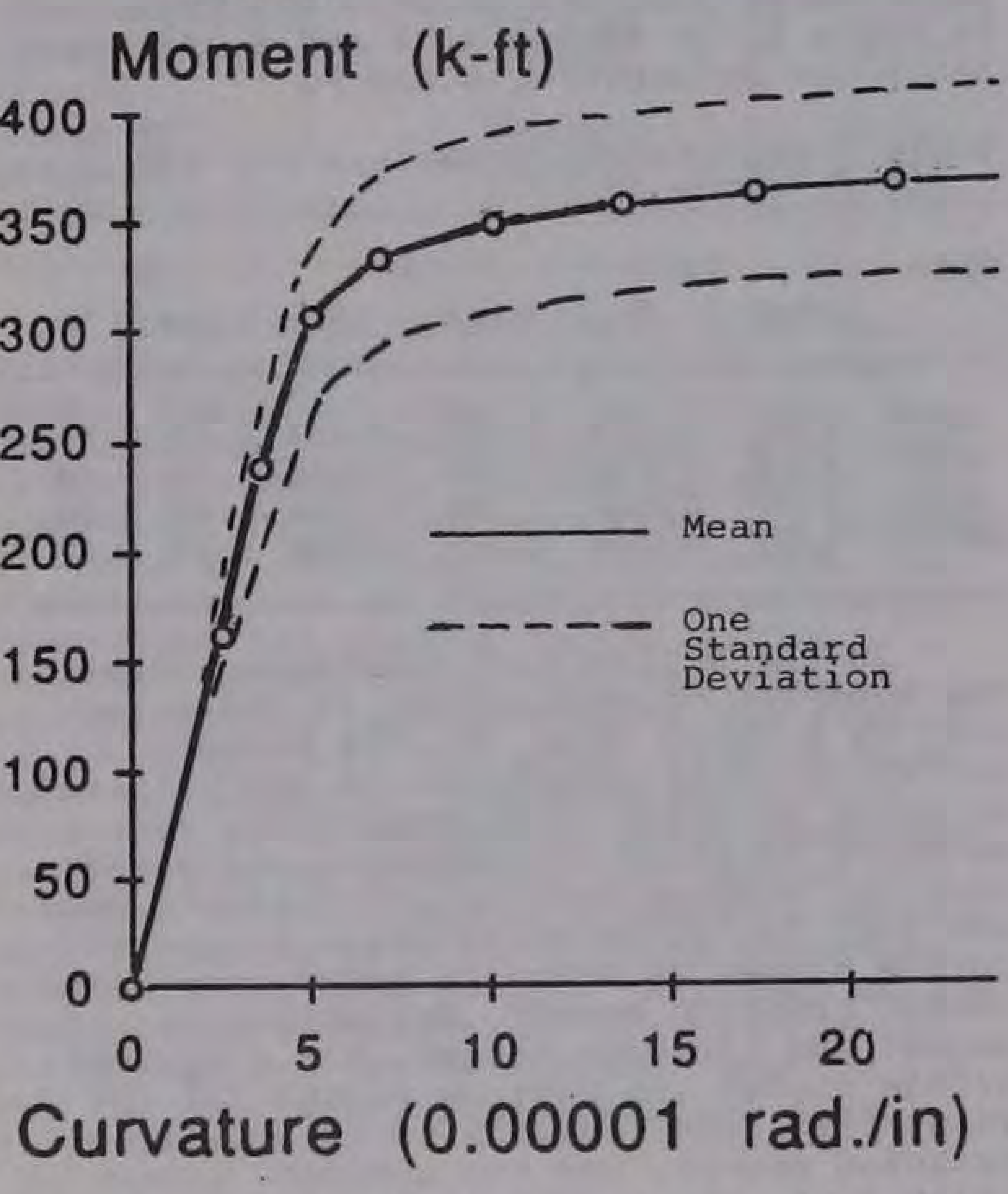

Fig. 8 Moment-Curvature Relationships for Composite Steel Girders.

The resistance parameters for composite steel girders are presented in Table 3.

Table 3 Resistance Parameters for Steel

Span	FM bias	FM V	P bias	P V	R bias	R V
40ft	1.03	0.089	0.99	0.08	1.02	0.12
60ft	1.02	0.0902	0.99	0.08	1.01	0.12
80ft	1.02	0.0866	0.99	0.08	1.01	0.12
100ft	1.02	0.0864	0.99	0.08	1.01	0.12

REINFORCED CONCRETE GIRDERS

The resistance parameters for reinforced concrete girders are presented in Table 4.

Table 4 Resistance Parameters for R/C

Span	FM bias	FM V	P bias	P V	R bias	R V
40ft	1.24	0.095	1.00	0.046	1.24	0.11
60ft	1.20	0.105	1.00	0.046	1.20	0.11
80ft	1.23	0.106	1.00	0.046	1.23	0.12

CONCLUSIONS

Statistical models are developed for bridge girders, in particular composite steel, reinforced concrete and prestressed concrete girders. The moment-curvature relationships are simulated using Monte Carlo technique. The basic parameters, bias factors (mean-to-nominal ratios) and coefficients of variation, are calculated for typical girders and spans. The results can be used in the developement of probability-based design and evaluation criteria for bridges.

ACKNOWLEDGEMENTS

The presented research has been sponsored by the National Science Foundation under Grant No. ECE-8413274, with John B. Scalzi as Program Director, which is gratefully acknowledged.

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DEVELOPMENT OF DESIGN RESPONSE SPECTRA FOR WESTERN WASHINGTON

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SYNOPSIS

Seismic guidelines currently used in Western Washington do not reflect the unique geology and seismicity of the area. This research will produce site-specific design response spectra appropriate for this area that will be used in the seismic design of highway bridges. Base spectra and soil amplification factors will be developed that are correlated with a mapped severity coefficient. The result is intended to replace corresponding sections in the currently used ATC-6 codes.

INTRODUCTION

Western Washington is a complex geotectonic province with a history of significant earthquake activity. Two recent earthquakes (1949, magnitude 7.1; 1965, magnitude 6.5) caused considerable structural damage in the highly-populated Puget Sound basin. Most of the destructive earthquakes have occurred at large focal depths and are associated with the subduction of the Juan de Fuca plate (see Fig. 1). It appears that the subduction process is still active and the possibility of a great earthquake (magnitude greater than 8) has been suggested.

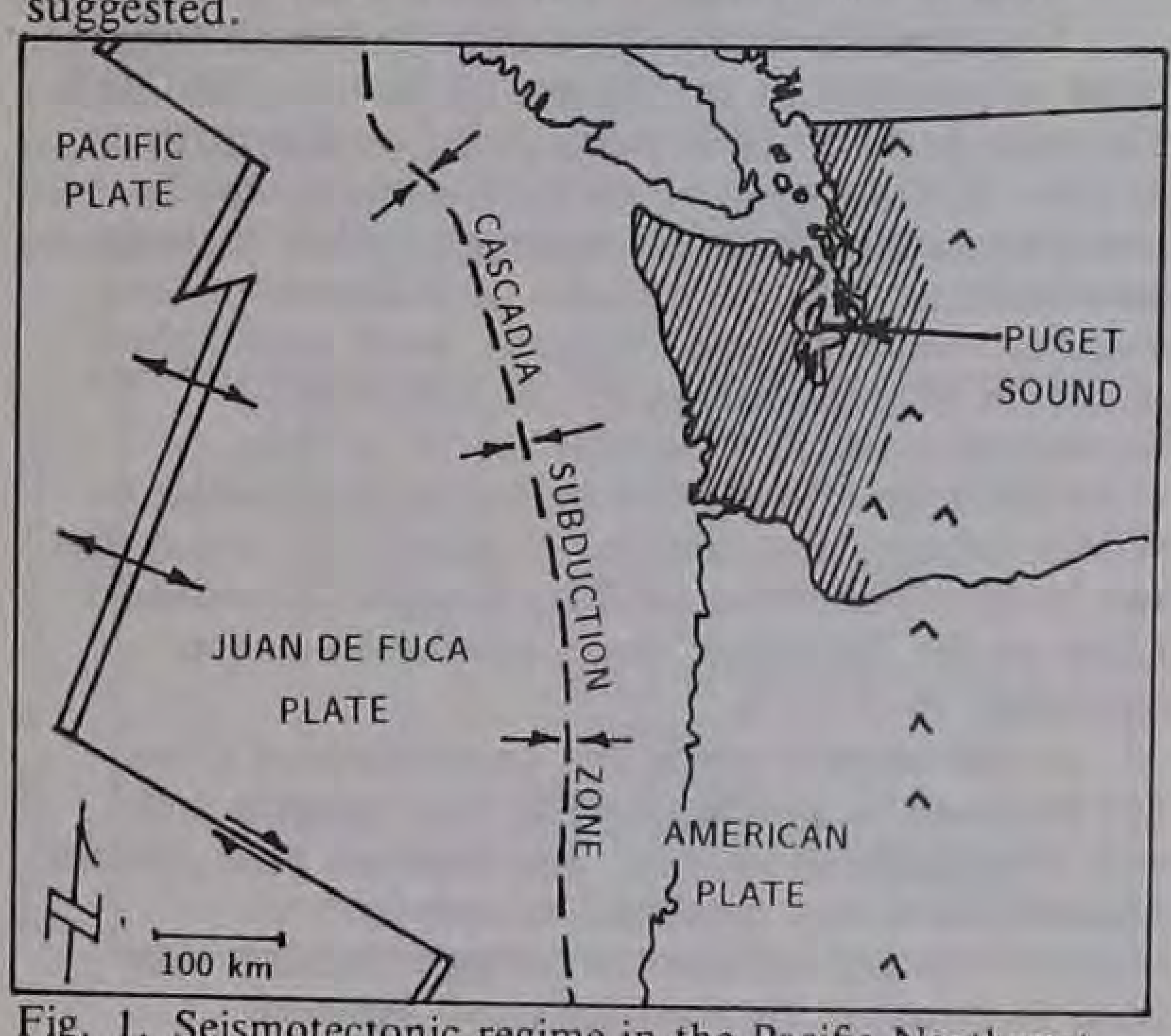

Fig. 1. Seismotectonic regime in the Pacific Northwest with arrows showing relative plate movement. Shaded area indicates study area and inverted v's show Quarternary volcanoes.

There is a vital need for rational seismic design of highway bridges in this region. Accurate representation of earthquake forces is required for economic as well as safety reasons since seismic forces frequently control bridge design. The Washington State Department of Transportation (WSDOT) is currently using AASHTO's 1983 ATC-6 seismic guidelines [1]. These guidelines were developed for general U.S. use and are based on research relying largely on data from California earthquakes. Since earthquakes occurring in Washington State are significantly different from those in California in terms of source mechanism, wave propagation paths, and site geology, it is essential to develop guidelines specifically for western Washington.

The present research addresses this need by developing design response spectra which reflect the unique seismicity and geology of the region. Base spectra and soil amplification factors will be developed that are correlated with a mapped severity coefficient. The result is intended to replace corresponding sections of the ATC-6 codes.

BACKGROUND

Many parameters affect ground shaking at any particular site. These parameters can be naturally divided into source, path and site categories. Source parameters include energy released (or magnitude), focal depth, and direction and dimensions of faulting. Path parameters include the epicentral distance and the geometry and attenuation properties of the crust between the source and site. Site parameters include the bedrock geometry and soils underlying the site.

The basic goal is to characterize the effect of these parameters at a site in a form useful to the bridge designer. The response spectrum, which shows the frequency response of an elastic SDOF oscillator to an earthquake acceleration record, is commonly used for this purpose in the dynamic analysis of bridges. The response spectrum used for design is typically a smoothed base or rock spectrum which has been scaled using the ground-shaking severity coefficient (e.g. expected peak ground acceleration) associated with the site. This spectrum is then modified to account for the effect of local soil conditions. The mapping of the severity coefficient involves identifying regional source zones and their associated seismicity, estimating attenuation relationships, and establishing mapped contours with equal probability of occurrence of the severity coefficient. Modification for local soil conditions is usually based on generalized site descriptions categorized by depth and type of geologic deposits.

The ATC-6 guideline were formulated using mapped values of expected peak ground acceleration and velocity in the contiguous United States. These values were used to develop maps of Effective Peak Velocity-related acceleration, a severity coefficient based on spectral amplification and velocity attenuation.

Rock spectra and modification factors for local soil conditions in ATC-6 were developed using a study by Seed, Ugas, and Lysmer [2]who found significant differences in spectral shapes for four different generalized soil conditions: (a) rock, (b) stiff soil, (c) deep cohesionless soil, and (d) soft to medium clays and sands. An ensemble of 104 strong-motion records were used in this analysis, the majority from California earthquakes. The rock and stiff soil categories were combined into one category in the ATC-6 study.

The approach used by ATC-6 is similar to seismic guidelines developed by Gates [3]and used by the California Department of Transportation. Gates also uses base spectra scaled by a mapped severity coefficient and modified for local soil conditions. Peak ground acceleration is used as the severity coefficient and base spectra were developed using scaled records based on rock attenuation relationships. The computer program SHAKE [4]was used to develop soil modification factors. This program models site response by assuming vertical wave propagation through horizontal soil layers. Soil amplification factors were developed for varying depths of alluvium.

The assumption made in each of these approaches is that the effects of each of the source, path, and site parameters are incorporated in the severity coefficient and/or the base spectra and/or the soil amplification factor. The energy released, the hypocentral distance (which is a function of the epicentral distance and the focal depth),and general crustal attenuation properties affect the value of the severity coefficient. The hypocentral distance will also affect the base spectrum by shifting the response to longer periods with greater distances. This is due to the more rapid attenuation of higher frequency waves as they move away from the source. Two effects of increasing focal depth are the suppression of very long period surface waves and an increase in higher frequency modes (for the same hypocentral distance) due to greater anelastic attenuation. Types and depths of soils underlying the site will directly affect the soil amplification factors used.

Some of these parameters are path-specific, that is, the effect on the response spectrum depends primarily on the physical relationship between the causative fault and the site. These parameters include direction and dimensions of faulting, variations in attenuation characteristics along the wave path, bedrock stratigraphy (in terms of reflection and refraction of incident waves), and certain bedrock profiles, such as valleys, which are known to produce focusing effects. These path-specific parameters can affect the severity coefficient (based on their relation to causative faults), and the base spectrum in terms of frequency content.

METHOD

The purpose of this research is to develop guidelines appropriate for Western Washington following procedures similar to those used in ATC-6. This involves developing a severity coefficient map, base spectra, and soil modification factors which reflect Western Washington unique seismicity and geology.

The severity coefficient mapping has already been accomplished by Higgins [5]for the WSDOT. He used Perkins' 1980 study [6]of probabilistic ground motion in the Pacific Northwest which considers geologic information as well as historic seismicity in the development of acceleration and velocity contour maps. Higgins' work established contours of Expected Peak Velocity-related acceleration which is similar to the severity coefficient used in ATC-6 in that it is adjusted to reflect velocity attenuation. It should be noted that the acceleration term used by Higgins is not the same as the "effective" peak acceleration term described in ATC-6.

In the present work, the procedures of Gates are followed in developing the base spectra and soil amplification factors. Appropriate base spectra are identified and modified as necessary to incorporate any differences between earthquakes used to develop the spectrum and Western Washington earthquakes. Amplification factors are then developed for typical Western Washington soil profiles.

The Gates rock spectrum was chosen as the base spectrum in the present work rather than the ATC-6 spectrum for several reasons. The severity coefficient used in this study is actual peak acceleration and not effective peak acceleration (as in ATC-6), which makes the Gates spectrum more appropriate because it is also based on actual peak accelerations. The Gates spectrum also incorporates the general lengthening of the period as the earthquake waves move away from the source (as indicated by separate curves for different acceleration values - see Fig. 2). The ATC-6 study does not consider this difference.

Gates used California earthquakes to develop base spectra but modified them to account for the change in frequency content with increasing epicentral distance. These are broad-band spectra in that they encompass a wide range of frequency contents and are do not necessarily reflect specific characteristics of California earthquakes except in a very general way. Modification of these spectra involves identifying those earthquake parameters that can affect the response spectrum that are not reflected in the Gates spectra.

To be able to use a single base spectrum for an area of this size, the effect of the path-specific parameters must be generalized, that is, if variations of the parameter change the frequency content of the resultant response spectrum, then the base spectrum should incorporate the expected variation. The use of a broad-band spectrum is a reasonable approach to encompassing this variation.

It is important to consider the greater focal depths that can be expected to occur in western Washington earthquakes, and the associated effects of greater expected high frequencies and lower expected long periods in the response spectral values. Because of the possibility of surface rupture in this area (considered by Perkins) and because of the adverse effect of long period motions on bridges, it would be inadvisable to make any reduction in the response spectral values at longer periods. Any increase in the high frequency response will be encompassed by the broad-band spectra used.

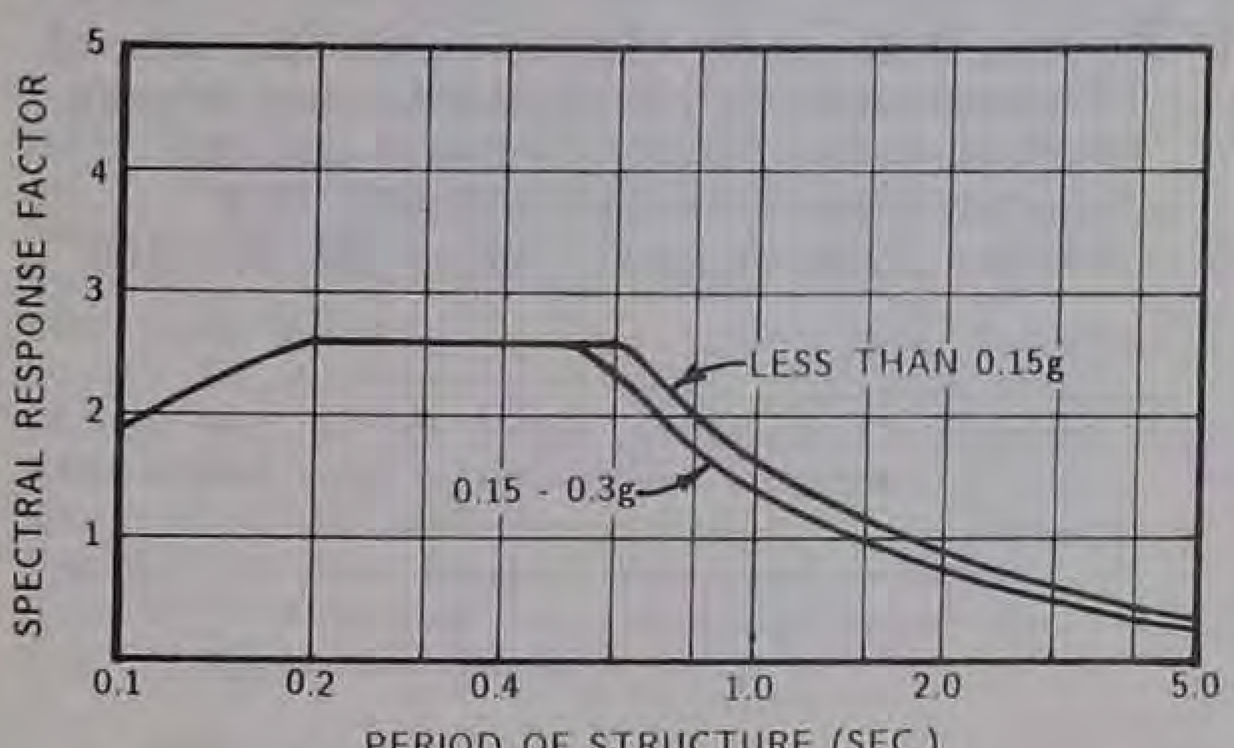

Fig. 2. Gates rock spectral curves for varying peak rock accelerations.

From this brief discussion, it seems reasonable to conclude that the Gates' rock spectra can be used without modifications. Because it is broad-banded, the effects of variations that can be expected in Western Washington strong ground motion have been considered. While it may be conservative for any particular site or earthquake occurrence, it encompasses the range of expected ground motions at a variety of sites.

Soil amplification factors are developed using the computer program SHAKE in a manner similar to the method used by Gates, although the soil categories are different. SHAKE has been used widely in this type of analysis with very good correlation between modeled and actual response. Because of the wide use of this program, the problems inherent with its use are well documented and this should lead to a more rational interpretation of the results.

Input requirements for SHAKE include a base motion acceleration time history, depths and properties of the soil layers, and strain-dependent damping and moduli values for the various types of soil. The surface response spectrum generated is divided by the base response to produce the soil amplification factor for any soil profile.

The non-linear behavior of soil response is taken into account in this program through the use of strain-dependent moduli and damping values. Although there are several newer analytical one-dimensional models which consider pore-pressure build-up, the difference in the results is not significant.

It appears that the time history input is not critical but there are differences in the results due to the change in strain-related damping and moduli values for different input motions. The input time histories used are the deconvoluted (using SHAKE) 1949 and 1965 Puget Sound strong-ground motion records, the 1952 Taft record, and simulated records. These are scaled to the peak ground acceleration values expected and encompass the targeted broad-band base spectra.

Soil profiles from actual bridge sites in Western Washington are used to develop typical soil profiles. Moduli and damping values are based on laboratory tests where available or on empirical relationships between standard penetration values and dynamic properties of the soils.

Much of the geology in the Puget Sound basin is dominated by the effects of the various advances and retreats of the Puget Lobe of the Cordillerian Ice Sheet. Deep layers of heavily consolidated till interspersed with glaciofluvial and glaciolacustrine deposits make up much of the geologic profile in the Puget Sound basin. Saturated sands overlying stiff clay layers in these deposits have been suspected of inducing severe ground shaking in past Puget Sound earthquakes. More recently deposited Holocene alluvium and fill are present and are known to cause amplification of ground motion in general. The marine depositional environment along the coast and inland waterways will also be of concern.

Generalized site categorization are made on recognizable geologic features. Results of borings at a future bridge site should allow the designer to determine the appropriate amplification factor to be used as well as identify any particularly sensitive formations.

The results of the base spectra plus soil amplification factors are compared to appropriate theoretical and empirical studies. The reported damage of the 1949 and 1965 earthquakes are also examined to determine if effects other than those modeled in this study are critical in this area.

DISCUSSION

Peak ground acceleration is not the best indicator of the severity of ground shaking at a site. In general, the peak acceleration is associated only with the very high frequency content of the ground motion. It has also been shown that there is little correlation between earthquake magnitude and peak acceleration for magnitude ranges between 4.5 and 7.1. The ATC-6 study used a severity coefficient related to spectral amplification values because of these difficulties with the peak ground acceleration. This parameter, however, is not well defined and has not gained acceptance in the engineering community.

Ground response at any site is a complex function of many parameters and much progress is being made in understanding and modeling these parameters and their effects. Unfortunately, when attempting to do an regional zonation study of this magnitude, the use of these sophisticated models becomes intractable. The result of this is the use of the broad-band base spectra for all sites to encompass the expected variations at different sites. This means the design spectrum used may be overly conservative for any particular site.

There are many elements in any seismic risk study which have high degrees of uncertainty. There is a definite concern that the common reaction is conservativism with respect to each of these uncertainties. This multiplicity of conservativism can lead to overestimations of ground shaking and hence uneconomical design. If this can be kept in mind at every decision level, it is possible to produce rational design coefficients that will lead to economical as well as safe designs.

ACKNOWLEDGEMENT

The support of this project from TRAC/WSDOT is gratefully acknowledged.

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LATERALLY LOADED PILES IN SHALE AND SANDSTONE

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SYNOPSIS

A series of field tests on eight model piles were used to determine p-y curve parameters for rock. The tests showed that the stiff clay p-y curve may be used to predict the lateral load behavior of piles in low and moderate strength clay shales. A p-y curve for sandstones and sandy shales was developed from the test data as well.

INTRODUCTION

The analysis of piles under lateral loading in Kansas began after exposure to the concepts in an FHWA pilot workshop on the subject in 1983. At this workshop, Dr. Lymon Reese provided information regarding the analysis of piles under lateral loading. In the course of Dr. Reese's presentation, he mentioned a procedure by which experimental soil response - pile deflection (p-y) curves could be generated using uninstrumented piles. The use of this procedure to develop p-y curves for rock is described in this paper.

SOIL CONDITIONS

The test program required sites which were level and had competent Pennsylvanian age shale or sandstone at or within a few inches of the ground surface. The locations which best suited these requirements were four sites in the Kansas City area along I-435 and K-10 in Johnson and Wyandotte Counties. Two sites were chosen in shales. These were at I-435 and Holliday Drive in the Lane Shale, and at I-435 and Johnson Drive in weathered Bonner Springs shale. The other sites were located in shaley sandstone and sandstone. These sites were at K-10 and Renner Road in Johnson County in the Bonner Springs member and at I-435 and Leavenworth Road in the Tonganoxie Sandstone.

TEST PILE INSTALLATION AND LAYOUT

The pile system chosen for the tests was tailored to meet the requirements of readily available components and ease of construction. Each test location had two 15 foot long model piles. The piles were constructed of eight inch drill casing filled with concrete. Each pile was instrumented with dial gauges at points above ground and with an inclinometer casing mounted on the centerline of the pile to measure deflections below groundline. The lateral load applied to the two pile system came from a manually operated 50 ton hydraulic jack set between two steel load transfer saddles. The load applied to the piles was measured with a calibrated electronic load cell. A swivel head was used between the load cell and one of the saddles to compensate for any rotation of the saddle due to deflection of the pile on two of the tests.

The model piles were installed by predrilling slightly undersized holes using eight inch hollow stem auger with a Mobile B-61 drilling rig and driving the eight inch casing into place with a 300 pound drop hammer. After the casing was seated the inclinometer casing was set and the bottom of the pile was filled with approximately two feet of quick setting concrete to fix the alignment of the inclinometer. After all four sets of piles had be installed in this fashion, each pile was filled with concrete. Steel saddles fashioned from drill casing and half inch plate were welded to the piles after the concrete had set. These saddles were used to transmit the load from the jack to the piles.

TEST PROCEDURE

The pile test procedure involved

a set of repeated 3 ton loadings with three to four repetitions of load to determine the sensitivity of the formation to cyclic loading effects. This was followed by a set of increasing load increments up to a maximum of 20 tons. At each load increment inclinometer and dial gauge readings were taken to determine deflection with respect to the applied load. The load was increased after each reading until the 20 ton limit was reached or until the pile exhibited sustained continuous deflection under steady load.

REDUCTION OF DEFLECTION DATA

Inclinometer and dial gauge readings from each test were plotted to observe the deflection of the piles under load. There was some variance between the surface level inclinometer and dial readings. Averaging the two sets of readings was chosen as the best way to estimate the groundline deflection. Linear, geometric, exponential, logarithmic, and parabolic regression methods were tried as averaging methods. Parabolic regression was chosen as the averaging method because it gave the best fit.

P-Y CURVE PREDICTION FROM LOAD-DEFLECTION DATA

The method used to develop these p-y curves for rock involved the use of the nondimensional curves developed by Reese and Matlock (1956). A linear variation of soil modulus with depth was assumed.

The method used with the data obtained from the field tests involved fitting the groundline deflection to a corresponding point on the nondimensional curve for pile head deflection. This was done by assuming a value for the soil modulus and running through a trial computation to develop a corresponding deflection. The modulus value was altered until the computed deflection at groundline matched the observed pile deflection. This modulus value was subsequently used to compute p-y values for the pile and loading under consideration. A set of p-y values for selected depths was built up in this way for each load increment. The validity of the resulting p-y values were tested by using them as input to the program COM 624 and comparing the output of the program with the observed deflection of the pile.

SUGGESTED P-Y CURVES FOR SOFT ROCK

The results of the load tests run on piles embedded in soft rock indicate that the stiff clay above the water table p-y curves may be used for weathered clayey sedimentary rock. The stiff clay curves have been used recently to predict the behavior of piles in rock. Use of the stiff clay curves should produce a reasonably accurate prediction with formations whose unconfined compressive strength does not exceed 4 TSF (55.6 psi).

A suggested p-y curve for unweathered sandstone and sandy shale is presented in Figure 1. The data from the present series of test do not indicate that a family of curves exist which represents increased resistance with depth. Therefore, this curve may be used for all depths. Load tests are recommended for piles designed using this curve if the deflection of the pile exceeds the value given for y_1. Brittle fracture of the rock is assumed if the lateral stress (force per unit length) against the rock is greater than the diameter of the pile times the shear strength of the rock. The proposed curve should be able to be used to predict the behavior of hard unweathered shale. Care should be exercised in this application as the use of this curve has not been proven in unweathered shale. As a final illustration, the p-y curves for stiff clay, vuggy limestone, and unweathered sandstone are plotted for a material with a shear strength s_u = 15 TSF (208 psi) in Figure 2. This figure demonstrates the great variability of p-y curves for different materials.

SUMMARY AND CONCLUSIONS

The results of this series of load tests in shale and sandstones showed that the stiff clay and vuggy limestone p-y curves are not sufficient to predict the behavior of all types of sedimentary rock. It also provided sufficient information to propose a p-y curve for unweathered sandstone and sandy shale and to prove the applicability of the stiff clay p-y curves for certain types of weathered shale.

A side benefit of this program was the discovery of how easily this data can be obtained using tools that are available to nearly every state highway agency. There is no reason why the uninstrumented pile method of lateral load testing cannot be used routinely to add to the information already available on the behavior of piles under lateral load.

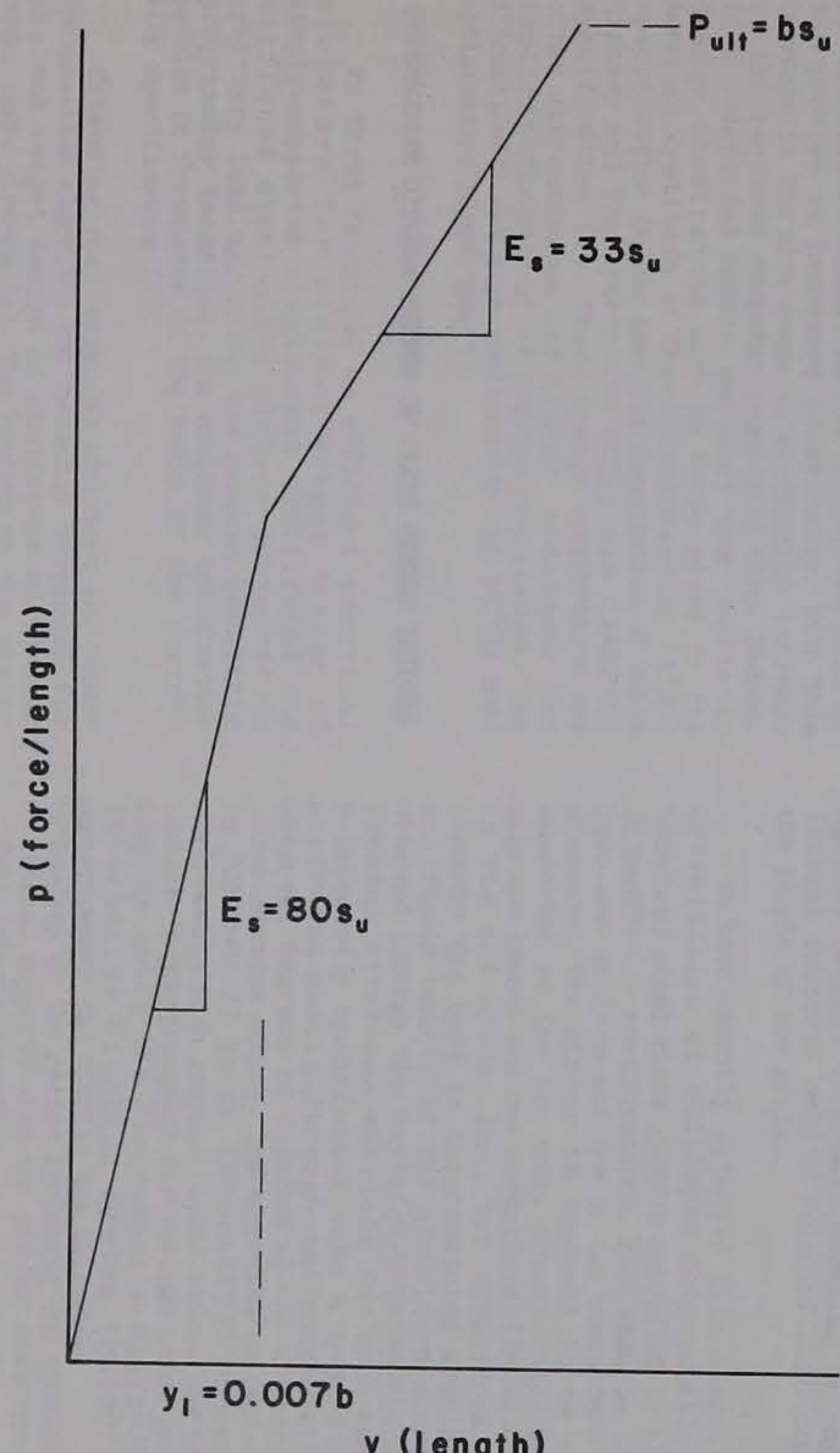

FIG. 1. p-y Curve for Sandstone and Sandy Shale

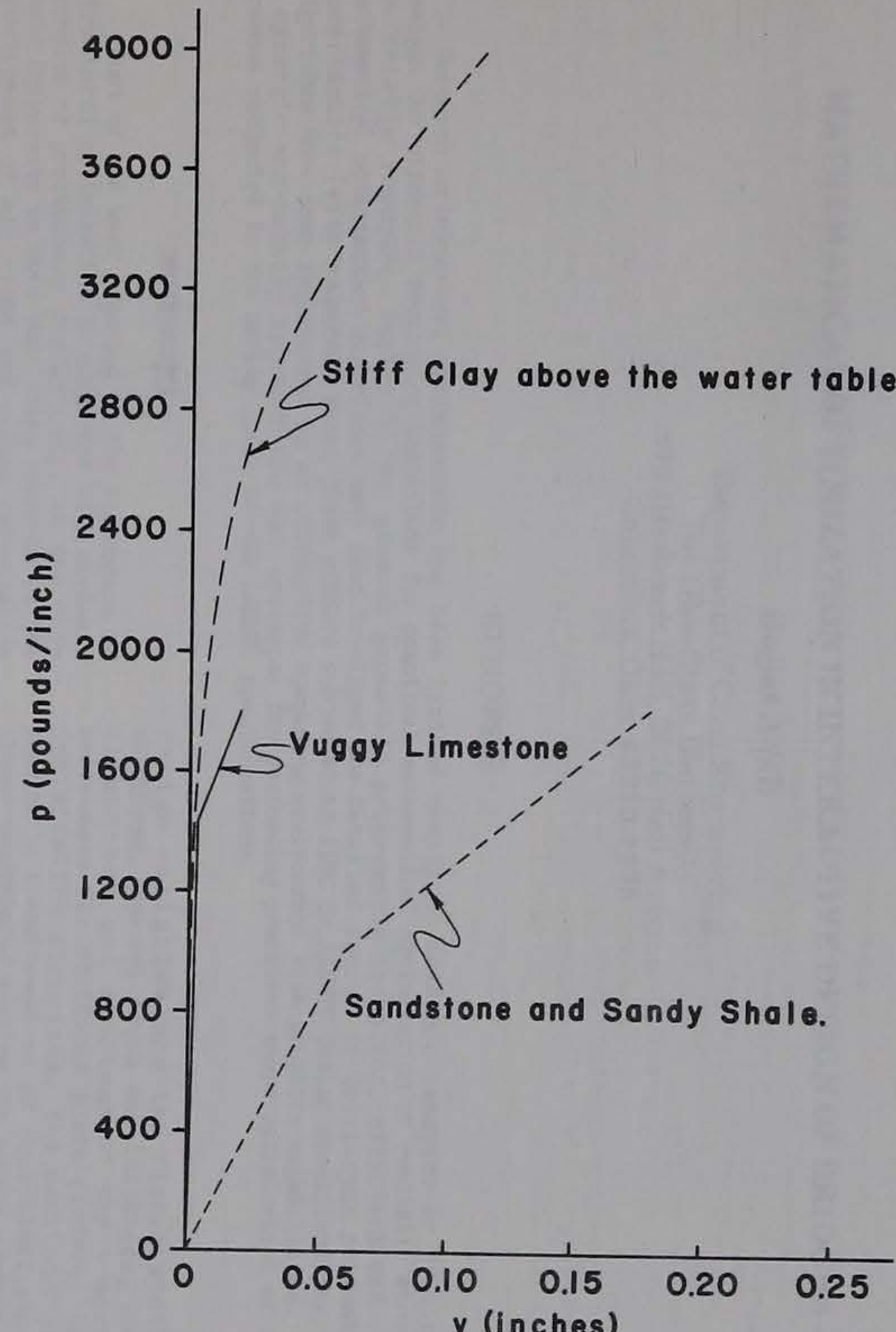

FIG. 2. Comparison of p-y Curves for $s_u = 15$ TSF

MATHEMATICAL OPTIMIZATION IN INTERACTIVE DESIGN OF BRIDGES

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SYNOPSIS

Research on structural optimization has been limited mostly to academic examples or preliminary design. Our effort in developing algorithms for practical mathematical optimization of realistic structures is briefly reported. Employing the general geometric programming technique, efficient and robust mathematical optimization algorithms have been developed for detailed design of multi-span prismatic or nonprismatic (with haunches) steel plate girders subjected to AISC or AASHTO design specifications. These algorithms have been implemented in an interactive computing environment with graphics capabilities. Also, a synergic man-machine approach has been developed for performing practical shape optimization of bridge trusses subjected to the moving loads of the AASHTO specifications.

INTRODUCTION

Most of the work reported in the literature on structural optimization is concerned with academic examples or preliminary design only. At the Ohio State University we have been doing research on the development of efficient and robust optimization algorithms for detailed design of structures during the last few years. Two major problems are encountered when mathematical optimization is used for detailed structural design. First, the size of the problem is increased drastically. With this increase in the size comes an exponential increase in the required computer processing time. Second, in the detailed design one must use realistic design constraints such as those given in the American Institute of Steel Construction (AISC) specification (6) or American Association of State Highway and Transportation Officials (AASHTO) specification (1). These design constraints are often discontinuous and highly nonlinear and implicit functions of design variables. The discontinuity and high nonlinearity can baffle most optimization algorithms(7).

INTERACTIVE OPTIMUM DESIGN OF PLATE GIRDER BRIDGES

We first developed an efficient practical procedure for minimum weight design of simply-supported single-span unstiffened and stiffened steel plate girders subjected to arbitrary loading, using the general geometric programming technique. The nonlinear optimization problem is formulated on the basis of the current AISC specification (2).

Extending that work, an efficient and robust optimization algorithm was recently developed for minimum weight design of continuous multi-span steel plate girders (4). The loading on the girder can be quite general consisting of any number of concentrated and uniformly distributed loads. The design variables are the flange width and thickness, the web depth and thickness, and the dimensions and spacings of the transverse stiffeners for stiffened plate girders. In the optimization algorithm, the nonlinear primal problem is transformed to an equivalent standard linear programming problem via double condensation. The algorithm is quite general and can be applied to stiffened or unstiffened, homogeneous or hybrid plate girders. The girder may be fully restrained against lateral torsional buckling or may have lateral supports only at selected locations along the length of the girder.

We have recently extended this algorithm to optimization of multispan nonprismatic (with haunches) steel plate girders subjected to AISC (6) or AASHTO (1) specifications. Each span is divided into one uniform and one or two nonuniform finite elements. The girder is assumed to be simply supported at the two ends. Consequently, there is only one haunch and one nonuniform finite element in the end spans. But, two nonuniform finite elements are used in the remaining span(s). The stiffness matrix of the nonuniform element is obtained through the Rayleigh-Ritz approach. The element stiffness matrices for each span are subsequently transformed into a superelement stiffness matrix through the condensation of internal degrees of freedom where a change of cross-section occurs. The algorithm is implemented in FORTRAN 77 in an interactive computing environment with graphic capabilities. The program includes a postprocessor for various color graphics display using the Programmer's Hierarchical Interactive Graphics System (PHIGS). The postprocessor can display the following:

1. Loading on the girder including the locations of lateral supports when the girder does not have full lateral support.
2. Elevation of the girder. It can display the complete plate girder or the elevation of each

individual span. In the latter case, the cross-sections of the plate girder at locations of intermediate stiffeners are also displayed.
3. Bearing stiffeners at supports.

The displays of the elevation and cross-sections of the girder include all the necessary details and dimensions for manufacturing the girder. The theoretical dimensions obtained from the optimization algorithm are rounded in the interactive program to practical values. Thus, the algorithm and computer program developed in this research can be used for practical design and manufacturing of continuous multispan nonprismatic steel plate girders.

Details of the optimization algorithm are presented in a forthcoming article (5). Figures 1 and 2 show the main menu and the display menu of the interactive optimization software for practical minimum weight design of multispan steel plate girders with haunches. The practical optimum design of a three-span hybrid stiffened plate girder according to the AASHTO specification (1) is presented as an example. The AASHTO design constraints include

a) bending stress constraints
b) shear stress constraints
c) intermediate stiffener spacing
d) aspect ratios of the web and flange plates, and
e) dimensional constraints of the intermediate stiffeners.

Figure 3 shows the loading on the girder as displayed by the interactive optimization software. Figure 4 shows the middle span of the practical minimum weight design.

PRACTICAL SHAPE OPTIMIZATION OF BRIDGE TRUSSES

We have developed a synergic man-machine approach to practical shape optimization of statically determinate or indeterminate steel bridge trusses. An optimization algorithm has been developed for minimum weight design of bridge trusses subjected to the AASHTO moving loads. The design variables are the cross-sectional areas of the truss members. The design constraints are the stress, displacement, slenderness, and fabricational constraints of the AASHTO specification(3).

The process of finding the maximum forces due to live loads acting on the bridge truss is not straightforward due to the complexity of AASHTO live loads. A heuristic procedure has been developed for finding the maximum compressive and tensile forces in the truss members based on the classification of the shape of the Influence Line Diagrams (ILD's) and the type of AASHTO live loads. The heuristic procedure is based on pattern recognition and uses information about the shape of ILD's for the bridge truss type under consideration. For statically indeterminate trusses, this information is obtained through numerical machine experimentation for any given type of truss. The ILD's for member axial forces of a bridge truss are classified according to their shapes. Decision trees and heuristic rules have been developed for finding the maximum compressive and tensile forces in the members of a given type of truss.

The synergic man-machine approach to shape optimization is based on changing the key layout parameters of the bridge truss (that is, the height and the number of panels) and performing optimization for each layout in an interactive computer graphics environment. The method is applied to four types of steel bridge trusses used for a span range of 100-500 ft. They are Pratt, Parker, parallel-chord K truss, and curved-chord K truss, as shown in Figure 5. An interactive menu-driven FORTRAN 77 program has been developed for shape optimization of these types of trusses,

```
MAIN MENU

1. READ INPUT DATA FROM A FILE

2. READ INPUT DATA INTERACTIVELY

3. EDIT INPUT DATA FILE

4. START DESIGN/OPTIMIZATION

5. DISPLAY MENU

6. QUIT

ENTER YOUR CHOICE
```

Figure 1

```
DISPLAY MENU

1. DISPLAY LOADING

2. DISPLAY THE ELEVATION

3. DISPLAY BEARING STIFFENERS

4. MAIN MENU

ENTER YOUR CHOICE
```

Figure 2

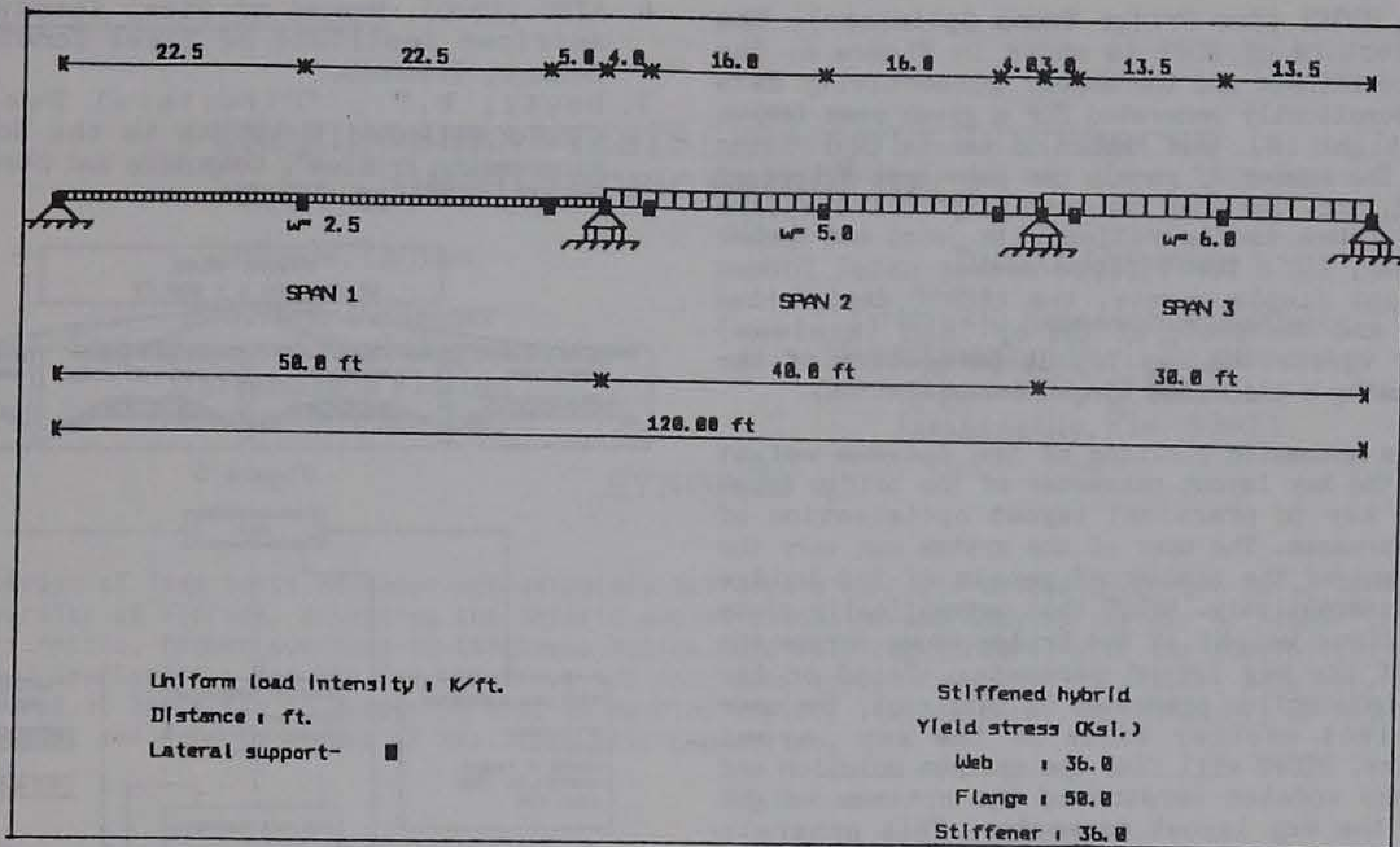

Figure 3 Loading on the three-span plate girder

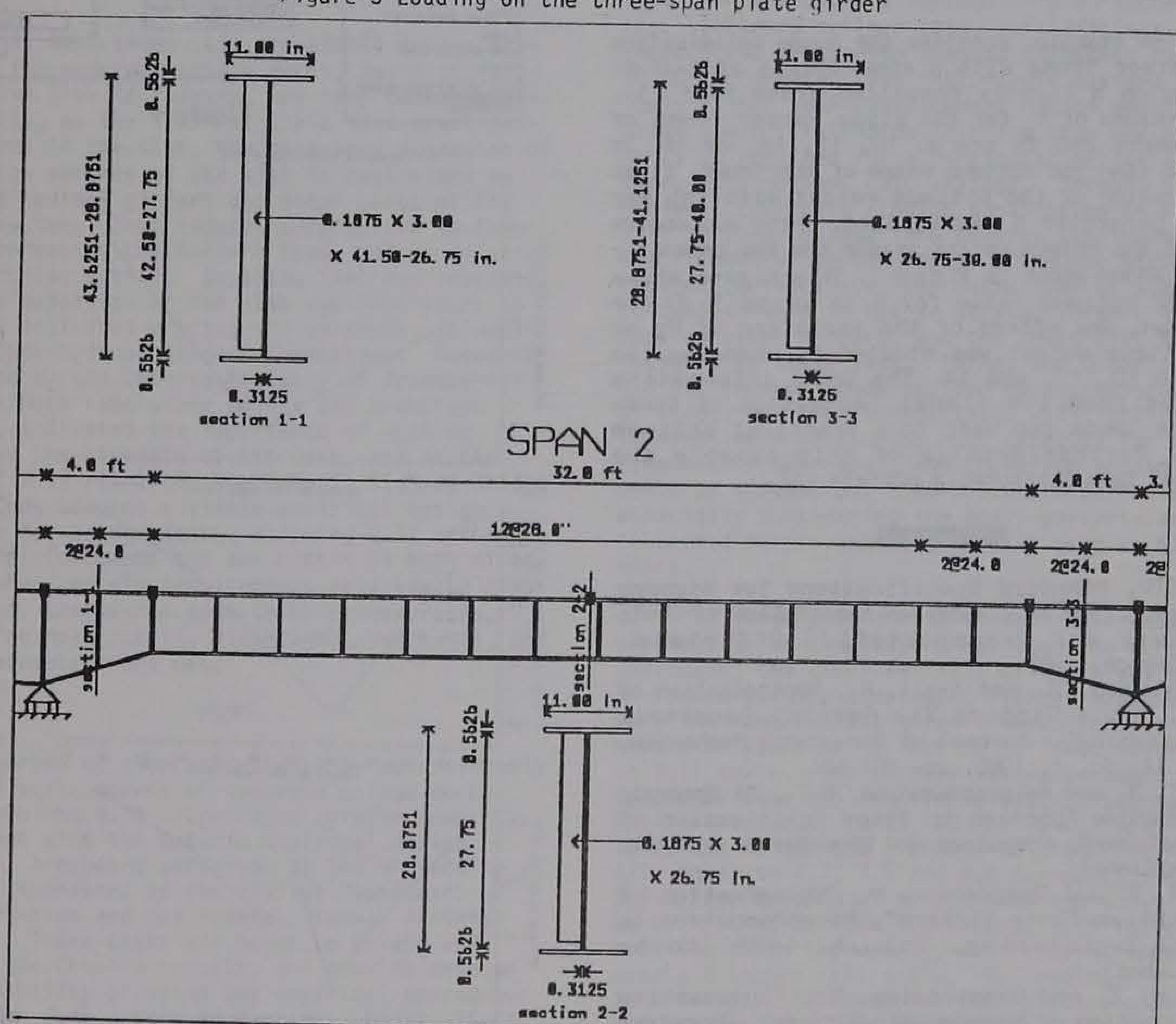

Figure 4 Elevation and cross-sections in the middle span

called BTOPT (for Bridge Truss Optimizer). The architecture of BOPT is shown in Figure 6. The nodal coordinate and the member connectivity data are automatically generated for a given span length (L), height (H), and number of panels (N_p) of the truss. The number of panels can take even values of 4, 6, 8, etc. The user can have graphical displays of the truss configuration with joint and member numbering, ILD's for various member axial forces and joint displacements, the AASHTO design live loads, and the plots of the optimum (minimum) weight versus the key layout parameters of the truss using a piecewise linear interpolation.

The automatic plotting of the optimum weight versus the key layout parameter of the bridge truss is the key to practical layout optimization of bridge trusses. The user of the system can vary the height and/or the number of panels of the bridge truss interactively. BTOPT then automatically plots the optimum weight of the bridge truss versus the value of the key layout parameter. Based on the trend information presented in this plot, the user can select another value of the key layout parameter. BTOPT will find the optimum solution and plot the updated version of the optimum weight versus the key layout parameter. This synergic man-machine approach leads to a practical optimum layout very quickly.

As an example, consider the shape optimization of a Parker truss with a span length of 240 ft located on a lightly travelled state road (3). Common values of N_p for the given Parker truss of span length 240 ft are 8, 10, 12, 14, or 16. In order to find the optimum shape of the truss, first the variation of the optimum weight with the key layout parameter H was studied. Three successive plots of the opimum weight versus the key parameter H for N_p=8 are shown in Figure 7. This figure shows that the optimum value for H is around 36 ft for N_p=8. Next, the effect of the variation of Np on the optimum weight was studied by changing its value to 10, 12, and 14. The trend information obtained from the visual inspection of these diagrams leads the user to a practical optimum shape. Further details of this example are presented in a forthcoming article (3).

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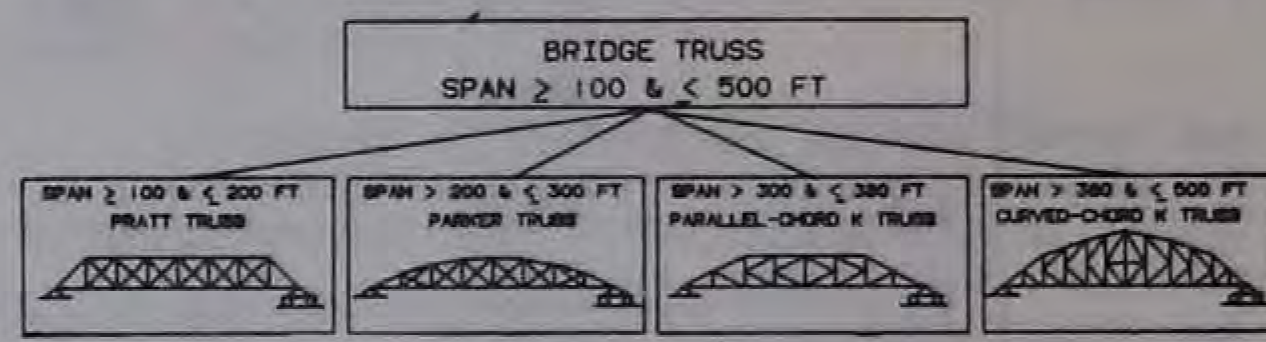

Figure 5

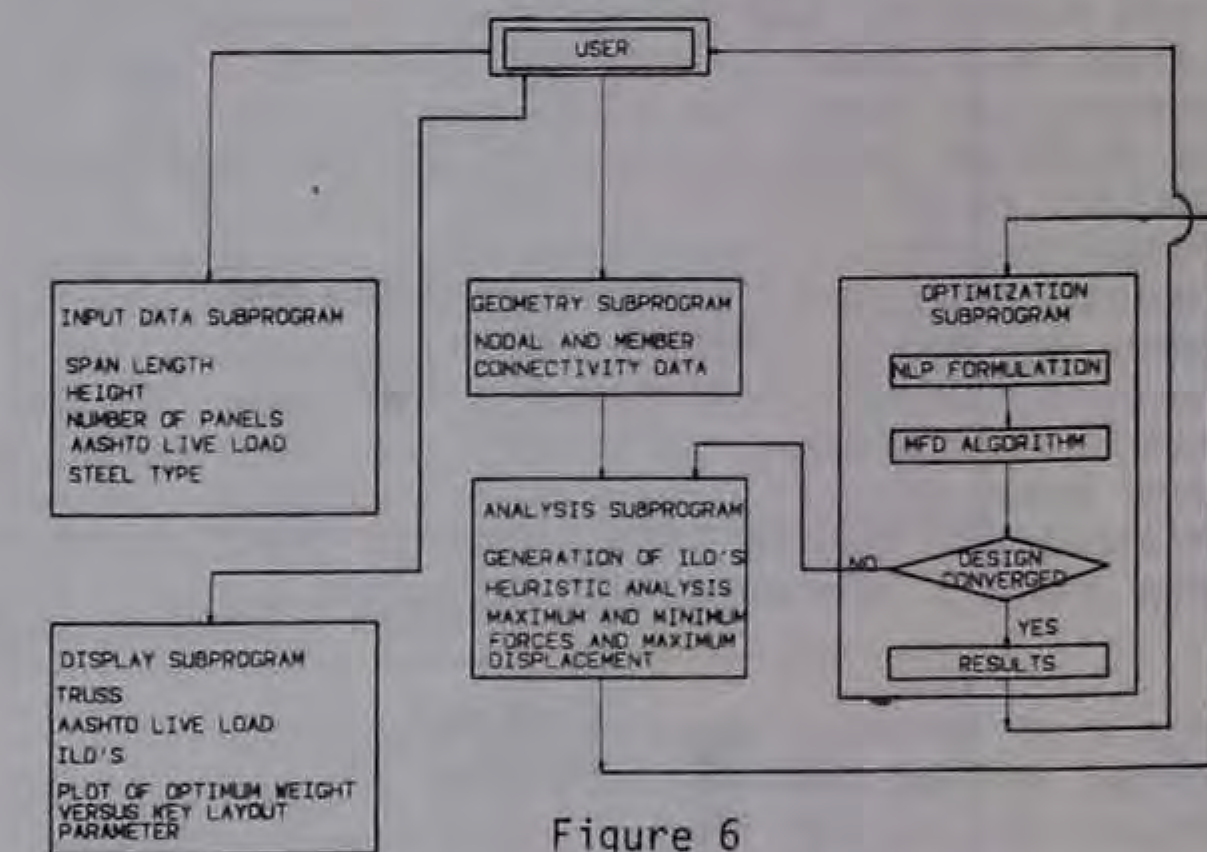

Figure 6

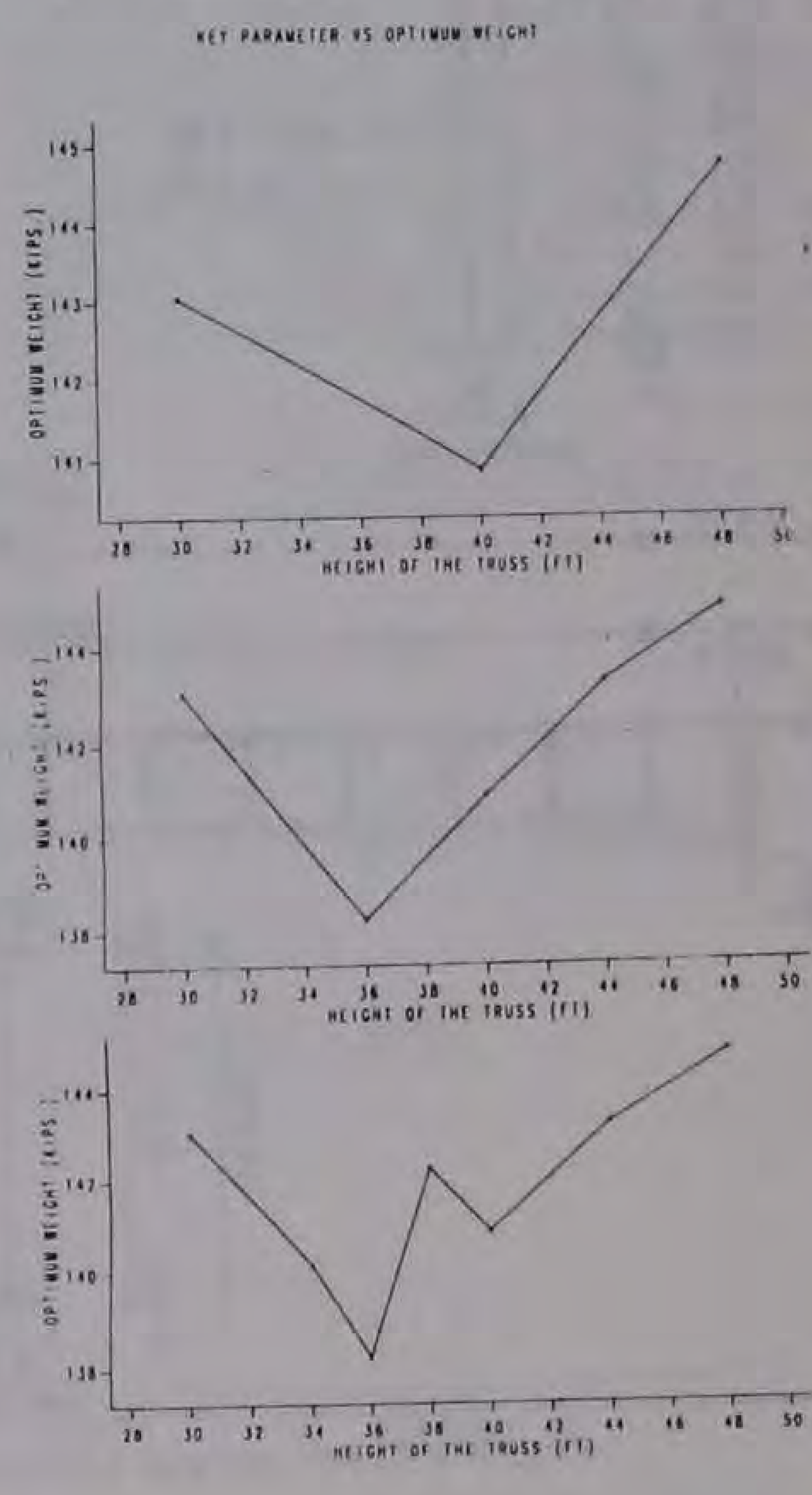

Figure 7

PUNCHING SHEAR TESTS OF LIGHTLY REINFORCED ORTHOTROPIC BRIDGE DECKS

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SYNOPSIS

A series of load tests on seven approximately half-scale concrete bridge decks are being performed at the University of Florida, extending the Ontario empirical design approach to the use of higher span-to-thickness ratios, higher overhang-to-thickness ratios, and bulb-tee girders, with the possibility of using the method in Florida. Results for the first two specimens appear favorable. Failures were very ductile and occurred at loads far in excess of that to develop a yield line pattern, far in excess of AASHTO design loads, and even in excess of available axle capacities.

BACKGROUND

It is well known that the AASHTO design provisions for concrete bridge decks, based on failure of the slab in flexure, are very conservative. Apparently, as the flexural yield line crack pattern forms in the slab, the necessary expansion of the bottom surface of the slab is restrained by the longitudinal girders and other parts of the bridge system. This induces compression in the deck, increasing the failure load, and is referred to as arching action. Anything that can restrain in-plane expansion of the slab can contribute to arching, including bracing and parapets, as well as the longitudinal girders themselves. Research performed by the Ontario Ministry of Transportation, on both laboratory models and prototype bridges, indicated the importance of arching action on the strength of the deck, and on the basis of this research, the Ontario Highway Bridge Design Code adopted a simple empirical design approach for bridge decks, allowing 0.3% orthotropic reinforcement top and bottom in both directions, when certain requirements relative to slab thickness, transverse span-to-thickness ratio, transverse span itself, diaphragms, overhangs, and other parameters are met.

SCOPE

A series of laboratory tests on approximately one-half scale models of concrete bridge decks built with the 0.3% orthotropic reinforcement consistent with the Ontario empirical design approach, are being performed at the University of Florida, sponsored by the Florida Department of Transportation and the Federal Highway Administration. These tests are meant to extend and enhance the Ontario results, and provide data on the feasibility of using the empirical approach for bridge deck design in Florida. First, it is common practice in Florida to use deck thicknesses less than the Ontario minimum of 8.85 inches. The test specimens reflect this, and additionally incorporate transverse span-to thickness ratios of 18 to 22, greater than the Ontario maximum of 15. These higher span-to-thickness ratios would permit using fewer longitudinal girders than would normally be required for a particular bridge structure, with obvious cost savings. Second, bulb-tee girders, currently finding application in Florida, will be incorporated into several specimens. As the flanges of these girders are quite wide, and tapered in thickness, the definition of span-to-thickness ratio is not so straightforward as for a deck on steel girders, and the tests will be designed to study this. Third, the Ontario tests did not include loads applied to overhanging edges of slabs, and the Florida tests will include this. It might well be thought that capacity for confinement in overhangs, and therefore arching, would be slight, but some tests are warranted, especially considering the heavy parapets and concentrated reinforcement generally used at slab edges.

The first year of the study involved static application of simulated wheel loads to three specimens, the cross-sections of which are shown in Figure 1. The steel wide flange shapes that were the longitudinal girders were simply supported on spans of 43 feet to 59 feet, converted to full scale. The primary variables among the specimens were the transverse span-to-thickness ratio, S/T, equal to 18, 20 and 22 for specimens 1, 2, and 3, and the overhang-to-thickness ratios, A/T, equal to 4.7, 7.5 and 9.8 for Specimens 1, 2 and 3, and B/T, equal to 3.0, 4.9 and 6.8. The deck thicknesses were 4.5 inches, 3.75 inches and 3.25 inches on the models, all implying approximately 8 inches full scale. By loading the deck in a variety of locations, the relative effects of the parapet, the free overhang, and the confinement of the interior spans could be observed. At this writing, results are available for the first two of the above specimens, while the third is about to be tested.

The second year of testing is to include three specimens with slabs cast on standard size bulb-tee girders, as recently adopted in Florida for spans in the range of 100 to 160 feet. The full-scale cross-section is shown in Figure 2, where A would vary from 8 ft to 2 ft and the ratio A/B would be constant at 2. A final specimen is to be similar to the three tested during the first year, but instead is to be subjected to repeated loading at several locations, providing some data on how the arching effect behaves under such conditions.

LOADING AND INSTRUMENTATION

Loads were applied to the deck through heavy steel plates, shaped and sized to model the imprint of one dual wheel formation in approximately one half scale. The loading positions and patterns for the first two specimens are shown in Figures 3 and 4. Loading on the first specimen represented either one set of dual tires or two sets of dual tires belonging to separate trucks passing close to one another, while loading on the second specimen represented the entire tandem wheel formation of one truck. Multi-imprint loadings were applied to distribute the load equally among the imprints. Note that all five basic load positions were covered in the study: interior, free edge, free corner, parapet edge, and parapet corner.

Vertical deflections were measured using 15 differential transformers, arranged in a pattern so as to define a deflection basin. Electrical resistance strain gauges were used at various locations in various tests, including the top and bottom surfaces of the deck, on the reinforcement, on the longitudinal girders, and on the bracing.

TEST RESULTS

The test results for both specimens are summarized in Figures 5 and 6 as variations of total applied load with deflection at the center of the tire imprint formation.The maximum load attained during each test, the load at which a flexural yield pattern was well-developed in the deck, and the maximum deflection attained beneath the load during each test, along with the above results converted to full scale, are given in Table 1.

Interior Tests

Comparison of the maximum loads and yield loads in Table 1 indicate the reserve strength of the deck relative to the load associated with the flexural yield line mechanism. For the first specimen, in-plane forces developed in the slab were sufficient to cause welds in the bracing adjacent to the test position to fail. In the second specimen, the bracing and welds were strengthened, and weld failure occurred in only one case. It is difficult to say how much of the increased strength for specimen 2 was due to the effect of the bracing on the in-plane forces and how much was due to the load pattern, but the reserve strength beyond yielding was certainly considerable, even though the transverse span-to-thickness ratio for this specimen was 20, as opposed to the maximum of 15 in the Ontario code.

The mode of failure for specimen 1 was clearly punching, involving the entire width of the panel between girders and an approximately equal distance along the length of the slab. For specimen 2, there was some tendency for the pattern to not be as circular and to involve a longer length, perhaps reflecting either the larger span-to-thickness ratio of the specimen or the larger pattern of the tire imprints. The basic ductility inherent in the failure mechanism for both specimens was apparent in the nonlinearity of the load-deflection relations, indicated in Figures 5 and 6. The deformations at the end of each test were dramatically visible to the eye.

Free Midspan and Corner Tests

For both specimens, as indicated in Table 1, even these tests, both at midspan and at the corner, indicated considerable reserve capacity beyond the load at which the yield line pattern formed. For specimen 1, the failure load for the midspan edge was equal to the lowest failure load recorded for the interior tests, while the midspan edge for specimen 2 had still not failed at 50 kips. The reserve capacity of the free edge is evidently aided by transfer of load directly to the longitudinal girder, as indicated by the similarity of stiffness between the midspan edge test and the adjacent interior test in specimen 1, by the linearity and high stiffness of the midspan edge test in specimen 2, and by the observation that the maximum deflection for both tests in specimen 2 occurred immediately to the inside of the girder.

Parapet Midspan and Corner Tests

Considering the behavior of the free edge, the reserve capacity of the parapet edge is not surprising. A major aspect of these test results was the strong interaction of the parapet with the slab. This was illustrated by load-deflection stiffnesses, which for both specimens were highest for the parapet edge tests, by the fact that for specimen 1 all three parapet edge tests involved the formation of wide inclined cracks over the entire height of the parapet, that failure was not attained for either specimen at midspan, even though comparable interior tests failed at loads lower or not much higher than the maximum applied, and from the observation in specimen 2 that failure in the corner test occurred only when the deck vertically separated from the parapet.

COMPARISON TO PROTOTYPE LOADS

The maximum measured loads are converted to full scale equivalents in the righthand portion of Table 1, by multiplying test results by the square of the scale factor. The AASHTO axle load for a single axle with dual wheels is 32 kips, implying 16 kips on each dual tire pattern. This is less than the maximum applied load for any of the single imprint tests, less than one-half the load for any of the dual imprint tests, and less than one-fourth the load for any of the four imprint (tandem) tests. Evaluating the results in a somewhat different manner, the heaviest commercial axle is rated at 23 kips. Applying a safety factor of 2.5 results in an ultimate axle load of 57.5 kips, which translates into 28.75 kips on one dual tire formation, 57.5 kips on two dual tire formations, and 115 kips total on an entire tandem assembly. All of the full-scale maximum applied loads in Table 1 exceed these levels.

CONCLUSIONS

Load tests of the first two of seven concrete bridge deck specimens with 0.3% orthotropic reinforcement top and bottom produced the following general results:

(1) All tests on both models indicated large amounts of reserve strength beyond that associated with the formation of the flexural yield line pattern, even though they incorporated transverse span-to-thickness ratios of 18 and 20, significantly in excess of the maximum of 15 allowed by the Ontario code.

(2) Those tests involving slab failure involved a punching type failure, accompanied by large amounts of deformation.

(3) Even tests on a free edge of the slab (overhang without parapet) indicated considerable reserve strength beyond the load corresponding to the formation of the flexural yield pattern. The stiffness levels and linearity of response for these tests suggested significant transfer of load directly into the longitudinal girder.

(4) Tests on the parapet overhang were characterized by the strong interaction of both the parapet and the longitudinal girder with the slab. In several tests wide inclined cracks appeared in the parapet, and in one corner test, failure occurred by vertical separation of the slab from the parapet.

ACKNOWLEDGMENTS

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Table 1 Summary of Maximum Loads and Deflections

Position	Load Pattern (Number of Imprints)	Specimen	Test	Test Result				Test Result Converted to Full Scale*	
				Yield Load** (kips)	Maximum Load (kips)	Failure?	Maximum Deflection (in.)	Maximum Load (kips)	Maximum Deflection (in.)
Interior	Single	1	1	30	46	Yes	1.52	149	2.7
	Double		4	32.5	72	Yes	2.35	233	4.2
	Double		5	25	46	Yes	2.12	149	3.8
	Single		8	27.5	37.5	Yes	2.00	122	3.6
	Quadruple	2	1	40	70	No	1.78	309	3.7
	Quadruple		3	40	70	No	1.41	309	3.0
	Quadruple		6	35	70	Yes	1.30	309	2.7
	Quadruple		8	35	69	Yes	1.43	304	3.0
Free Midspan	Single	1	2	25	37.5	Yes	1.83	122	3.3
	Quadruple	2	2	25	50	No	0.77	221	1.6
Free Corner	Single	1	9	10	19.5	Yes	0.68	63	1.2
	Quadruple	2	5	25	58	Yes	0.73	256	1.5
Parapet Midspan	Single	1	3	46	50	No	0.68	162	1.2
	Single		7	25	50	No	0.57	162	1.0
	Quadruple	2	4	25	60	No	0.71	265	1.5
Parapet Corner	Single	1	6	32.5	44	Yes	0.58	143	1.0
	Quadruple	2	7	25	55	Yes	0.48	243	1.0

* Test result multiplied by the scale factor for deflection and by the square of the scale factor for load (Scale factor equals 1.8 for Specimen 1 and 2.1 for Specimen 2).

** Load at which flexural cracking pattern was fully developed.

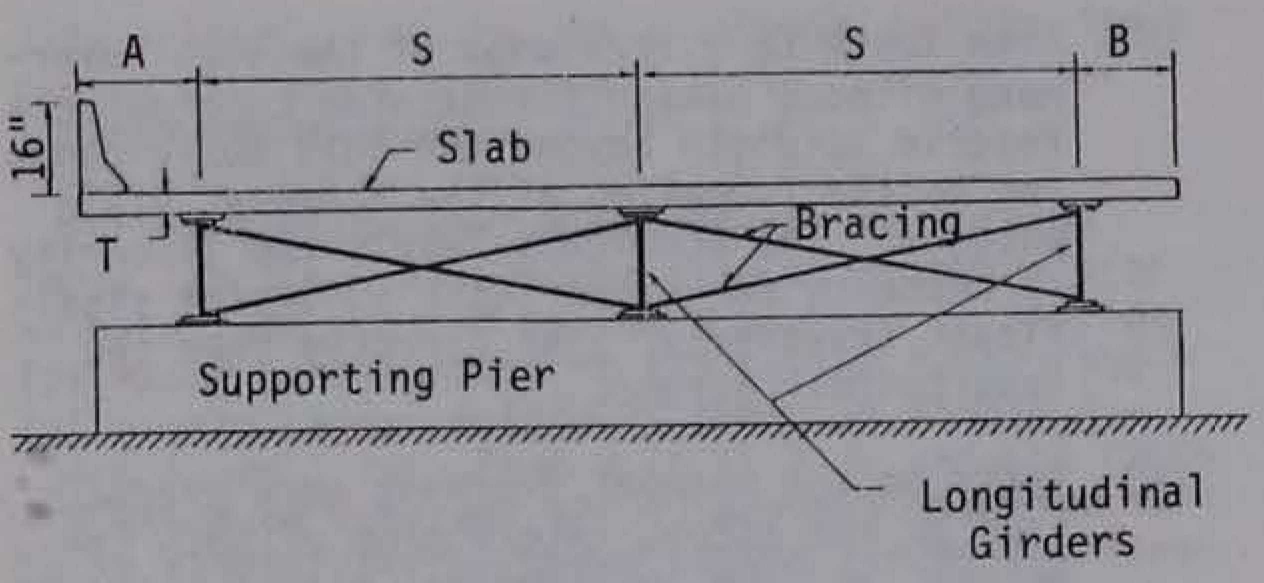

Figure 1 Section Properties for Decks with Steel Girders

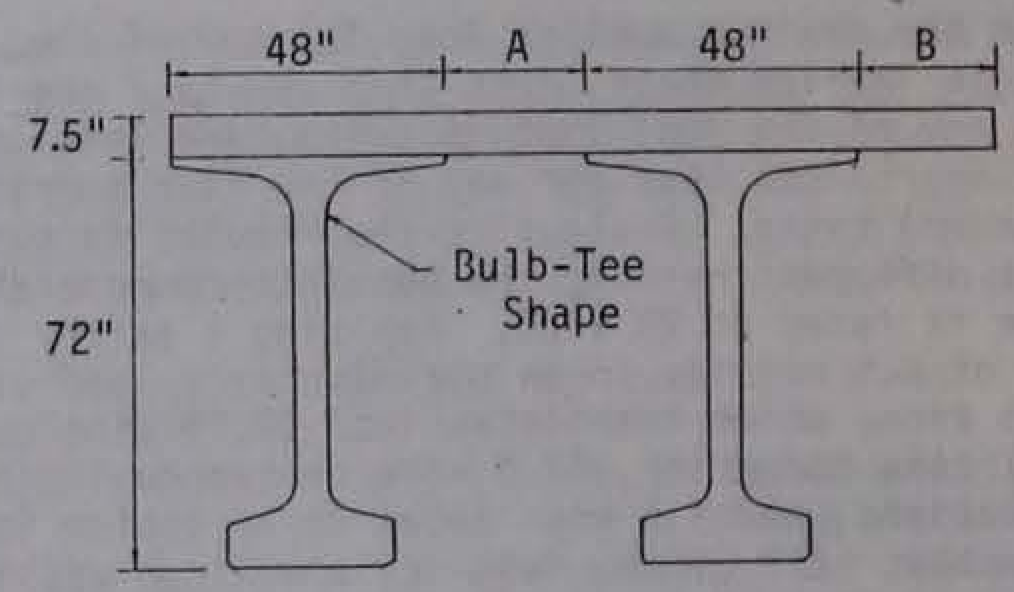

Figure 2 Section Properties for Decks with Bulb-Tee Girders

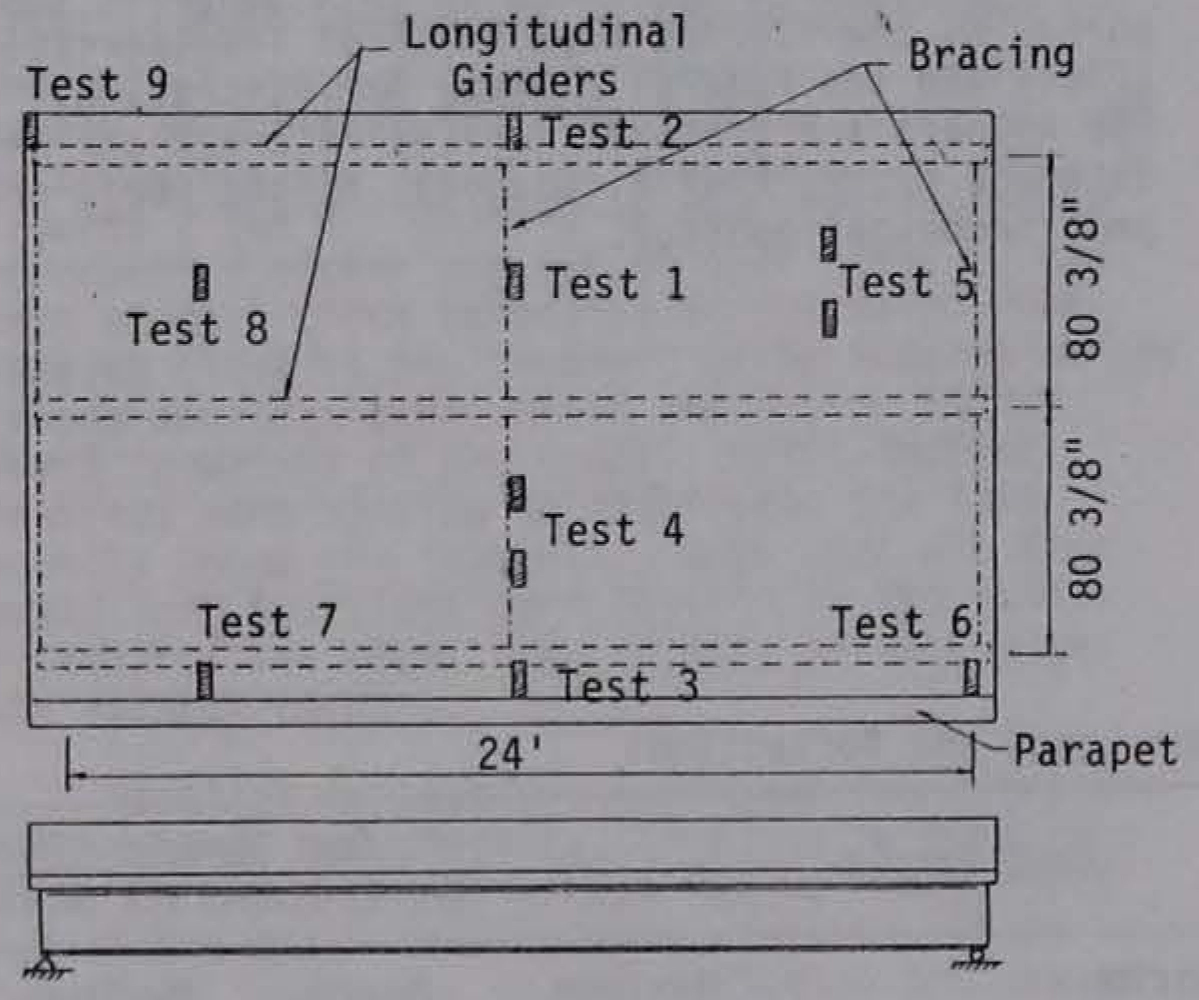

Figure 3 Loading Positions for Specimen 1

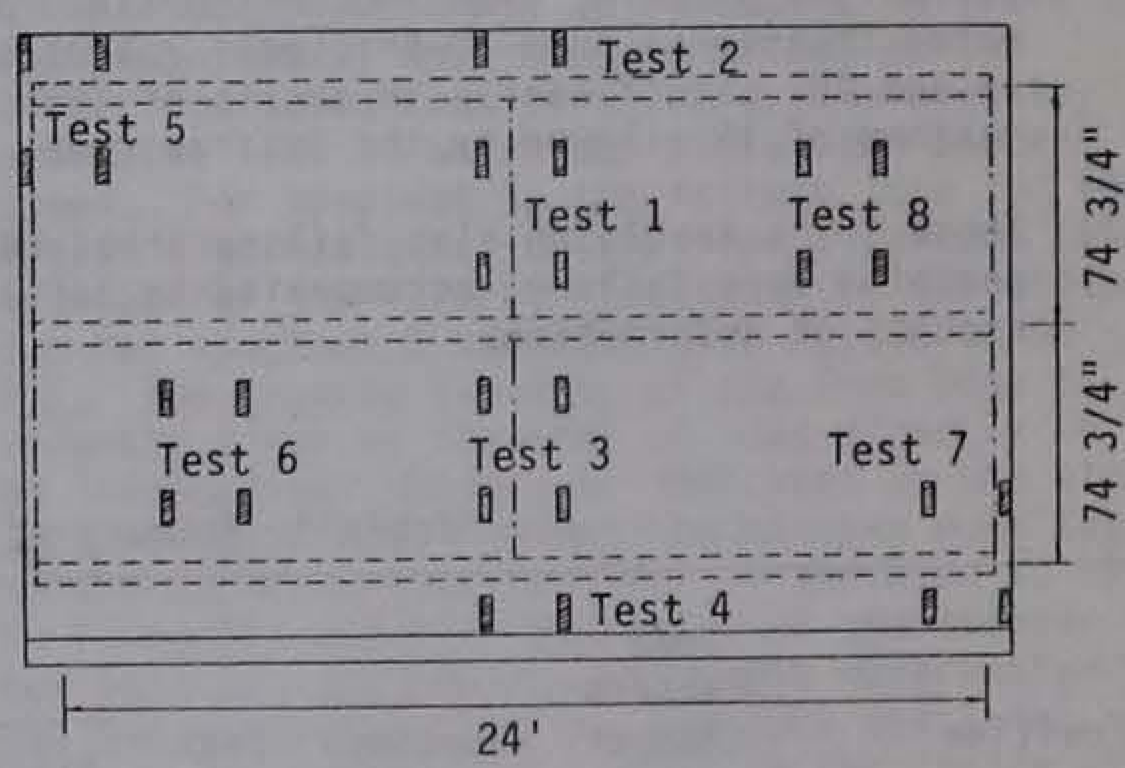

Figure 4 Loading Positions for Specimen 2

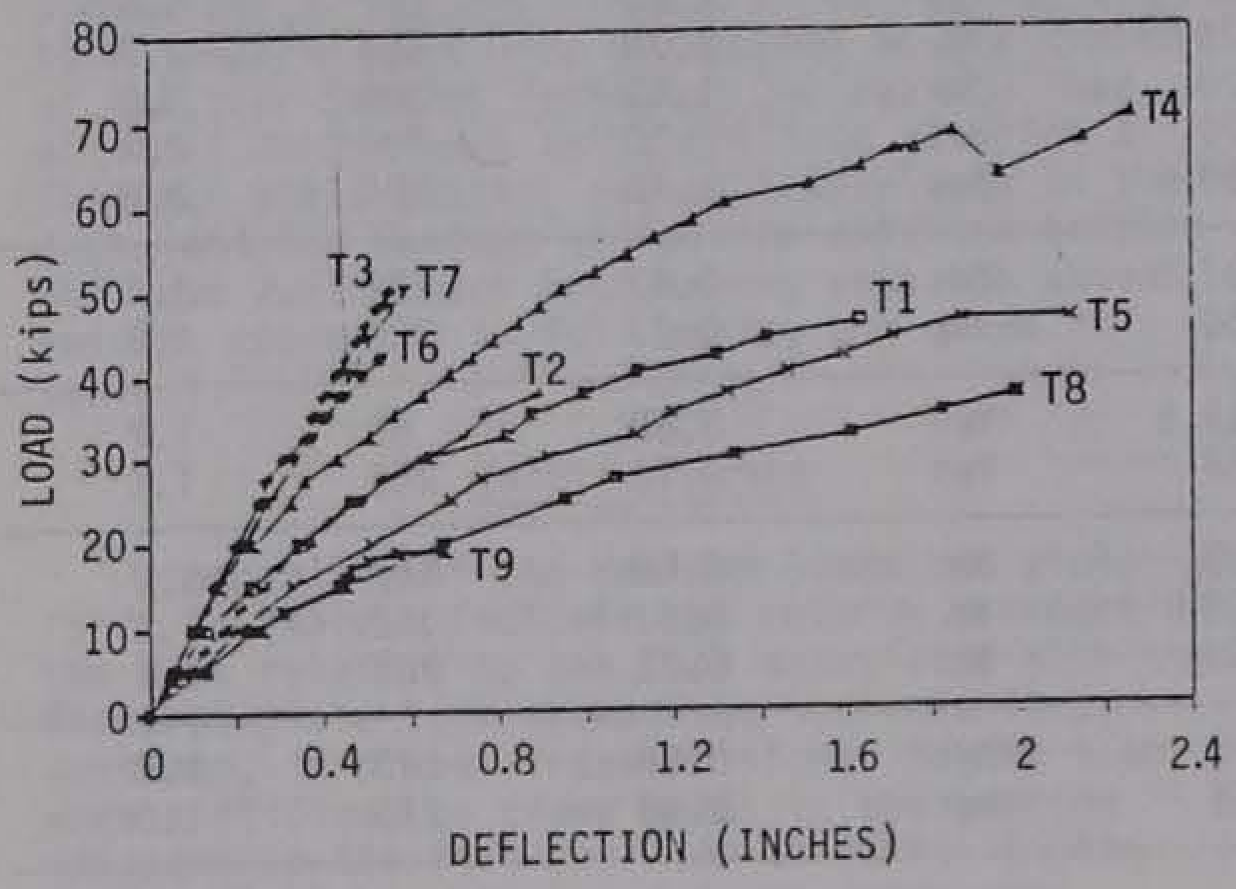

Figure 5 Load-Deflection Results for the Various Test Positions on Specimen 1

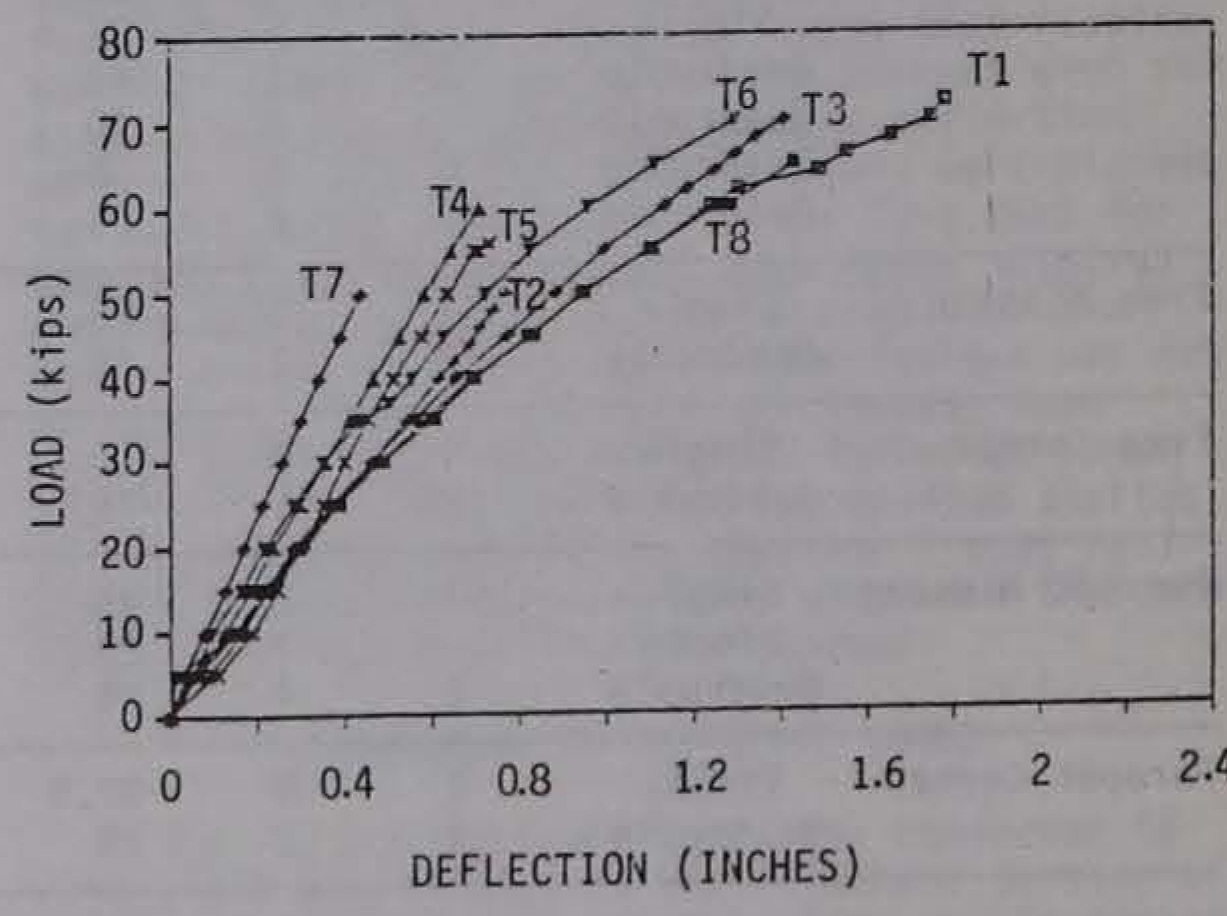

Figure 6 Load-Deflection Results for the Various Test Positions on Specimen 2

RELIABILITY AND REDUNDANCY ASSESSMENT OF PRESTRESSED TRUSS BRIDGES

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SYNOPSIS

Methods for the structural stiffness analysis, the assessment of the reliability and redundancy of prestressed trusses are suggested. The stiffness matrices of straight, one-drape and two-drape tendon layouts are developed. Design guidelines and equations are given. Internal prestressing is discussed. The effect of cable layout and ductility on the reliability and redundancy of prestressed trusses is investigated. Prestressing enlarges the elastic range, increases fatique resistance, redundancy, reliability, and reduces deflection and member stresses.

STIFFNESS ANALYSIS OF PRESTRESSED TRUSSES

The derivation of the stiffness matrices of the tendons is based on the direct stiffness method. Every tendon layout is treated as a separate member like any other truss member. The tendon force is assumed to be constant throughout the length of the tendon regardless whether the tendon is straight or draped. A tendon layout need not coincide with truss members. Tendon ends are to be anchored to truss joints, and in the case of a draped tendon, where a pulley is used, the pulley must be attached to a truss joint.

The effect of prestressing on truss bridges is a function of the truss type, tendon layout and magnitude of the prestressing force. A closed form solution for the relationship between the cross-sectional area, the prestressing force of the tendon and the desired final member stress, after prestressing, is derived for a statically determinate truss. The required tendon cross-sectional area A_c is given by

$$A_c = \frac{T_D + T_L - f_m A_m}{f_t} \qquad (1)$$

Where T_D and T_L are the dead and live loads, f_m is the final required member stress, A_m is the truss member cross-sectional area and f_t is the allowable stress for tendon. The required prestress force P_F is given by

$$P_F = (T_D + T_L - f_m A_m)\left[\frac{T_D + A_m (f_t - f_m)}{T_D + T_L + A_m (f_t - f_m)}\right] \qquad (2)$$

The analysis of a prestressed truss can be divided into two stages. In the first stage, an analysis is performed using the dead load of the truss and the prestressing load applied to the truss without considering the tendon stiffness. In the second analysis stage, the live load and the stiffness matrix of the tendon are considered. The final solution is achieved by imposing the solutions of the two stages.

For the purpose of illustration, the effect of prestressing on a statically determinate truss is evaluated using a truss shown in Fig. 1. The data of the truss members and the prestressing cable are summarized in Table 1. For the truss members, the mean compression capacity is considered as the smaller of the mean yielding capacity in compression and mean buckling capacity.

The truss is prestressed using a straight cable, which coincides with the entire bottom chord of the truss as shown in Fig. 1. The effect of prestressing on the truss member forces is summarized in Table 1.

COMPONENT RELIABILITY

The objective of reliability analysis is to insure the event (R > L) throughout the useful life of an engineering system where, the strength or capacity of a structural component R and load effect L are random variables and are assumed to be statistically independent. This assurance is possible only in terms of the probability P(R > L). Conversely, the probability of the complimentary event P(R < L) is the corresponding measure of unreliability or risk of failure. Therefore, the probability of failure P_F of the structural element according to the following performance function:

$$G = R - L \qquad (3)$$

is given by

$$P_F = P(R < L) \qquad (4)$$

For normally distributed R and L, and a linear performance function of R and L, the probability of failure is given by

$$P_F = 1 - \Phi \left[\frac{\mu_R - \mu_L}{(\sigma_R^2 + \sigma_L^2)^{0.5}} \right] \tag{5}$$

In which σ_R and σ_L are the standard deviations of R and L, respectively, μ_R and μ_L are the mean values of R and L, respectively, and Φ is the cumulative probability distribution function of the standard normal.

SYSTEM RELIABILITY OF PRESTRESSED TRUSSES

In this study, two potential failure modes are considered: yielding or buckling of a truss member. The failure event E_i can be expressed as

$$E_i = (R \text{ and } L \text{ such that } G_i < 0) \tag{6}$$

Failure modes of structural elements can be classified into ductile and brittle. The distinction between brittle and ductile failure modes of elements in the reliability analysis of a statically determinate truss system is irrelevant. The total system fails as soon as one element fails regardless whether it is brittle or ductile.

The exact probability of failure or reliability of the system depends on the correlation levels among the failure modes and generally is difficult to evaluate. Therefore, approximations are necessary and the probability of failure or reliability is evaluated in terms of lower and upper bounds. The probability of failure of a nonredundant system can be bounded for small P(E) as follows :

$$\max_{i=1}^{n} P(E_i) \le P(E) \le \sum_{i=1}^{n} P(E_i) \tag{7}$$

where P(E) is the probability of occurrence of the event E. The lower bound in Eq. 7 represents the case of perfect correlation among all the failure modes of the elements, i.e., the correlation coefficient, ρ, between any pair of elements is equal to 1. The upper bound corresponds to the case of no correlation among all failure modes of the elements, i.e., $\rho = 0$.

The effect of prestressing on the reliability of the statically determinate truss system is evaluated, for the purpose of illustration, using the statically determinate symmetrical truss shown in Fig. 1. A normally distributed resistance R with a coefficient of variation of 0.10 and a normally distributed load effect L with a coefficient of variation of 0.25 are assumed for all truss elements. A normally distributed resistance R is assumed for the prestressed cable layout shown in Fig. 1 and a coefficient of variation of R is assumed to be 0.10.

The addition of the prestressing cable changes the condition of the truss to a statically indeterminate one. Failure of any truss member results in deformation in the truss and the loss of the prestressing force. The event tree modeling technique is used to evaluate the reliability of the truss system after prestressing. All the possible failure modes of the redundant system are evaluated. The event tree model is shown in Fig. 2 with an initiating event, E, and a number of possible failure paths, n. Every path j consists of a possible sequence of events, E_{ji}, i=1,.., k. For a given path to occur, a sequence of subsequent events in the event tree must occur. For example, the path shown in dotted lines in the event tree represents failure of the truss system due to failure of the cable and member (1). These subsequent events are mutually exclusive. The probability associated with the occurrence of a specific path is the product of the sequential conditional probabilities of all the events on that path as follows:

$$P(\text{path } j) = \prod_{i=1}^{k} P(E_{ji}) \tag{8}$$

where $P(E_{ji})$ is the conditional probability of occurrence of $i^{\underline{th}}$ event in path j given that all the events $E_{j1}, \ldots, E_{j(i-1)}$ have occurred. This means that the occurrence event of path j is given by the intersection of all the events of the path.

$$\text{Path } j = \bigcap_{i=1}^{k} E_{ji} \tag{9}$$

The lower bound of the probability of failure of a redundant system is the probability of the most likely failure path, i.e., the largest probability of occurrence of all paths. The upper bound is the probability of the union of all the failure paths of all members that define the intact structure. The probability of failure of any path is given by Eq. 8. Therefore, the probability of system failure P_{Fs} can be bounded as

$$\max_{j=1}^{n} P(\text{path } j) \le P_{Fs} \le 1- \prod_{j=1}^{n}(1-P(\text{path } j)) \tag{10}$$

where the lower bound corresponds to $\rho = 1$ and the upper bound corresponds to $\rho = 0$.

The results of the reliability analysis are summarized in Table 2. The first row shows the results of the truss before prestressing, where the components perform brittle failure. The second and third rows represent the results of the prestressed truss, where the components are assumed to exhibit brittle and ductile failure modes, respectively. The upper bound, the lower bound and the average probability of failure, and the corresponding probability of survival of the original truss system are shown in the second, third and fourth columns, respectively.

Due to the assumption of the ductile failure, the cable and truss members maintain their maximum load-bearing capacity after failure. Then, the mean value of the load effect on other truss members, μ_L, is given by

$$\mu_L = \mu_{L1} + \mu_{L2} \tag{11}$$

where μ_{L1} is the mean value of the effect of the live and dead loads, and μ_{L2} is the mean value of the effect of the yielding load of the failed elements. The variance of the load effect σ^2_L is given by

$$\sigma^2_L = (\delta_L * \mu_{L1})^2 + (\delta_R * \mu_{L2})^2 \tag{12}$$

where δ_L and δ_R are the coefficients of variation of the load L and the strength R, respectively.

It is evident that a great improvement in the system reliability due to prestressing is achieved. For the truss discussed herein, the bottom chord members are the critical ones. On the other hand, the effect of the assumption of ductile components, that are in parallel, on the system reliability as compared to the assumption of brittle components is small. However, the advantages of the ductile behavior are apparent on the level of the component and path reliabilities. For example, in the path shown with dotted lines in the event tree, the probability of failure of member (1), given that the cable (c) failed in brittle mode, is equal to $1.35\text{x}10^{-3}$. The probability of failure of member (1) given that the cable (c) failed in ductile mode, dropped to $9.20\text{x}10^{-5}$. The probability of failure of the path (c-1) is equal to $1.15\text{x}10^{-8}$ for brittle failure of components, while the probability of failure of the same path for ductile failure of components is equal to $7.85\text{x}10^{-10}$.

SYSTEM REDUNDANCY OF PRESTRESSED TRUSSES

The classical definition of the degree of redundancy in a structural system is the number of stress resultants and reactions that cannot be determined from the conditions of statics alone. This definition does not constitute an adequate measure of system redundancy as demonstrated by the event tree shown in Fig. 2.

Three different prestressing cable layouts for the truss of Fig. 1 are considered, case (a) where one drape cable coincides with members (11 and 12), case (b) where two drape cable coincides with members (7, 2, 3 and 9) in addition to case (c) with straight cable as shown in Fig. 1. These cable layouts result in a truss that is indeterminate to the first degree. Based on the reliability analysis of the truss system, it is evident that redundancy depends on which member fails first and whether an alternative load path exists, and if the other members in that path can sustain the load. It is, therefore, particularly significant that a new viewpoint for the structural redundancy based on system reliability concept be considered.

The effect of prestressing on the redundancy of a truss system is evaluated using a redundancy factor. The redundancy factor is defined as the ratio of the average system probability of failure before prestressing to the average system probability of failure after prestressing. In an equation form, the redundancy factor, R, is given by

$$R = \frac{P_{Fs}\ \ \text{(before prestressing)}}{P_{Fs}\ \ \text{(after prestressing)}} \tag{13}$$

In which R is the redundancy factor of the system, and P_{Fs} is the average of the upper and lower bounds of the probability of failure of the system. For an effective prestressing configuration, the redundancy factor R must be larger than one.

The redundancy factors R for the prestressed truss using these cable layouts are shown graphically in Fig. 3. Therefore, prestressing results in an improvement in the system redundancy, i.e., R is larger than one.

CONCLUSIONS

Prestressing statically determinate trusses using internal prestressing tendons results in a great force reduction in only the tension members. If the tendon coincides with truss members, then these members are the only members affected by the prestressing force.

A method for determining bounds on the system reliability of prestressed trusses based on event tree analysis is suggested. A great improvement in the reliability and the redundancy of the truss system can be achieved by prestressing. Ductile failure modes result in improvement in the component and the path reliabilities.

It is clearly demonstrated that the approach of evaluating the redundancy of a prestressed truss system based on the reliability of the truss system, provides realistic estimates. Although prestressing the truss with different cable layouts makes it redundant to the first degree according to the classic definition of redundancy, the results show that every cable layout result in different load paths which reflects different levels of structural redundancy.

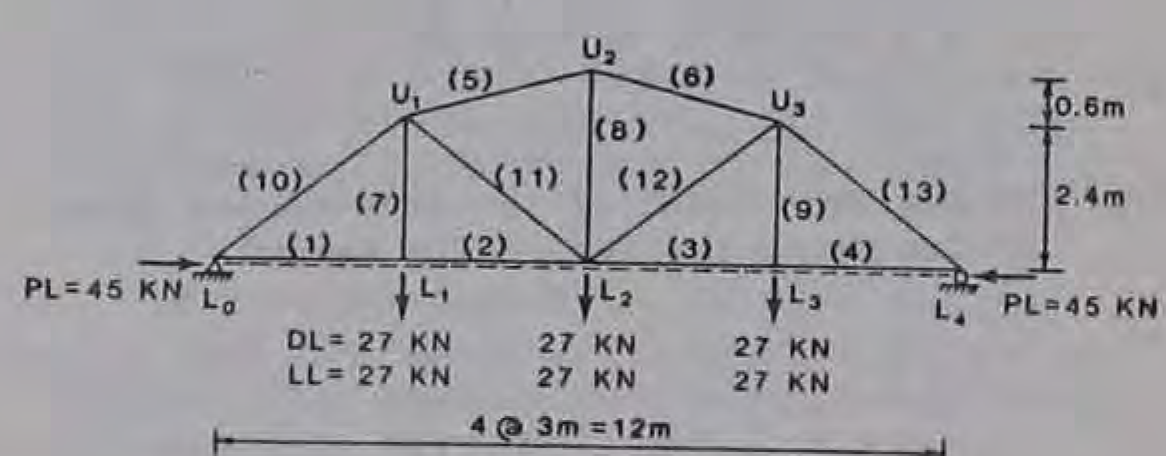

Fig. 1-Prestressed Truss Configuration

TABLE 1-Truss Members Data and Force Summary

Members and Cable (1)	Cross Sectional Area (CM^2) (2)	Mean Tensile Capacity (KN) (3)	Mean Compressive Capacity (KN) (4)	Force before Prestressing, Tension (Compression) (KN) (5)	Force after Prestressing, Tension (Compression) (KN) (6)
L_0 L_1	7.87	196	178	100	48
L_1 L_2	7.87	196	178	100	48
U_1 U_2	30.97	165	231	(109)	(109)
L_1 U_1	4.52	111	53	53	53
L_2 U_2	4.52	116	58	43	43
L_0 U_1	30.97	120	285	(128)	(128)
L_2 U_1	0.90	22	31	8	8
Cable	2.90	147	—	—	52

TABLE 2-Safety Measures of the Prestressed Truss

Truss System (1)	Probability of Failure P_F x10^{-3} / Probability of Survival $(1-P_F)$: Upper Bound (2)	Lower Bound (3)	Average Bound (4)
Original Truss brittle	7.390 (0.992)	1.350 (0.9986)	4.360 (0.9956)
Prestressed Truss brittle	1.990 (0.998)	0.434 (0.9996)	1.212 (0.9987)
Prestressed Truss ductile	1.990 (0.998)	0.434 (0.9996)	1.212 (0.9987)

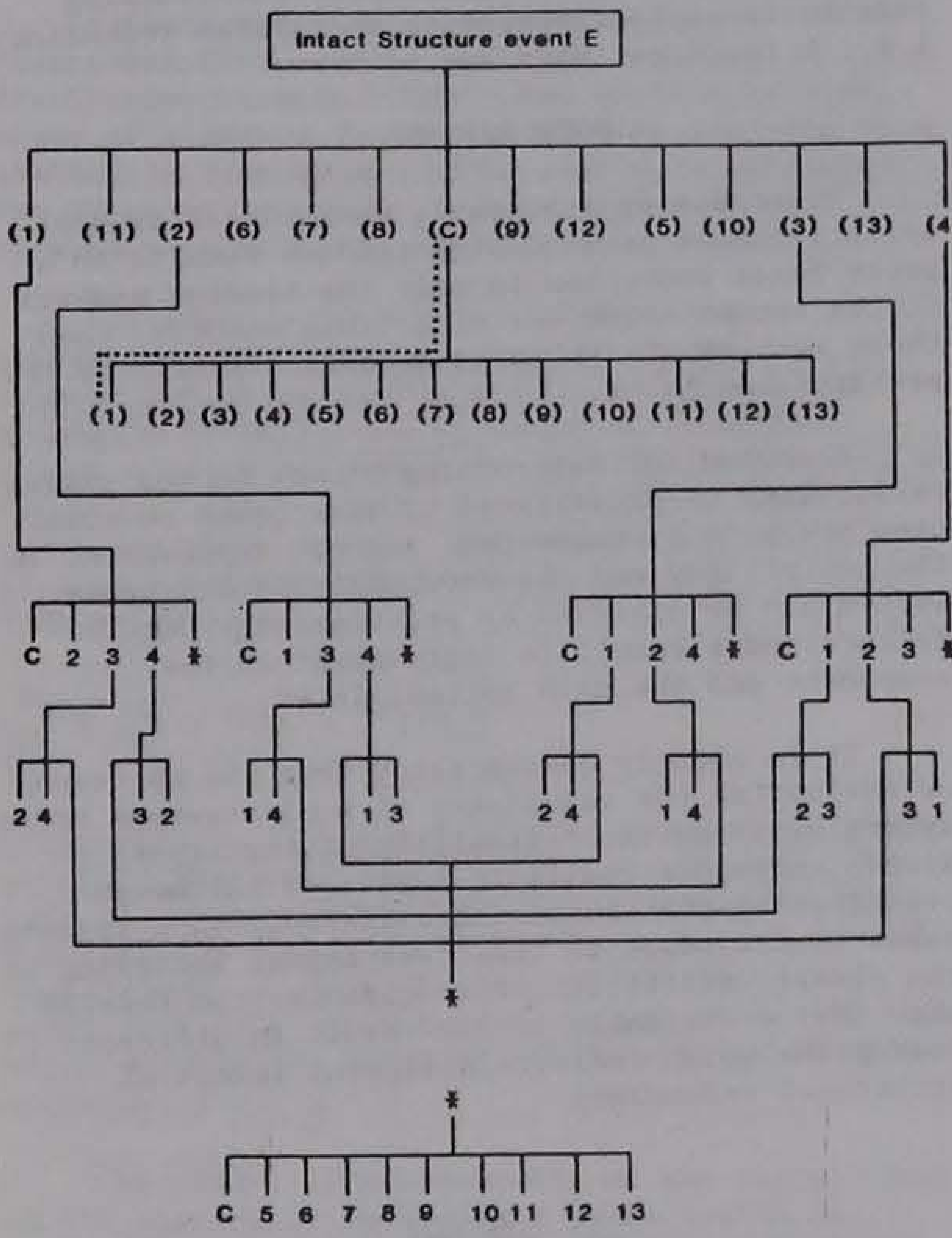

Fig. 2-Event Tree Model of the Prestressed Truss System Using Straight Cable Layout

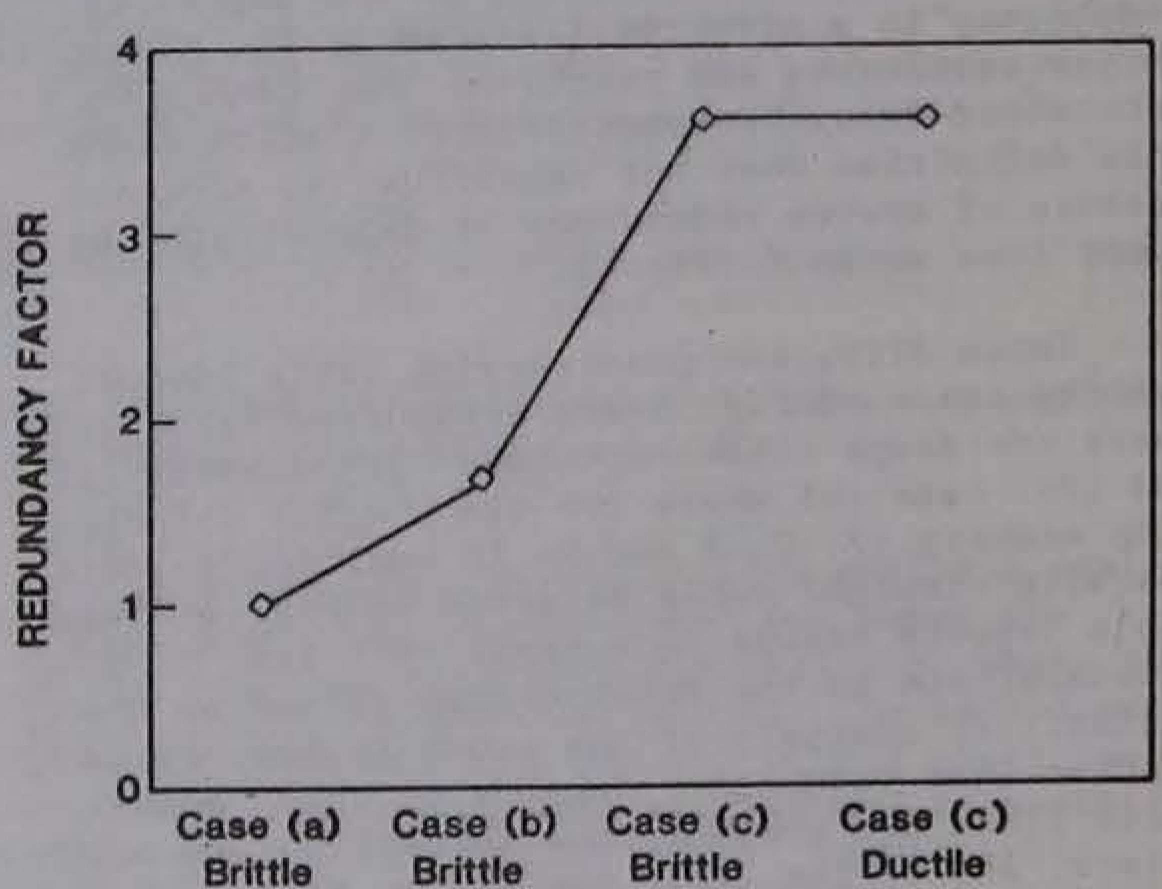

Fig. 3-Redundancy Factor of Prestressed Truss System

SCALED MODEL TESTING OF BRIDGE HINGED PIERS SUBJECTED TO LATERAL LOADS

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SYNOPSIS

An experimental study of reinforced concrete one-way hinges has been undertaken with the aim of developing design guidelines for shear response of hinges subjected to lateral load. This article briefly describes a preliminary study of four scaled columns and the current study in which some 20 specimens are planned to be tested.

INTRODUCTION

One-way hinge details are incorporated at the base of many reinforced concrete bridge columns to eliminate moment transfer to the footing in one direction. No specific guidelines are available to determine the shear strength of reinforced concrete hinges. Consequently, designers have adopted the shear friction method (SFM) for this purpose. The SFM is appropriate for R/C members which have a predetermined shear failure plane and in which the two segments of the member slip relative to each other, thus maintaining a friction force. In bridge piers, there is usually a significant moment present at the hinge area, thus potentially causing a wide crack over a major part of the cross section. Due to a lack of experimental data on shear response of hinges, it has not been possible to quantify the effect of flexural cracking on the shear strength.

A preliminary study of R/C hinges subjected to shear and flexure was recently completed at the University of Nevada, Reno. A more extensive study of R/C one-way hinges is underway. The purpose of this article is to present some of the preliminary results and an outline of the current study.

PRELIMINARY STUDY

Four one-eighth scale specimens were constructed and tested in the laboratory. The test variables and load ing types were chosen to simulate realistic conditions for actual bridge columns.

Test Specimens

The four test specimens were identical. They were designed to produce a one-eighth scale model of the columns in the Rose Creek Bridge in Nevada (Fig. 1) [1]. The cross section of each column specimen was 6" x 12" and was reduced to 2" x 12" in the hinge region (Fig. 2). The reinforcement consisted of six-#2 grade 40 plain bars placed in one row in the strong direction. A normal-weight concrete with a maximum aggregate size of 1/2" was used. The concrete compressive strength for the hinge throat region ranged from 5000 to 6000 psi on the day of testing. Each specimen was cast in two stages. First, the concrete for the middle stub was poured and cured in the moist room for three days. After one day of moist curing, the forms perpendicular to the steel were removed and the middle 2" x 12" shear key on each side was scraped to a roughness amplitude of approximately 0.25". At the end of the three-day period, the side segments were cast and moist-cured for a week.

The steel was distributed in one row, and the bars were spaced at 2" on center. The same steel detail was used through the entire length of each specimen (Fig. 2).

Instrumentation

Electric strain gages were bonded to the bars in the hinge throat region (Fig. 2). The deflection at the center of the middle stub was measured by a dial gage and a Celesco displacement transducer. Other instrumentations included a combination of dial gages and LVDT'S (linear variable differential transformers), placed horizontally to measure the rotation of the "column" segments relative to the middle stub. In addition, vertical dial gages were used to measure the relative slip between the middle stub and the "column" segments in the vertical direction (Fig. 4). The data were collected and stored using a Hewlett Packard (HP) 3054 data acquisition system.

Test Program

Two test variables were considered in the study: (1) cyclic versus monotonic loading and (2) the shear-span to depth ratio. The effect of the first variable involved two specimens, CH1 and CH4, which were subjected to monotonic and cyclic loads, respectively. The shear-span to depth ratio in both of these specimens was three.

Three specimens were involved in the study of the effect of the shear-span to depth ratio, CH1, CH2, and CH3. The ratios for these specimens were three, two, and one, respectively. In the Rose Creek Bridge, the effective shear-span to depth ratio changes from 3.0 to 1.2. The ratios in testing of small-scale specimens were chosen to correspond to this range.

The test specimens were positioned as shown in Fig. 2 except that the supports were moved inward in CH2 and CH3 to produce the desired shear span. The supports for CH4 were detailed to allow for load reversal. The loads were applied in the form of a point load on the middle stub. All the loads were effectively static and were applied to produce small force or displacement increments.

Results from the Preliminary Study

Figure 3 shows the load deflection responses for all of the specimens. In all specimens, the initial part of the load-deflection diagram was linear up to the yielding of the first layer of steel. No clear cracking point could be identified on the curves, indicating that all the hinge throats were initially cracked.

The responses for specimens CH1 and CH2 were similar in that both specimens were able to sustain relatively large displacements without any significant loss of strength. The ultimate load, however, was higher for CH2 because of the shorter span length. The ultimate load for CH3 was 12,500 lbs. which was approximately twice that of CH2. This was due to the fact that the l/h (Fig.2) in CH3 was one-half that in CH2. Specimen CH3, however, showed a rapid strength deterioration as soon as the peak load was reached.

The fact that the load capacities for CH1, CH2, and CH3 were inversely proportional to l/h and that the hinge throat in these specimens had approximately the same moment capacity indicates that the flexural capacity and not that the shear capacity controlled the ultimate load. This is particularly interesting for CH3 where the shear-span to total depth ratio was one. The rapid deterioration of the load capacity in CH3, however, indicates the dominance of shear as soon as the peak load was developed.

The cyclic response of specimen CH4 shows relatively significant pinching especially at high amplitudes. The observed pinching effect is due to the separation of the columns from the middle stub and a reduction in stiffness before the closing of the cracks.

The common method for the design of column hinges is the shear friction method [2]. While this method is appropriate for elements with aspect ratios of 0.5 or less, its applicability to bridge column hinges is questionable. Based on the shear friction theory, the shear strength of the hinge throat in the test specimens is 12,900 lbs. The test for CH3 indicated that the actual shear strength may be approximately 7,000 lbs. The reason for the difference was quite clear in the experiments - the wide flexural crack allowed for friction to take place only over a short segment of the hinge throat section. According to the shear friction theory, all six bars crossing the hinge throat will produce compression perpendicular to the hinge section. Detailed information about the results may be found in Ref. 3.

CURRENT STUDY

Approximately twenty one-sixth scale specimens are to be tested in the current study. All these specimens will utilize deformed bars and will be subjected to the simultaneous effect of axial load, shear, and flexure. The specimens are categorized into two groups: (1) standard (SD) specimens and (2) modified (MF) specimens. At the time of this writing the SD specimens are being constructed. The dimensions and details for the MF specimens will be finalized after the SD group has been tested.

Standard (SD) Specimens

Figure 4 shows the details of specimens in the SD group. Four specimens are in this group, SD1M, SD2M, SD1C and SD2C. The hinge sections are approximately one-sixth scale models of the pier hinges in the Rose Creek Bridge. The numeral in each specimen designation refers to the shear span-to-depth ratio. The last letter identifies the loading type: M for monotonic and C for cyclic loading. A normal weight concrete with a specified compressive strength of 4000 psi will be used. The steel crossing the hinge throat consists of 6-#3 deformed bars with a yield stress of 60 ksi. The bars will be placed in a single row to model common R/C hinge details. The specimens will be constructed in an upright position. The concrete will be placed in the footing first and it will be cured for three days before the concrete for the column is poured. A rough surface at the interface of the footing and column will be provided.

Instrumentation for SD Group

Each specimen will be instrumented by electrical strain gages placed on the dowels on and in the vicinity of the hinge throat (Fig. 5). In addition, a combination of LVDT's (linear variable differential transformers), displacement potentiometers, and dial gages will be used to measure displacement at the loading point, rotation, and the slippage of pier relative to the footing. The data will be collected using an HP-3054 data acquisition system. A 9000 Series HP microcomputer will be used to trigger data collection, data storage, and to plot load-deformation response during the test.

Test Program for SD Group

All specimens will be subjected to a constant axial load of 26 kips using a Riehle 300,000 lb. machine. This load would produce an axial stress on the hinge throat which is approximately the same as the dead load stress on the Rose Creek Bridge hinges. The specimens will also be subjected to monotonic or cyclic horizontal loads applied at 6" from the top of the specimens using a 55 kip MTS actuator. All the loads will be applied at small increments and at a slow rate. The footing in each specimen will be anchored to the loading frame to avoid rocking.

Specimens SD1M and SD2M will be loaded to failure (defined at a point where the load capacity has dropped to eighty-five percent of the peak load). The other two specimens will be subjected to several cycles of deformations of increasing amplitudes and ductility level. No particular earthquake load history will be simulated.

Anticipated Results

The major differences between the study of the SD Series and preliminary study (CH Series) are (1) the scale in the SD specimens is one-sixth as opposed to one-eighth scale used in the pilot study, (2) deformed bars will be used in the SD specimens whereas plain bars had been used in the CH Series, and (3) no axial forces were applied on the CH Specimens, but the SD Series will be subjected to a constant axial force. The first factor is not expected to lead to any major difference in the behavior. The use of deformed bars is likely to reduce the bond slip deformations and crack width at the hinge throat. This could improve the shear resistance of the specimens. The presence of the axial force is expected to enhance the shear strength of hinge throat.

DISCUSSION

Designers determine the shear strength of reinforced concrete one-way hinges using the shear friction method (SFM) due to the absence of a more specific method for hinges. Preliminary tests at the University of Nevada, Reno have indicated that the SFM can overestimate the shear strength of hinges by as much as 100 percent. Test results have shown that the mechanism for shear resistance is drastically different from what is assumed in the shear friction theory. To more realistically model the behavior of R/C hinges subjected to lateral loads, an extensive study of some 20 specimens is in progress at the University of Nevada, Reno. It is expected that the new test data will lead to a preliminary design guideline for shear behavior of common R/C hinge details and will identify means to improve the behavior.

ACKNOWLEDGEMENTS

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as PI's. The authors are grateful for the support. The preliminary tests were carried out by Jim Orie currently with the California Department of Transportation.

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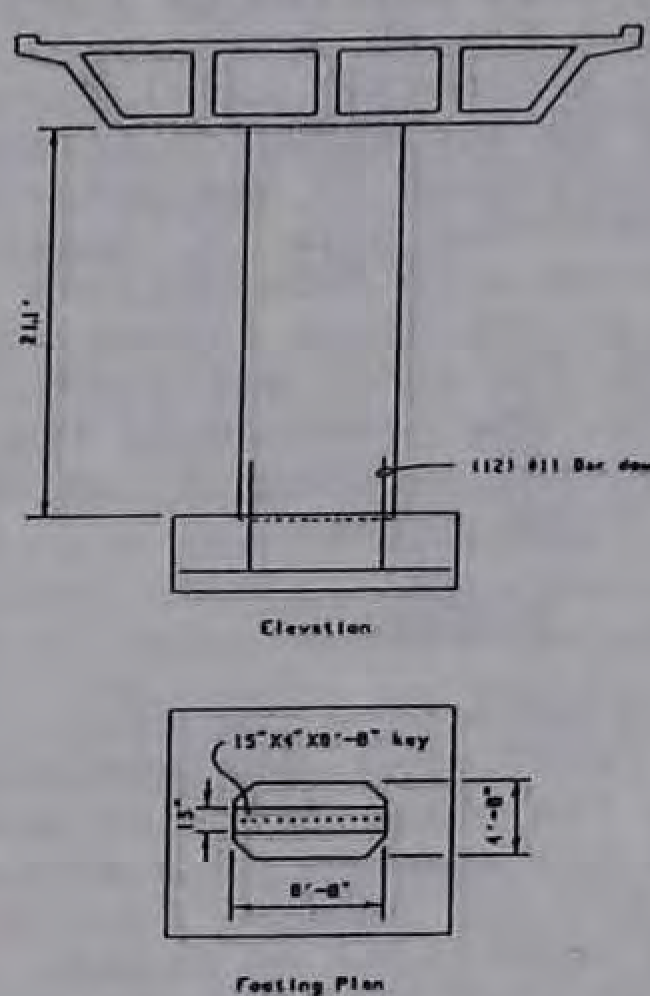

Fig. 1 - Piers in the Rose Creek Bridge

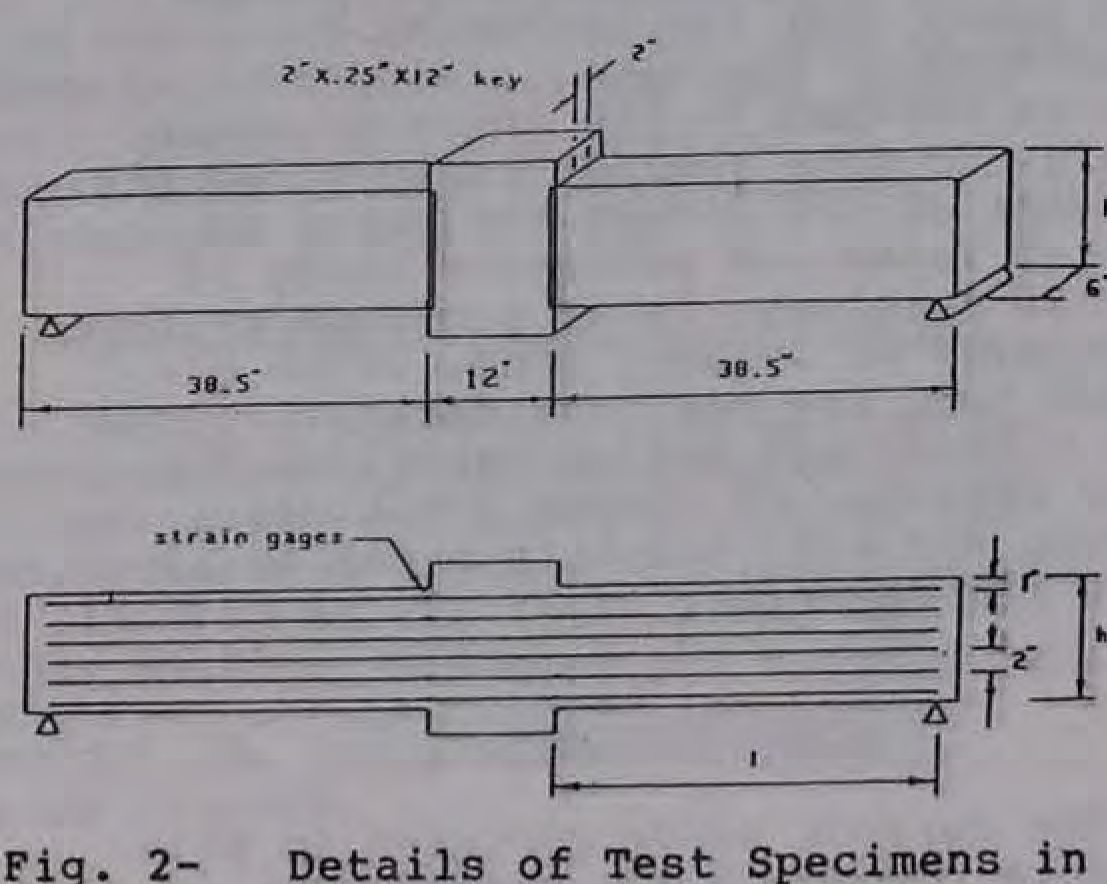

Fig. 2- Details of Test Specimens in the Pilot Study

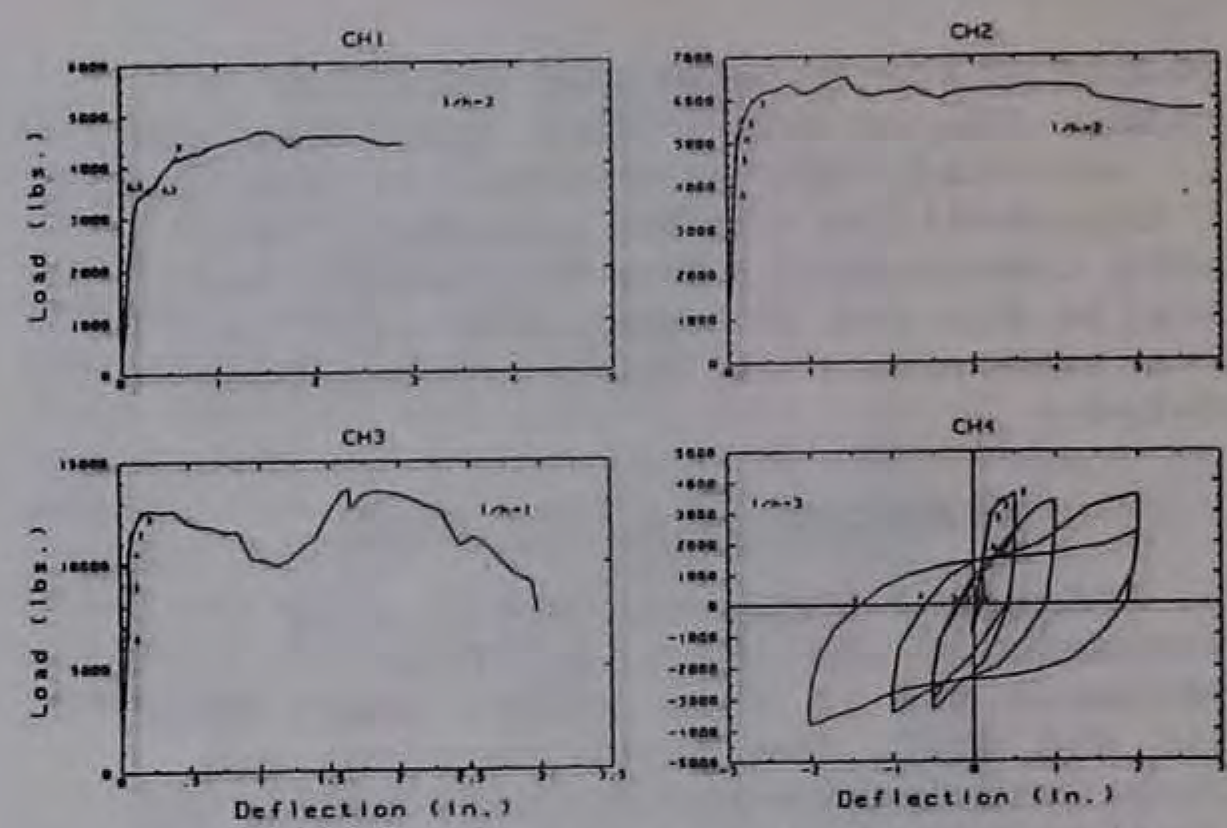

Fig. 3 - Load-Deflection Diagrams for Specimens in the Pilot Study

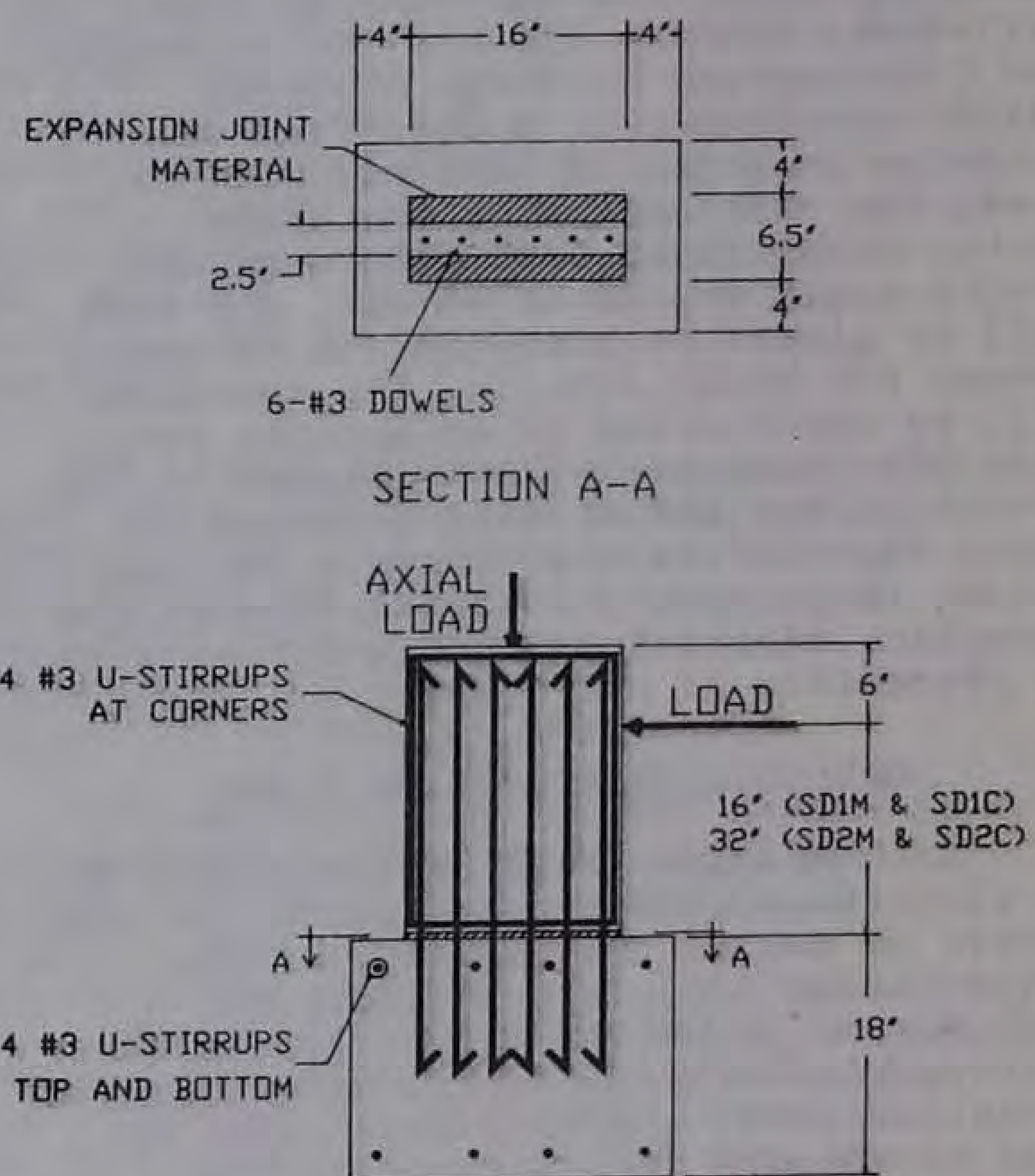

Fig. 4 - Details of Specimens in the SD Group

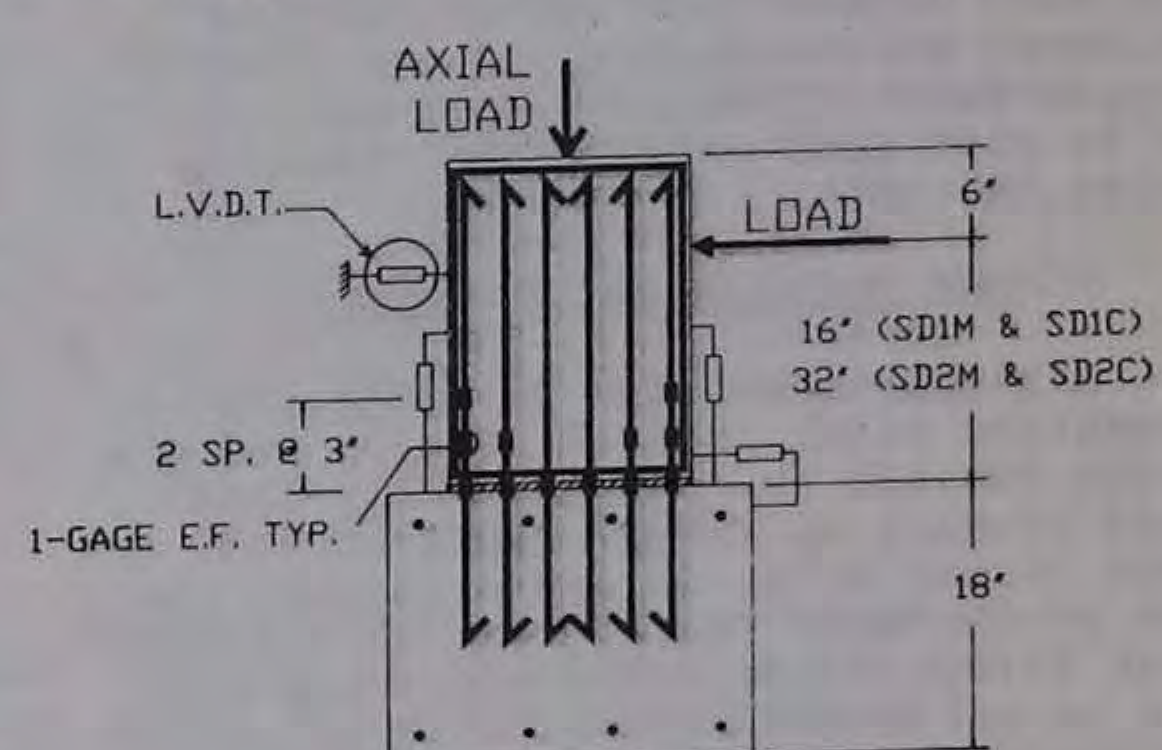

Fig. 5 - Instrumentation in the SD Group

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