

NATIONAL COOPERATIVE
HIGHWAY RESEARCH PROGRAM REPORT

✓**280**

**GUIDELINES FOR EVALUATION AND
REPAIR OF PRESTRESSED
CONCRETE BRIDGE MEMBERS**

TRANSPORTATION RESEARCH BOARD EXECUTIVE COMMITTEE 1985

Officers

Chairman

JOHN A. CLEMENTS, *President, Highway Users Federation for Safety and Mobility*

Vice Chairman

LESTER A. HOEL, *Hamilton Professor and Chairman, Department of Civil Engineering, University of Virginia*

Secretary

THOMAS B. DEEN, *Executive Director, Transportation Research Board*

Members

RAY A. BARNHART, *Federal Highway Administrator, U.S. Department of Transportation (ex officio)*
JOSEPH M. CLAPP, *Vice Chairman-Corporate Services, Roadway Services, Inc. (ex officio, Past Chairman, 1984)*
LAWRENCE D. DAHMS, *Executive Director, Metropolitan Transportation Commission, Berkeley, California (ex officio, Past Chairman, 1983)*
DONALD D. ENGEN, *Federal Aviation Administrator, U.S. Department of Transportation (ex officio)*
FRANCIS B. FRANCOIS, *Executive Director, American Association of State Highway and Transportation Officials (ex officio)*
WILLIAM J. HARRIS, JR., *Vice President for Research and Test Department, Association of American Railroads (ex officio)*
RALPH STANLEY, *Urban Mass Transportation Administrator, U.S. Department of Transportation (ex officio)*
DIANE STEED, *National Highway Traffic Safety Administrator, U.S. Department of Transportation (ex officio)*
ALAN A. ALTSHULER, *Dean, Graduate School of Public Administration, New York University*
DUANE BERENTSON, *Secretary, Washington State Department of Transportation*
JOHN R. BORCHERT, *Regents Professor, Department of Geography, University of Minnesota*
ROBERT D. BUGHER, *Executive Director, American Public Works Association*
ERNEST E. DEAN, *Executive Director, Dallas/Fort Worth Airport*
MORTIMER L. DOWNEY, *Deputy Executive Director for Capital Programs, New York Metropolitan Transportation Authority*
JACK R. GILSTRAP, *Executive Vice President, American Public Transit Association*
MARK G. GOODE, *Engineer-Director, Texas State Department of Highways and Public Transportation*
WILLIAM K. HELLMAN, *Secretary, Maryland Department of Transportation*
LOWELL B. JACKSON, *Secretary, Wisconsin Department of Transportation*
JOHN B. KEMP, *Secretary, Kansas Department of Transportation*
ALAN F. KIEPPER, *General Manager, Metropolitan Transit Authority, Houston*
HAROLD C. KING, *Commissioner, Virginia Department of Highways and Transportation*
DARRELL V MANNING, *Adjutant General, Idaho National Guard, Boise*
JAMES E. MARTIN, *President and Chief Operating Officer, Illinois Central Gulf Railroad*
FUJIO MATSUDA, *Executive Director, Research Corporation of the University of Hawaii*
JAMES K. MITCHELL, *Professor, Department of Civil Engineering, University of California*
H. CARL MUNSON, JR., *Vice President for Strategic Planning, The Boeing Commercial Airplane Company*
MILTON PIKARSKY, *Distinguished Professor of Civil Engineering, City College of New York*
WALTER W. SIMPSON, *Vice President-Engineering, Norfolk Southern Corporation*
LEO J. TROMBATORE, *Director, California Department of Transportation*

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Transportation Research Board Executive Committee Subcommittee for NCHRP

JOHN A. CLEMENTS, *Highway Users Federation for Safety and Mobility (Chairman)* FRANCIS B. FRANCOIS, *Amer. Assn. of State Hwy. & Transp. Officials*
LESTER A. HOEL, *University of Virginia* RAY A. BARNHART, *U.S. Dept. of Transp.*
JOSEPH M. CLAPP, *Roadway Services, Inc.* THOMAS B. DEEN, *Transportation Research Board*

Field of Design

Area of Bridges

Project Panel C12-21(1)

CLELLON L. LOVEALL, *Tennessee Dept. of Transportation (Chairman)* WALTER PODOLNY, JR., *Federal Highway Administration*
JAMES COLVILLE, *University of Maryland* RAYMOND J. SCHUTZ, *Protex Industries, Inc.*
ARTHUR J. HAYWOOD, *Florida Dept. of Transportation* CARL E. THUNMAN, JR., *Consultant*
ELDON D. KLEIN, *California Dept. of Transportation* CRAIG A. BALLINGER, *FHWA Liaison Representative*
HEINZ P. KORETZKY, *Pennsylvania Dept. of Transportation* GEORGE W. RING, III, *TRB Liaison Representative*
ADRIAN G. CLARY, *TRB Liaison Representative*

Program Staff

ROBERT J. REILLY, *Director, Cooperative Research Programs* CRAWFORD F. JENCKS, *Projects Engineer*
ROBERT E. SPICHER, *Deputy Director* R. IAN KINGHAM, *Projects Engineer*
LOUIS M. MACGREGOR, *Administrative Engineer* HARRY A. SMITH, *Projects Engineer*
IAN M. FRIEDLAND, *Projects Engineer* HELEN MACK, *Editor*

✓ NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
REPORT

280

GUIDELINES FOR EVALUATION AND REPAIR OF PRESTRESSED CONCRETE BRIDGE MEMBERS

G. O. SHANAFELT and W. B. HORN
Consulting Engineers
Olympia, Washington

RESEARCH SPONSORED BY THE AMERICAN
ASSOCIATION OF STATE HIGHWAY AND
TRANSPORTATION OFFICIALS IN COOPERATION
WITH THE FEDERAL HIGHWAY ADMINISTRATION

AREAS OF INTEREST

STRUCTURES DESIGN AND PERFORMANCE
MAINTENANCE
(HIGHWAY TRANSPORTATION)
(PUBLIC TRANSIT)
(RAIL TRANSPORTATION)

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C.

DECEMBER 1985

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as: it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state, and local government agencies, universities, and industry; its relationship to its parent organization, the National Academy of Sciences, a private, nonprofit institution, is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the bases of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the Academy and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are the responsibilities of the Academy and its Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

NCHRP REPORT 280

Project 12-21(1), FY'82
ISSN 0077-5614
ISBN 0-309-04013-2
L. C. Catalog Card No. 85-51921

Price \$9.20

NOTICE

The project that is the subject of this report was a part of the National Cooperative Highway Research Program conducted by the Transportation Research Board with the approval of the Governing Board of the National Research Council, acting in behalf of the National Academy of Sciences. Such approval reflects the Governing Board's judgment that the program concerned is of national importance and appropriate with respect to both the purposes and resources of the National Research Council.

The members of the technical committee selected to monitor this project and to review this report were chosen for recognized scholarly competence and with due consideration for the balance of disciplines appropriate to the project. The opinions and conclusions expressed or implied are those of the research agency that performed the research, and, while they have been accepted as appropriate by the technical committee, they are not necessarily those of the Transportation Research Board, the National Research Council, the National Academy of Sciences, or the program sponsors.

Each report is reviewed and processed according to procedures established and monitored by the Report Review Committee of the National Academy of Sciences. Distribution of the report is approved by the President of the Academy upon satisfactory completion of the review process.

The National Research Council was established by the National Academy of Sciences in 1916 to associate the broad community of science and technology with the Academy's purposes of furthering knowledge and of advising the Federal Government. The Council operates in accordance with general policies determined by the Academy under the authority of its congressional charter of 1863, which establishes the Academy as a private, nonprofit, selfgoverning membership corporation. The Council has become the principal operating agency of both the National Academy of Sciences and the National Academy of Engineering in the conduct of their services to the government, the public, and the scientific and engineering communities. It is administered jointly by both Academies and the Institute of Medicine. The National Academy of Engineering and the Institute of Medicine were established in 1964 and 1970, respectively, under the charter of the National Academy of Sciences.

The Transportation Research Board evolved from the 54-year-old Highway Research Board. The TRB incorporates all former HRB activities and also performs additional functions under a broader scope involving all modes of transportation and the interactions of transportation with society.

Special Notice

The Transportation Research Board, the National Academy of Sciences, the Federal Highway Administration, the American Association of State Highway and Transportation Officials, and the individual states participating in the National Cooperative Highway Research Program do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

Published reports of the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Transportation Research Board
National Academy of Sciences
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

Printed in the United States of America

FOREWORD

*By Staff
Transportation
Research Board*

This report provides specific guidance for the evaluation and repair of damaged prestressed concrete bridge girders. Bridge and maintenance engineers will be able to make immediate use of the findings and guidance included in the report. Researchers will find the report useful in the further study of effective means to evaluate and repair prestressed concrete bridge members.

Prestressed concrete bridge members often are subjected to accidental damage because of vehicle impact, mishandling, or fire. Repair methods need to be identified and evaluated for various levels of damage. As a consequence, a two-phase project (NCHRP Project 12-21, "Evaluation of Damage and Methods of Repair for Prestressed Concrete Bridge Members") was initiated under the NCHRP and performed by G. O. Shanafelt and W. B. Horn, Consulting Engineers, of Olympia, Washington.

Phase I synthesized available information including recommendations for assessing and selecting repair techniques. The results of the Phase I research have been published in *NCHRP Report 226*, "Damage Evaluation and Repair Methods for Prestressed Concrete Bridge Members."

Phase II, the subject of this report, was designed to further evaluate the more promising methods of repair and to prepare guidelines for damage evaluations and repair techniques. External post-tensioning was also studied as a bridge strengthening technique. The research effort and commentary on the findings are detailed in Chapter One. The results of laboratory tests on a girder that was repaired using various techniques (external post-tensioning, internal splicing of severed strands, and a metal sleeve splice) are documented in Appendix A. These repair-in-place techniques are intended to be cost-effective alternatives to replacing damaged members. Guidelines for assessing and calculating the extent of damage and for designing and performing the appropriate repair techniques are covered in Chapter Two. Each technique is addressed thoroughly; numerous detailed drawings, photographs, and computations are included to supplement the written text. Examples of good and bad techniques are provided to illustrate their usefulness. It is emphasized that the information contained in Chapter Two is of immediate use to bridge and maintenance engineers. Researchers will find that both this report and *NCHRP Report 226* provide the support and encouragement for studies on the continued evolution of repair techniques.

CONTENTS

SUMMARY	1
CHAPTER ONE Introduction	2
Background	2
Commentary	5
CHAPTER TWO Manual of Recommended Practice	9
Guidelines for Inspection of Damage	9
Office Responsible for Inspection	9
Initial Inspection and Action	9
Inspection Sequence and Record	9
Inspection Equipment and Skills	10
Inspection Report	10
Monitoring of Repairs	11
Guidelines for Assessment of Damage	11
Assessment of Damage by Whom and Where	11
Strength of Damaged Member	11
User Inconvenience and Speed of Repairs	12
Fracture Critical Members	12
Primary Members	12
Secondary Members	12
Minor Concrete Nicks, Spalls, and Scrapes	12
Concrete Gouges	12
Concrete—Cracks	13
Concrete—Loose or Shattered	13
Strands or Other Prestressing Elements	13
Web Reinforcement	14
Member Displacements	14
Guidelines for Selection of Repair Method	15
External Post-Tensioning	15
Internal Splicing	15
Metal Sleeve Splice	15
Combining Splice Methods	15
Complete Replacement	16
Service Load Capacity	16
Ultimate Load Capacity	16
Overload Capacity	16
Fatigue	18
Durability	20
Cost	20
Aesthetics	21
Repair Method to Consider	21
Guidelines for Repair of Damage	21
Office Responsible for Repair Method	21
Accomplishment of Work	21
Equipment Required	22
Concrete Nicks, Spalls, Scrapes, and Gouges	22
Concrete Cracks	22
Major Loss of Concrete	22
Prestressing Steel and Concrete Repair	23
Post-Tensioning	24
Metal Sleeve Splice	27
Internal Strand Splices	30
Girder Replacement	38
Fire Damage	39

APPENDIX A Load Tests	41
Summary	41
Test Girder Design	43
Component Testing	48
Single Strand Internal Splice	48
Two-Strand Internal Splice	48
Concrete Corbels	50
Full-Scale Load Tests	57
Effect of Creep on Slab Strains	57
Effect of Strain Gage Accuracy	57
Unusual Strain Gage Readings	58
Load Test 1	59
Load Test 2	64
Load Test 3	65
Load Test 4	65
Load Test 5	66
Load Test 6	72
Load Test 7	72
Load Test 8	74
Load Test 9	76
Load Test 10	81
APPENDIX B Notations and Definitions	83
APPENDIX C References	84

ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 12-21(1) with Willis B. Horn and George O. Shanafelt as co-principal investigators. W. B. Horn and G. O. Shanafelt, self-employed consulting engineers, are the authors of this report.

The authors express appreciation to the staff members of the Concrete Technology Corporation of Tacoma, Washington, where all load tests were performed. Their cooperation in the testing phase was excellent. Special thanks are extended to Arthur R. Anderson for his advice and

consultations, and Richard G. Anderson, who was responsible for overall management of the test program.

Steven J. Cospser, Development Engineer, was responsible for the supervision of the testing procedures. James Chittenden, Laboratory Superintendent, was responsible for ensuring that work was performed properly, and also for the photographic coverage. Steven Cospser and James Chittenden provided valuable assistance during all test setups and testing. Harold J. Jobse, Managing Director of Research and Development, assisted in the initial development of the test program.

GUIDELINES FOR EVALUATION AND REPAIR OF PRESTRESSED CONCRETE BRIDGE MEMBERS

SUMMARY

This report is primarily a practical user's manual for dealing with accidentally damaged prestressed concrete bridge members. It is the second phase of a two-phase research program conducted under NCHRP Project 12-21. The first phase report, *NCHRP Report 226*, found that 40 states reported a total of 191 bridges damaged per year. Overheight vehicles caused 80 percent of the damages. Other reported causes of damage were through fire and during manufacture. The types of prestressed bridge most frequently damaged severely are precast prestressed I-beam bridges. Prestressed box girder bridges are usually subject to minor damage only. Twenty states reported using epoxy injection for repair-in-place. Only 9 states used steel strengthening when repairing-in-place. The Phase I report dealt primarily with state practices used to assess damage and make repairs. Existing techniques were evaluated and areas in need of investigation were identified. Additionally, several promising methods of repair were included that had not been tested.

There is no known published report that deals with the entire problem of accidental damage to prestressed concrete bridge members. This lack of information and guidelines has resulted in repair techniques that are not always appropriate for particular damage incidents. The decision to repair or replace a damaged member is often based on an evaluation made under the emergency pressure to restore the facility to use. The findings indicate that some repair-in-place methods do not adequately restore members to their original condition, and some members have been replaced where repair-in-place techniques would have been more appropriate.

The material provided in the following chapters of this report establishes guidelines for evaluation and repair of damaged prestressed concrete bridge members. The guidelines are based on information developed during the Phase I report plus extension of the development and evaluation of promising methods of repair during the Phase II research. Several methods of repair were tested as splices for severed strands of a full-scale AASHTO Type III girder. The tested methods included adding external post-tensioning, internal splicing of broken strands, and a metal sleeve splice. Tests were also made with added external post-tensioning to the girder without severed strands. This method could be used to strengthen existing girders carrying excessive live load. Ten separate tests were made for various combinations of strengthening, splicing, and number of severed strands. Maximum load for the first nine tests was approximately equal to 75 percent of the calculated ultimate load capacity. The girder was loaded to its calculated ultimate load capacity for Test No. 10. Component testing was used to evaluate three concrete corbels with differing methods of attachment to the girder; it was also used to evaluate two methods of internal strand splicing. The test results are detailed in Appendix A.

The following chapters of this report are organized in a format that will facilitate the use of the guidelines. Background information, including details of the research effort through which the manual was developed, and commentary on the findings that led to the development of the manual are provided in the introduction of Chapter One.

The guidelines in Chapter Two are structured to lead to a logical and practical format to facilitate their use in the inspection and assessment of damage, selection of repair method, and repair of damage. Primary factors included are strength of damaged member, user inconvenience and speed of repairs, durability of repair, strength of repair, relative cost, and aesthetics. Methods of repair include epoxy injection, concrete patching, preloading, internal strand splicing, adding external post-tensioning, metal sleeve splicing, and girder replacement. Each repair technique is addressed in a thorough and logical manner, with a description of its use, detailed construction procedures, and the mathematical calculations required. A feature of the guidelines is the inclusion of examples of good and poor techniques and extensive use of drawings and photographs to supplement the text. In addition, for convenience of the user, notations and definitions are provided in Appendix B. Pertinent references are included in Appendix C.

CHAPTER ONE

INTRODUCTION

BACKGROUND

This report is the second phase of a two-phase program reporting on research that originated under NCHRP Project 12-21, "Evaluation of Damage and Methods of Repair for Prestressed Concrete Bridge Members." The first phase, documented in *NCHRP Report 226*, "Damage Evaluation and Repair Methods for Prestressed Concrete Bridge Members," had as its specific objective the task of synthesizing available information on the subject and of identifying areas in need of further investigation. From the data furnished to that study it was found that the average incidence of damage to prestressed concrete bridge members is approximately 0.86 percent per year. Prestressed concrete bridge members are often damaged by overweight vehicle impact. Damage during fabrication occurs occasionally. Damage due to mishandling during construction has also been noted. Fire damage occurs on rare occasions. Girders may become inadequate for current loads.

Severity of damage varies from minor concrete spalls to complete fracture of the member. It is believed that some bridge engineers have the impression that replacement is the conservative and safe method of repair. Replacement of an entire bridge member is both costly and time consuming, and such replacement may be a complex operation. The entire replacement of a prestressed member may also induce stresses that are very difficult to analyze. At present, the decision to repair or replace a damaged member and the techniques used are determined on the basis of the inspection or the engineer's evaluation of the situation, with little published information available for guidance. Repair-in-place techniques have been successfully used by a few states for restoring strength of girders with broken strands.

This practice is approximately 10 years old; however, it is not a generally accepted method by most highway agencies. It is believed that the primary reason for nonacceptance is the lack of practical guidelines for assessment of damage and methods of repair. The lack of actual testing of these repair methods may also be a contributing factor.

Based on the findings of Phase I, the primary objectives of Phase II were to (1) develop promising methods of repair and perform tests to demonstrate the effectiveness of those methods, and (2) produce a manual of practice for dealing with the problem of damaged prestress concrete bridge members.

The accomplishment of these objectives involved completing the five tasks described as follows:

1. *Evaluate available information on selected repair methods.* The specific objective of the Phase I project was to synthesize available information regarding evaluation of damage and methods of repair for prestressed concrete bridge members and to identify areas in need of investigation. The Phase I report did go a step beyond the current state of the art and offered several promising repair techniques that had not been tested and in some instances had not been previously used. In the conduct of this task all available information regarding the repair methods identified in Phase I (an external post-tensioning system with concrete jacking corbels, single-strand internal splices, two-strand internal splices, and a metal sleeve splice) was carefully evaluated during Phase II. This information included calculated strength, availability of materials, speed of repair, relative cost, range of applicability, and durability of repair.

2. *Develop concepts for improved repair methods and perform component testing as needed.* This task was devoted to developing

all concepts noted in task 1 to the point where the investigators were satisfied that (1) the concept was practical and should be tested, (2) the concept was not practical and further consideration was unwarranted, and (3) ideas for improving the concept had been thoroughly explored. Component testing included full-scale load tests of splices in a tensile test machine. Concrete corbel tests were accomplished by attaching full-scale cast-in-place corbels to the test girder and jacking between corbels.

In achieving the goal of this task, an internal single-strand splice was considered worthy of development and evaluation (see Figs. 19, 20, and 22 in Chapter Two). Ordinary machine shop fabrication was required to develop the components of this splice. The primary concern was access for torqueing and a practical method of measuring stress in the splice. It was proposed that component and full-scale girder testing be performed.

An internal splice that would splice pairs of strands, proposed in Phase I, was also evaluated. A chief concern regarding this splice was the strength required of the small strand grips. It was anticipated that short lengths of strand with small diameter swage fittings could be used. The pairs of strands could then be spliced with ordinary Supreme splice chucks, or equal, as shown in Figures 23 and 24. Component testing of this splice assembly was proposed.

An external post-tensioned system would have wide applicability. Similar versions of this system had been successfully used by the State Transportation departments of South Carolina, Connecticut, and Washington. Figures 9, 10, and 11 in the manual (Chapter Two) show details of high strength rods used for external post-tensioning. Full-scale load tests were performed on this system. Another external post-tensioned system, as shown in Figures 13 and 14, utilizes prestressing strands. These details are very similar to the type used by the State of Washington. Washington's standard prestressed concrete I-beam differs in shape from AASHTO I-beam types. Conceptual study for the external post-tensioned splice system was focused primarily on the best method of attaching jacking corbels to the girder. Corbel reinforcing was evaluated. Full-scale girder tests with post-tensioned 150 K thread bar (ASTM-A722) were performed.

Another concept concerned a metal sleeve splice (Figs. 15, 16, and 17) that might become widely accepted because of its adaptability for splicing a large number of strands and as a web splice. Further study was given to the required extension of the sleeve past the damaged area. A full-scale girder test of this splice was also made.

In studies made, during Phase I, of a post-tensioned steel corbel splice that was developed in that phase, it was apparent that this splice would be relatively expensive. It had the advantage of a lower center of gravity for the post-tensioned rod. However, on the basis of further studies of this splice during task I, full scale testing of the splice was not proposed for this project.

3. Perform full-scale tests to demonstrate the effectiveness of the repair methods developed in task 2. Under this task, an AASHTO Type III girder with a 60-ft span length was tested. The girder had a composite deck slab 90 in. in width and $6\frac{1}{2}$ in. thick, as shown in Figure A-2 in Appendix A. Materials used in fabrication were representative of materials available nationwide. The girder was fabricated at Concrete Technology Corporation's plant in Tacoma, Washington. Aggregate from the Willamette River in Oregon was used. A design strength, f'_c , of 5,000 psi was proposed for the girder. A design strength,

f'_c , of 4,000 psi was proposed for the composite slab. Both component and full-scale girder testing was accomplished at the Concrete Technology Corporation's plant. Figures 1 and 2 show stages of the girder during construction and testing. Note the crack pattern that formed during the first load test. (This firm has an on-going program wherein they not only do testing of their own products, but also do testing for an international

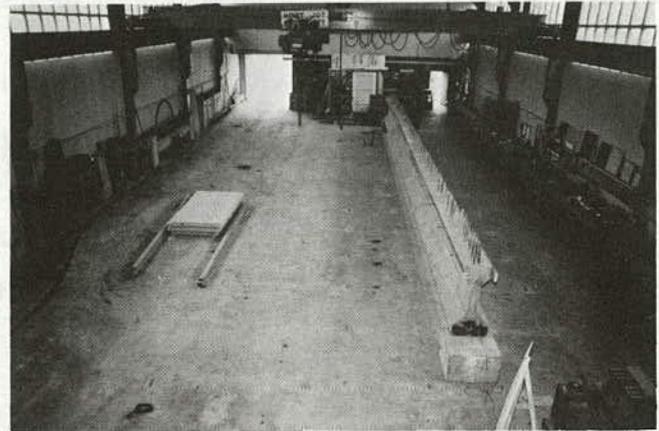


Figure 1(a). Construction of test girder—girder cast and on supports.

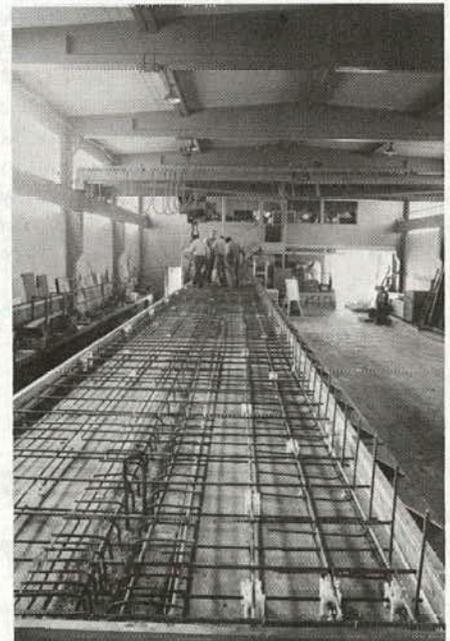


Figure 1(b). Construction of test girder—deck forms and reinforcing steel.

concrete users group of which they are a member.) The following full-scale girder testing program was accomplished:

a. A minimum of two full loading cycles was applied to each of the first nine tests. Tests wherein the girder was uncracked initially or had been patched required three full load cycles. The

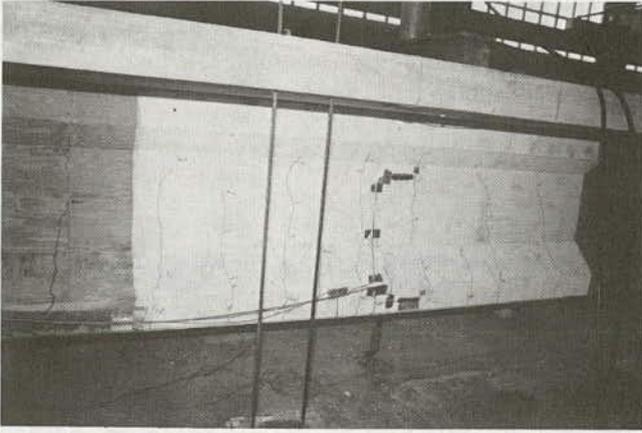


Figure 2(a). Girder during load tests—girder cracks from first load test.

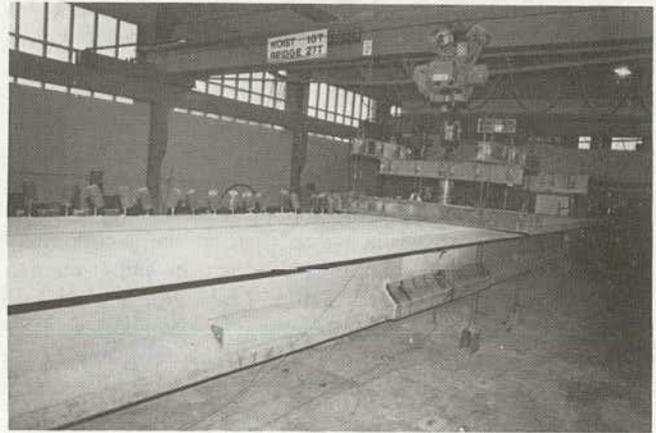


Figure 2(b). Girder during load tests—girder prior to sleeve splice load test 9.

first cycle data, because of the the initial uncracked state of the girder, were not expected to and did not corroborate the data of the initial cracked section cycles. In all nine tests, corroboration of test data between load cycles was accomplished within three cycles. A partial “working in” cycle was used as appropriate prior to each test. Test 10 was a 100 percent ultimate load test with one full cycle.

b. Strains, deflections, and loads were recorded during each cycle.

c. The maximum total load for each of the first nine tests was limited to 75 percent of the calculated ultimate moment capacity. This loading is the maximum overload permitted by the AASHTO “Manual for Inspection of Bridges.” Stresses in post-tensioned high-strength bars were checked to ensure that they did not exceed $0.9 f_t$ (see Appendix B for symbol notations and definitions). Web reinforcing was furnished to limit shear stresses to less than or equal to 75 percent ultimate shear capacity. Maximum deflections were limited to the maximum deflection recorded for Test 1, which was a test of the undamaged girder.

d. The test load was located at centerline of span.

e. Patching portions of concrete was accomplished prior to certain tests as noted. No epoxy injection of girder cracks was performed.

f. The need for preload was determined as a routine step in making girder repairs. It was required and used for Test 5, the internal strand splicing test.

The following tests and procedures were accomplished:

a. Test 1—Set up girder and instrument. Load the undamaged girder to 75 percent of the calculated ultimate load capacity.

b. Test 2—Add concrete corbels and post-tension high-strength bars. Load test. This test consisted of a girder with no broken strands, strengthened by adding external post-tensioning.

c. Test 3—Disconnect high strength bars and load test. This test is the same as Test 1 except the girder was uncracked initially during Test 1. The cracks were closed at initial loading for all tests.

d. Test 4—Break out specified concrete to sever 4 strands, equal to 25 percent of the 16-strand total. Load test.

e. Test 5—Splice four strands with the single-strand internal splice and preload as required. Patch the girder and release preload after patch has gained required strength. Load test.

f. Test 6—Reconnect and post-tension high-strength bars. This test is the same as Test 5 except for added external post-tensioning. Load test.

g. Test 7—Disconnect bars, break out concrete, sever the four strands spliced in Test 5. Load test.

h. Test 8—Preload not required, patch the girder, post-tension external high-strength bars. Load test.

i. Test 9—Disconnect bars, break out concrete, sever six strands out of a total of 16, patch the girder, and install the metal sleeve splice. Load test.

j. Test 10—Load test the sleeve spliced girder to 100 percent of calculated ultimate moment capacity.

Load strain and load deflection curves were prepared for all tests. All tests were evaluated.

4. *Prepare manual of recommended practice.* This task resulted in the manual provided in Chapter Two. This user-oriented manual is addressed to bridge engineers. It is believed that portions will be useful to highway administrators, researchers, members of repair crews, and others. It is a practical user’s guide for dealing with accidentally damaged concrete bridge members. The guidelines are based on information evaluated from both Phase I and Phase II research. The manual covers all aspects of the problem, including assessment and repair procedures.

The outline for sections of the manual of recommended practice is as follows:

- Inspection of damage
- Guidelines for assessment of damage
- Selection of repair method
- Guidelines for repair of damage

5. *Preparation of final report.* The final task was the preparation of the manual, the major portion of which consists of the guidelines in Chapter Two. In main, information needed to use the manual is included therein. The load test results of promising methods of repair, including applicable technical data, tables,

and charts, are provided in Appendix A. Additional information of interest is contained in Appendixes B and C—notations and definitions in Appendix B and pertinent references in Appendix C. The commentary that follows pertains to material that may not be required by users of the manual but provides additional basis for the manual guidelines and should be reviewed by all users.

COMMENTARY

The findings of Phase I plus the testing required by this project in Phase II show that repair-in-place methods performed in accordance with the guidelines presented in Chapter Two can be used successfully to restore strength and durability of damaged prestressed concrete bridge members. These techniques are less time consuming and less costly than replacement. With more bridges being designed for live-load continuity, repair-in-place methods for these bridges will be even less expensive when compared to girder replacement. Complex engineering calculations are not required to develop specific repair-in-place repairs; however, straightforward calculations should be made to ensure that strength and durability have been restored. For all bridge design, the Standard Specifications for Highway Bridges (1) adopted by the American Association of State Highway and Transportation Officials (AASHTO) sets forth the minimum design standards and should be followed in all repair work.

Inspection Procedures

From information gained through this research, the damage inspection phase should be differentiated and separated from the engineering assessment phase. Inspection should report the factual, pertinent damage information. Damage assessment should then be accomplished through logical engineering calculations.

It is also advised that personnel responsible for inspection and damage assessment develop their specific procedures prior to the time when it is required to respond to a damage incident. This will ensure more uniform treatment and more orderly progress. Establishment of procedures, without pressure from an emergency, should result in best practices. The state bridge engineer is normally responsible for the preparation of repair plans. Since the bridge engineer is responsible for repair design, this office should be responsible for inspection. One major component for assessment of damage is the field inspection of damage. The bridge engineer's office should be best qualified for inspection of damage. Where organization constraints place the responsibility with others, cooperative action with bridge engineer personnel should be a requirement. The inspection of damaged prestressed concrete bridge members should be performed by, and/or with direct supervision by structural bridge engineers.

Existing Assessment of Damage

The assessment of damage appears to be largely judgmental, supplemented by brief calculations. This procedure is adequate during the inspection phase to determine whether to restrict traffic or close the bridge. These preliminary calculations are

generally acceptable to determine the necessity of preliminary strengthening in order to prevent further damage to the bridge. During final assessment of damage, a complete evaluation of strength should be made. Stresses should be compared with the design stresses. All preliminary calculations and decisions made during the inspection phase should be reviewed.

Documentation

Documentation of damage assessment and repair appears to be disorganized. There is a tendency to rely on the knowledge and expertise possessed by one or two individuals in a particular state. When these individuals retire or change jobs, substantial knowledge may be lost. A cross-referenced record system should be established to provide access to all damage incidents. The system should provide ready access to the inspection reports. Damage should include type of structure, cause of damage, and type of member. Assessment calculations, repair calculations, and repair details should be included. Good documentation will build up a background of experience that can be very useful in future damages.

Selection of Repair Method

A principal finding of this report is that very few states use repair-in-place techniques that require preload or strengthening by prestressing. Many states may believe that these techniques do not restore girder strength or that these techniques are time consuming and costly when compared to replacement. The full-scale load tests performed in Phase II demonstrate the effectiveness of repair-in-place methods. External post-tensioning was used to restore loss of strength due to severing four strands (25 percent of the 16 strands total). External post-tensioning was also used to demonstrate its effectiveness in increasing the load capacity of a girder without severed strands. Internal strand splicing was developed and used to splice four severed strands. A metal sleeve was used to splice six severed strands (37.5 percent of the 16 strand total). The test results were completely satisfactory.

The research also indicated that consideration should be given to the combining of repair methods for severely damaged members. Where many strands are damaged, internal splices could be installed in addition to the use of an external post-tensioning system or a steel sleeve splice. Combining repair methods could provide a fast repair and reduce user inconvenience and cost.

The scope of the project did not include fatigue testing. It might be felt by some that the internal splice systems could be subject to fatigue failure. The research indicates that internal splicing systems be limited to approximately 25 percent of the prestressing strands. Where more than 25 percent of the strands are internally spliced, additional external strengthening should be installed. Internal splice repairs should be monitored by inspection. Test results indicate that strand failures would result in localized concrete cracking because of the configuration of the splice.

Repairs in precompressed areas are of some concern. The findings of this research indicate that many agencies permit these repairs after the release of prestress. This means that the concrete in the repaired areas will not be in compression and may crack with the application of additional dead and live loads.

Unless the depth of defect is very superficial, this practice could lead to moisture penetration, corrosion of prestressing elements, and reduced structure life. One acceptable method is to apply preload, patch, and remove preload after patch has gained required strength. The preferable method is to repair prior to release of prestress.

Repair of Damage

Replacement and repair-in-place techniques have been developed and successfully used for the repair of damaged prestressed concrete bridge members. The full-scale and component test results of Appendix A demonstrate that several methods of repair can be successfully used. Most of the findings in the form of applicable technical data, tables, and charts are included in Appendix A. The discussion that immediately follows is a commentary on the tests performed and the methods of repair tested.

Load Deflection Curves

Load-deflection data were recorded for all load tests and the data are summarized in Figure 3. The load tests are numerically listed in the order they were performed. Unless otherwise noted, the load-deflection curves are for the last cycle of each test. For this discussion of the load-deflection curves, the tests are grouped in relation to the repair system applied to the girder. More and wider cracks developed under the applied moment

that were anticipated. Tests 1 and 3 were performed on the as-cast girder and slab; however, all tests following the first cycle of Test 1 were on a cracked section. Tests 2, 6, and 8 were performed with post-tensioned rods in place. Tests 4 and 7 were performed with four strands severed. Test 5 was performed with the four severed strands repaired with single-strand internal splices. Tests 9 and 10 were performed with six severed strands and the steel sleeve splice in place. The equivalent load, $P=46.47$ kips, plotted on Figure 3, gives a moment numerically equal to AASHTO HS-20 live loading.

In comparing the first cycle with the last cycle of Test 1, one can see the relationship of the uncracked beam to the cracked beam. The deflections of the cracked beam (last cycle) are significantly greater than that for the uncracked beam (first cycle) throughout most of the loading range. However, the final deflections under the maximum test load of 90.4 kips, for both cycles, are very nearly equal. Test 3 was the same as the last cycle of Test 1. The load-deflection curves are nearly identical.

Tests 2, 6, and 8 show the effectiveness of external post-tensioning. The load-deflection curves for Tests 2 and 6 are nearly identical, verifying the effectiveness of single-strand internal splices. All post-tensioned rod tests demonstrate the stiffening obtained by this method of post-tensioning. For all practical purposes the load deflection curves for Tests 3 and 5 are identical. This corroboration also verifies the effectiveness of the internal splice system. Figure 4 shows the load-deflection curves for the last cycle of Tests 1, 3, and 5, including the existing deflection from previous cycles. These curves show the

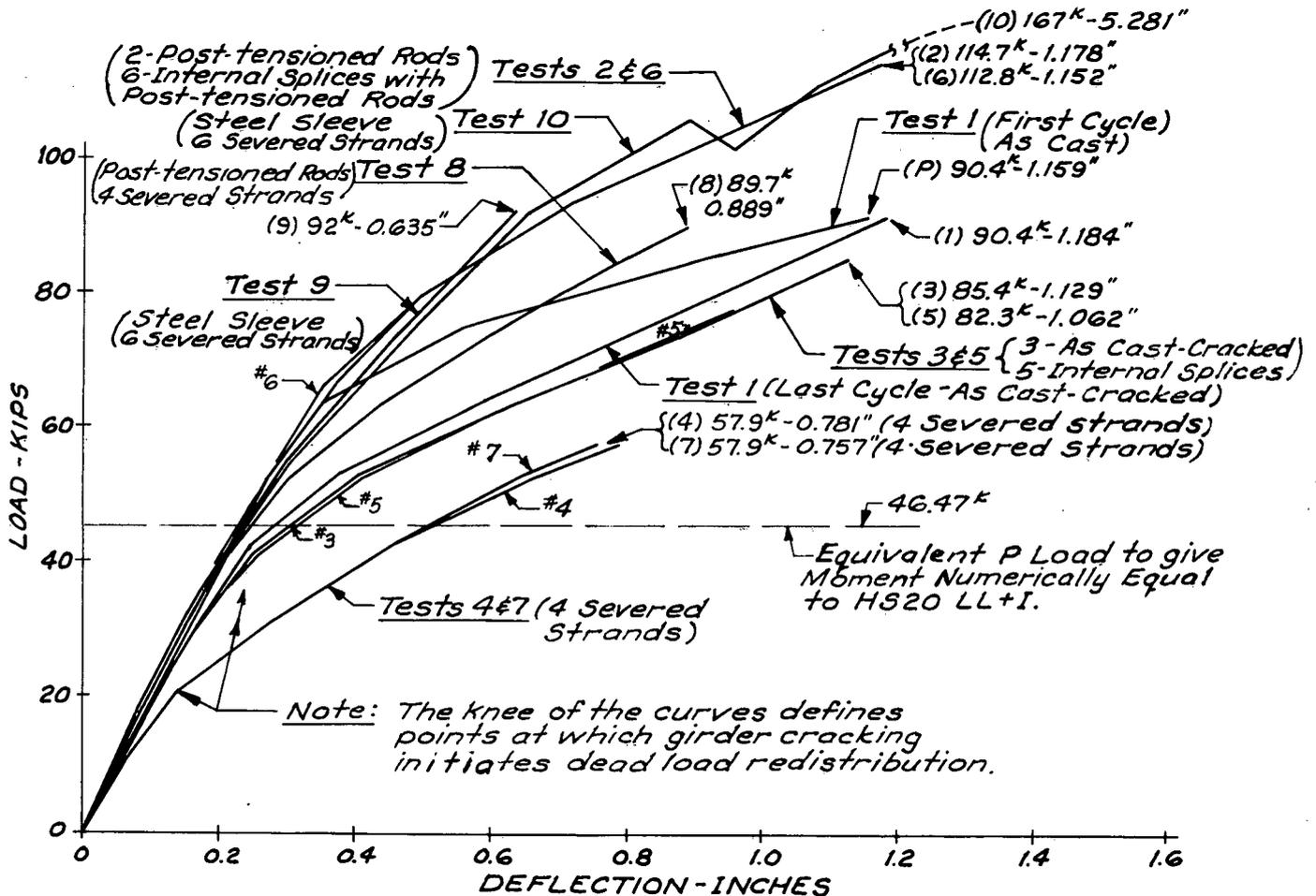


Figure 3. Load-deflection curves for all tests.

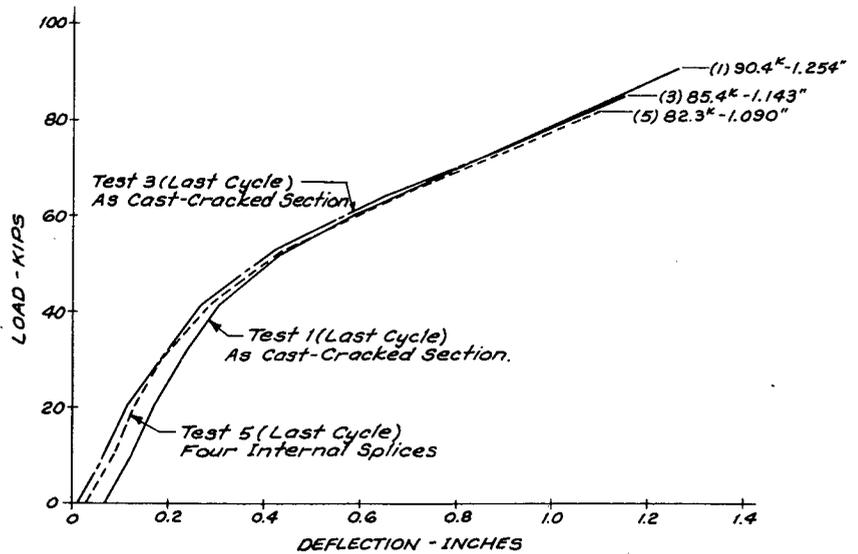


Figure 4. Load-deflection curves for Test Nos. 1, 3, and 5.

close correlation between the as-cast girder and the internal splice system.

Tests 4 and 7 were performed with four severed strands. The test results were substantially identical. This girder section should obviously have the least capacity. At the test load of 57.9 kips the difference in deflections was less than $1/32$ in.

Tests 9 and 10 show the effectiveness of the steel sleeve splice. This method of repair provided the greatest stiffness. Test 10 was the final and ultimate load test. Under the applied load of 167 kips, the deflection was approximately $5\frac{1}{4}$ in., and the unloaded girder had a permanent deflection of about $1\frac{1}{2}$ in. The load deflection curve for Test 10 indicates that the girder yielded at approximately 150 kips.

Load-deflection and load-strain curves were developed for all load tests. The individual curves for each test are included in Appendix A. It is interesting to note from the load-strain curves the approximate point where the girder section transforms to a partially cracked section. The knee of the curves for gages 110 and 111, measuring prestress strand strains, depicts this point. The calculations in Appendix A corroborate the conclusions drawn from the load-deflection and load-strain curves.

Table 1 shows the ratio of test load moment to the AASHTO specification HS-20 live-load moment. As can be seen, the ratio of the test load moment to the live-load moment varied from a low of 1.25 for Tests 4 and 7 to a high of 3.59 for Test 10.

Girder Cracking Due to Overloads

The girder testing phase of this project required the application of overload moments equal to 75 percent of the ultimate capacity of the girder. This overload is permitted by the AASHTO "Manual for Inspection of Bridges." The maximum crack widths due to this loading varied from 0.020 in. (0.5 mm) for Test 1 to 0.028 in. (0.7 mm) for Test 5. In general, there were eight major cracks that extended from the bottom of the girder to within a few inches of the bottom of the slab. Minor,

Table 1. Ratio of test load moment to HS-20 LL + I moment.

	Test No. 1	Test No. 2	Test No. 3	Test No. 4	Test No. 5	Test No. 6	Test No. 7	Test No. 8	Test No. 9	Test No. 10
P test ^K	90.4	114.7	85.7	57.9	83.7	113.4	57.9	89.7	92.0	166.8
Moment P test	1356	1721	1286	869	1256	1701	869	1346	1380	2502
Ratio	1.95	2.47	1.85	1.25	1.80	2.44	1.25	1.93	1.98	3.59

Notes: Maximum test load for each test is used.

Moments are in kip feet.

LL Distribution factor = $S/5.5 = 7.5/5.5 = 1.36$

$M [HS20 LL+I] = 697 \text{ k}'$; $M \text{ test} = \frac{P \text{ test} (60')}{4}$

Ratio = $M \text{ test}/697$

but observable, cracks occurred between the major cracks. The researchers were surprised at the magnitude and depth of these cracks. Because of the conservative load distribution factors presently specified by AASHTO specifications, however, it is highly unlikely that this magnitude of cracking occurs. If load distribution factors are eventually revised to more nearly equal their true values, cracking due to overloads may become a point of concern.

Girder Stress at Cracking Load

Test 1 was a test of the uncracked girder. The first observed cracks occurred at a test load of 55.1 kips. This load gave a calculated tensile stress in the concrete of 2.7 or $4.5 \sqrt{f'_c}$. At time of testing of f'_c was 6,030 psi. AASHTO specifications give an allowable tensile stress of $6 \sqrt{f'_c}$ and modulus of rupture as $7.5 \sqrt{f'_c}$. It should be noted that this was the result of a single test. It was not a primary objective of this test to determine cracking stress, and no attempt was made to determine the effect of differential shrinkage. Despite reservations about the accuracy of the cracking stress, it can be said that for this particular test, cracking occurred at a lower calculated stress than $6 \sqrt{f'_c}$.

Epoxy Injection of Cracks

Although 20 agencies reported the use of epoxy injection, only six of these agencies reported the use of preload. Even though a vertical crack appears to be closed in the precompressed zone, the entire surface area will not be in compression. This observation is confirmed by the fact that if the entire surface were in compression, it would not be possible to inject the crack. Thus the precompressed zone consists of compressed and non-compressed areas. When the cracks are injected, the non-compressed areas will be filled with epoxy resin, but they will not be in compression. Upon additional loading, the compressed areas will experience reduced compression; however, the non-compressed areas will immediately experience tension. If the crack occurs in the girder after bridge construction, the tensile stress will be equal to the full live load stress with no reduction due to prestress. If the crack is injected prior to pouring the roadway slab, the tensile stress will be equal to the dead load stress of the slab plus the full live load stress. Either of the foregoing repairs will probably lead to re-cracking of the repaired area. Because epoxy resin is normally much stronger in tension than concrete, the crack will probably occur adjacent to the filled crack.

To determine the required preload to prevent cracking, calculate the stress due to additional loads to be applied. Deduct allowable tension if any. Calculate load to give net stress. Apply this load (preload) to the girder. The load may consist of one or more loaded trucks or vertical jacking, as shown in Figures 25 and 26 in Chapter Two. Inject cracks with epoxy resin. When repaired cracks have gained required strength remove preload.

Box Girder Repair

The principles of repairing box girders are the same as for AASHTO prestressed girder shapes, but configuration of repairs may be different. Overheight vehicles generally damage the bottom exterior corner of box girders. Typical repairs of box girders are shown in Figure 5. With the exception of the internal strand splice, these splices will require access to the inside of the box. It is believed that most box girders do not have access to the inside cells. It may be necessary to provide a temporary access hole through either the top or bottom slab. Post-tensioned high-strength bars can be coupled together in place. Cored holes can be cut in diaphragms. If damage occurs in interior cells, they can be repaired in much the same manner as exterior cells. Repairs for AASHTO prestressed girders can normally be converted into box girder repairs.

Environmental Damage Repair

Environmental damage to girders is closely related to environmental damage to roadway slabs. Damage is generally caused by deicing chemicals, salt-water spray, or industrial pollutants. The assessment of damage should determine whether the damage to a girder is reversible. It is a known fact that roadway slabs may reach a saturation point that is irreversible. All agencies have ample literature, laboratory facilities, and skill to provide reasonable assessment of damage. When an assessment has been made that determines that a girder is repairable, the method

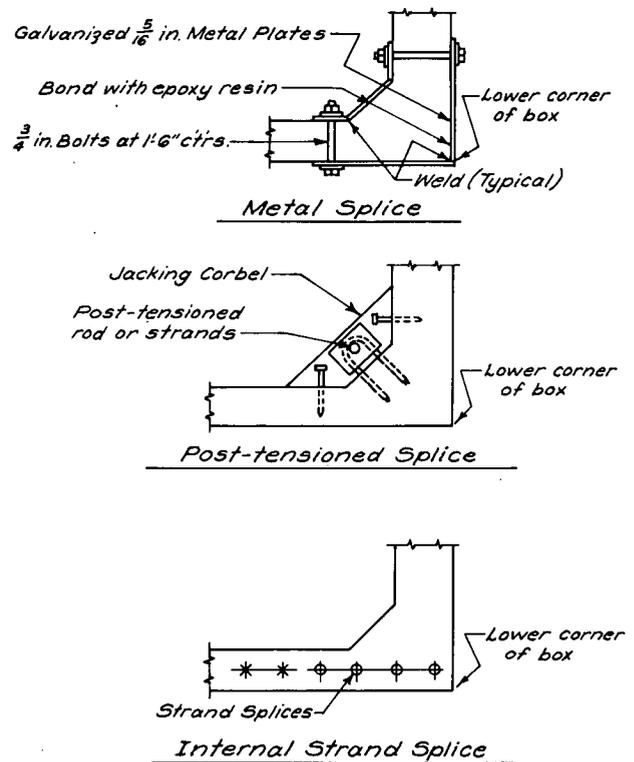


Figure 5. Box girder repairs.

of repair is very similar to repair of accidental damage. Removal of concrete, post-tensioning, sleeve splicing, internal strand splicing, and preloading are all appropriate methods of repair.

Increasing Capacity of Reinforced Concrete Girders

Girder Tests 2 and 6 demonstrated the effectiveness of increasing the capacity of prestressed girders by post-tensioning. These tests showed that an increase in live-load capacity of about 50 percent could be achieved with the addition of two 1 in., 150 K thread bars (ASTM-A722). Similar results could be obtained by post-tensioning reinforced concrete girders. One of two concepts could be used. Straight post-tensioned bars could be added at the top of the girder to increase negative moment capacity. In addition another set of post-tensioned bars could be added near the bottom of the girder to increase the positive moment capacity. Consideration could be given to adding draped post-tensioned tendons. The force components due to draping would be included in the design. This concept might not be acceptable aesthetically, depending on site parameters.

MANUAL OF RECOMMENDED PRACTICE

This user's manual establishes guidelines for evaluation and repair of accidentally damaged prestressed concrete bridge members. Included in the guidelines are inspection of damage, assessment of damage, selection of repair method, and repair of damage. Methods of repair include epoxy injection, concrete patching, preloading, internal strand splicing, adding external post-tensioning, metal sleeve splicing, and girder replacement. The guidelines for assessment of damage and selection of repair members are structured to lead to logical and appropriate methods of repair. However, this manual is not a handbook. The commentary in Chapter One explains in further depth why certain guidelines were adopted. The guidelines are practical in nature and based on the best information currently available, including the test results of Appendix A. The authors believe that sufficient research has been performed to document the effectiveness of the methods described in this chapter. No additional research is required prior to implementation of these methods in the field.

GUIDELINES FOR INSPECTION OF DAMAGE

Best inspection and assessment of damage will be accomplished by establishing standard procedures for inspection, reporting, engineering assessment, and the monitoring of completed repairs. The monitoring of repairs has been included in inspection of damage to afford personnel responsible for inspection the opportunity to improve their damage inspection and assessment techniques. From information gained through this research project, it is recommended that personnel responsible for inspection and damage assessment develop their specific procedures prior to the time when it is required to respond to a damage incident. This will better ensure uniform treatment and more orderly progress. Establishment of procedures, without pressure from an emergency, should result in best practices. Fortunately, few prestressed member damages are fracture critical. For purpose of definition in this manual *fracture critical* means that if the damaged member failed, the bridge (or span) would collapse. Accompanying this definition is the understanding that the damage inflicted poses a reasonable threat to the capacity of the member to carry the applied loads. Coordination and sharing of input should occur between all personnel that will participate in the inspection and assessment process. All departments of transportation that participated in this research indicated the interest and expertise required to organize this area of the problem in a manner that will suit their particular needs.

All sections of this manual should be thoroughly studied prior to starting work on any single phase.

Office Responsible for Inspection

Because the bridge engineer is normally responsible for the preparation of repair plans and repair design, this office should

be responsible for inspection. One major component for assessment of damage is the field inspection of damage. The bridge engineer's office should be best qualified for inspection of damage. Where organization constraints place the responsibility with others, cooperative action with bridge engineer personnel should be a requirement.

Damage inspection should be performed by competent personnel. Severe damage should be inspected by competent structural engineers.

Initial Inspection and Action

The primary objectives of initial inspection should be to ensure safety to the user and to reduce further damage to the bridge. When damage is severe, an experienced structural engineer should make the initial inspection and determine whether to restrict traffic or close the bridge. Temporary support systems, if required, should be placed immediately to prevent further damage. Preliminary strengthening may also be made to allow traffic on the bridge. These preliminary actions are normally based on judgment supplemented by brief calculations. Prestressed members will normally sustain large vertical deflections without collapsing. These deflections are readily visible. If a severely damaged member is fracture critical, immediate steps should be taken to prevent bridge collapse. When a member is damaged beyond repair, which is unusual, the engineer may recommend at this time to replace the member.

Few bridge closures are necessary for accidental damage to prestressed concrete members. However, when bridge closure is required, sufficient time must be allowed to ensure that adequate inspection and comprehensive evaluation are achieved. When safety of the user is in question, the bridge should be closed until it is conclusively determined that traffic can be safely restored. Political and public pressure should not be allowed to cause unwarranted decisions when safety is involved. Damage inspection and evaluation involving structural safety may normally require engineering office time. Temporary support systems may require plan preparation and procurement of materials. All temporary support systems must be designed to ensure safety to the user. The time required to restore traffic will depend on severity of damage and will vary with each accident. The engineering staff should recognize the need for expediency. Damage evaluation and plan preparation should have immediate priority. However, user safety must take precedent over restoration of traffic.

Inspection Sequence and Record

Damage inspection normally begins with the most critically damaged area first, followed by inspection of other damage in descending order of severity. Inspect the girders first. Inspect diaphragms or other secondary members last. When more than one girder has been damaged, a complete description of damage

for each member should be given. All areas inspected, including those areas inspected that did not suffer damage, should be recorded. This procedure aids the decision-making process of what, if anything, should be done to repair a member.

Inspection Equipment and Skills

Good access to the damaged area is necessary. A platform truck with adjustable height platform or a "cherry picker" is very useful. Check to ensure that the damaged girder or girders can support inspection loads before putting any loads on the structure. It may seem obvious, but a large amount of information can be gained from visual inspection. Personnel assigned to inspection should have good eyesight and a critical mind. The fact should be accepted that some engineers have better qualities of observation. For the best inspection, personnel should be assessed accordingly. Significant lateral or vertical displacements, rotations, and cracks can often be detected by eye alone. A good light is very useful for checking areas between girders. A simple carpenter's level is useful in checking whether a girder is plumb. More sophisticated levels are available that will give degrees out of plumb. With good lighting, a crack of only 0.004 in. (0.1 mm) wide can be detected. A magnifying glass should be included with inspection equipment. With 10X magnification, small cracks can be measured. Under normal conditions, cracks of 0.004 in. (0.1 mm) or greater in width can be measured. With excellent lighting, a crack width of one-half the above amount can be approximated. California uses an instrument called a Micro-Mike. This 20X microscope will measure crack widths to 0.002 in. (0.05 mm). A flashlight and mirror are good tools for checking inaccessible locations.

The inspection team should have good camera equipment. It is recommended that, in addition to a conventional camera, an instant photo camera be included. The instant photo camera will provide immediate pictures for evaluation purposes, while the conventional photographs will provide the more permanent record. Training should be provided to ensure that a sufficient number of properly descriptive pictures are taken of each damage.

String lining can be used to determine lateral offsets and curvature. Vertical displacements relative to adjacent girders can best be obtained by taking elevations along centerline of girders at top of roadway slab. String lining along the bottom of girders will give approximate relative vertical displacements. Many prestressed girders have different initial vertical deflections due to time dependent effects. Casting the roadway slab and diaphragms greatly reduces or eliminates subsequent displacement differences. Concrete cores can be taken to assess environmental damage. They can also be taken during fabrication in areas that are suspect due to faulty casting. It is obvious that strand locations must be considered. It is believed that impact damages do not affect the strength of concrete. These damages will crack, loosen, or shatter concrete. In some accidents all of the concrete in the bottom flange and web of a girder has been removed because of impact, with no apparent damage to strands. Small concrete spalls or gouges should also be measured because reduced coverage may affect strand durability.

The effect of damage on prestressing elements is of prime importance in determining whether to repair in place or replace a girder. Strands that are severed, nicked, gouged, deformed,

or sharply bent should be described and located. If necessary, concrete can be carefully removed to more closely inspect strand damage. A rivet gun chipper is an excellent tool to be used near strands to remove concrete without damaging strands. When all of the concrete in the bottom flange and a portion of the web have been shattered for a considerable length, the amount of prestress force in the strands may be in question.

Inspection must be closely correlated with damage assessment. For example, it would be unproductive to take strand samples if the girder will require replacement.

Inspection Report

It is useful to develop a standard form to be used in reporting damage. Items that are common to all damaged members, such as bridge name, location, and site conditions may be shown on a single form. A separate form sheet will generally be required for each member. If necessary, attachments to the form can be made to more fully describe the damage. The usefulness of a standard form is primarily to set forth in an organized manner the results of the inspection. An inspection form also serves as a reminder for the inspector of the various items to inspect. All reports should be clear and legible.

Most agencies have plans for existing bridges. These plans are valuable to have on-hand during the bridge inspection and may be used to supplement the standard forms. When plans are not available, more sketches may be needed. Members should be precisely identified in accordance with the plans and/or with clearly detailed sketches. Good photographs are very helpful in making damage assessments and subsequent repairs. All photographs should be clearly labeled, giving name of object, direction of view, and approximate distance to object. The inspection report should not normally contain recommended repair procedures. Factors that might alter solutions should be included.

A cross-referenced record system should be established to provide ready access to all damage incidents. This system should provide ready access to the inspection reports. Damage categories should contain type of member and cause of damage. Damage assessment calculations, repair calculations, and repair details should be included.

The contents of the inspection report should include:

1. Bridge name.
2. Bridge location description including location map.
3. Date of damage.
4. Date of inspection.
5. Law enforcement accident report.
6. Cause of damage.
7. Site conditions.
 - a. Damage over traffic.
 - b. Damage over water.
 - c. Other.
8. Information on user inconvenience.
9. Bridge plans if available.
10. Supplementary sketches.
11. Type of member.
12. Member identification.
13. Member category.
 - a. Fracture critical.
 - b. Primary.
 - c. Secondary.

14. Damage to prestressing elements.
 - a. Exposed.
 - b. Nicked.
 - c. Gouged.
 - d. Deformed.
 - e. Severed.
 - f. Damage locations.
 - g. Narrative description.
15. Damage to concrete.
 - a. Spalls, nicks, scrapes, and gouges (give sizes).
 - b. Cracks including width, length, and configuration.
 - c. Loose concrete.
 - d. Shattered concrete.
 - e. Damage locations.
 - f. Narrative description.
16. Member displacement.
 - a. Lateral.
 - b. Vertical.
 - c. Rotation or twist.
 - d. Narrative description.
17. Photographs.
18. Factors that may affect repair solutions.
19. Description of initial action.
 - a. Traffic restriction.
 - b. Member strengthening.
 - c. Other.

Monitoring of Repairs

Follow-up inspection of repairs should be on a regular basis. All of the recommended repairs in this manual are intended to restore members to their required strength and durability. However, experience has shown that restoration may not be complete for all damages. Environmental or fire damage may have an element of doubt as to complete restoration. Concrete patches and cracks that are repaired without the use of preload or post-tensioning, may crack and affect durability. Members that are completely restored should be inspected with the same frequency as the complete bridge. Member repairs containing reasonable doubt regarding strength or durability should be inspected at more frequent intervals. The frequency should be established by persons responsible for repair, and the bridge maintenance engineer should be advised.

GUIDELINES FOR ASSESSMENT OF DAMAGE

These guidelines have been developed from the evaluation of present techniques, full-scale and component testing, and an objective/subjective analysis gained from actual working experience. Guidelines have not been included for established procedures that are commonly used in this type of work. Except as specifically noted, guidelines for accomplishing work have not been repeated where information on these items is covered elsewhere in this manual. This manual does not endorse products or manufacturers. Trade or manufacturer's names, if included, are for guidance and possible contact by those who have not developed their own specifications for materials that might be used in repair techniques.

As noted earlier in this report, the most frequent cause of damage is due to overheight vehicle impacts. Other reported

causes were fire and during manufacture. Guidelines furnished in this manual are applicable to all of the foregoing types of damage.

The purpose of assessment of damage is to decide what should be done to an accidentally damaged member. One of three decisions should be made:

1. Do nothing.
2. Repair the member.
3. Replace the member.

Assessment of damage stops short of "Selection of Repair Method".

Assessment of Damage by Whom and Where

Preliminary assessment of damage should be made during inspection of damage as described under "Guidelines for Inspection of Damage/Initial Inspection and Action."

Final assessment of damage is preferably made by the same persons who inspected damage. Assessment of damage can also be made by other persons experienced in dealing with accidentally damaged prestressed members. At least one experienced structural engineer should participate in the final assessment of damage. Final assessment of damage should be made in an office environment away from the bridge site. The lack of adequate time is often a critical element in the assessment and repair of seriously damaged members. Personnel directly involved in assessment, selection of repair, and repair should certainly be made aware of the importance of time. Management, however, should not place undue distractions or pressure on the persons directly involved. The best repair decisions are made when pressure is limited, and generally minimize overall repair time.

Strength of Damaged Member

Brief calculations determining the approximate strength of the member may be made during the inspection phase. These calculations are normally made to determine whether to restrict traffic or close the bridge. These preliminary calculations may also be made to determine the necessity of temporary support systems in order to prevent further damage to the bridge.

During assessment of damage, a complete evaluation of strength should be made. This analysis should determine stress levels in the damaged member, and these stresses should be compared to the design stresses and to allowable stresses. Service load stresses and ultimate load capacity should always be computed. Overload stresses and fatigue stresses should be computed as appropriate. All preliminary calculations and decisions made during the inspection phase should be reviewed. It is important to assume all damaged strands as severed, and all loose or shattered concrete deleted when making these calculations.

The existing prestress force in exposed strands, particularly if the damage is in the vicinity of an end block or a harping point, is a critical factor in assessment of damage. A nondestructive test method to determine strand tension has been used by Michigan. Prestressed strands were exposed approximately 18 ft, after loose and shattered concrete was removed from a severely damaged fascia girder. The remaining force in each strand was calculated by measuring the deflection of each strand.

with a micrometer caliper. A small load of 20.5 lb was placed at the center point of each exposed strand, and deflection was measured from the strand above. Assuming a straight line deflection of the strand, the calculation for tension is: $T = P\ell / 4d$, where ℓ is the clear span of exposed tendon in feet; d is the measured deflection in feet; and T is tension in the strand.

A destructive test method can also be effective for determining strand tension. By carefully marking and cutting out a 10 ft length of strand, the stress can be calculated with reasonable accuracy. A 10 ft length of strand with a stress of 144 ksi will shorten approximately 0.62 in. This shortening is based on a modulus of elasticity of 28,000 ksi. An electric sander is a useful tool for cutting strands. Care must be taken to prevent injury due to whipping action when strands are cut.

California has tried using a Kuhlman bar with a 10-in. gage length to define stress characteristics of exposed tendons. Results with this device have not been consistent, and they do not place much value on these tests at this time.

User Inconvenience and Speed of Repairs

User inconvenience and speed of repairs are so interrelated they are considered as one in this manual. The following are general guidelines that should be considered to address these items:

1. Safety to users and strength of the structure must have primary importance. The decisions as to usability of the damaged structure, or portions thereof, should be conservative.
2. Depending on the actions required by the damage, the best traffic control system should be established. To reduce user inconvenience, the establishment of the best detour route should be carefully considered.
3. Recognizing that the initial decision probably was made only on inspection of damage, the engineering assessment should be made immediately. Should the engineering assessment determine that initial traffic restrictions were too severe, they should be revised to reduce user inconvenience.
4. Procedures for obtaining replacement girders should be established in advance of accident happenings. The need for any specialized equipment and materials should be established. The work that can effectively be accomplished by state forces should be known by all personnel that may be involved in assessment and repair.
5. The acceptability of various repair-in-place techniques should be established prior to an accident. Should any of the repair techniques outlined in this manual appear advantageous, it is recommended that they be studied at an early date. Single-strand internal splices could be fabricated in advance. Methods of applying preload, epoxy injection, attaching concrete patches, casting concrete patches, removing loose and spalled concrete, cleaning exposed strands, coating exposed strands, and sealing minor concrete nicks and gouges should be studied.
6. Replacement methods should also be studied prior to accidents to facilitate fast action.
7. Personnel responsible for repair designs should be trained prior to having to work under emergency pressures.
8. The required speed of repairs may dictate replacement or repair-in-place techniques, and each agency's ability to respond to a particular situation may dictate the final decision. Climatic conditions may influence decisions affecting user inconvenience and speed of repairs.

9. In general, repair-in-place techniques will result in less inconvenience to users. Because of less removal work, repair-in-place techniques should also result in faster repairs.

Fracture Critical Members

Fracture critical members should receive a more rigorous assessment of damage than nonfracture critical members. Selection of repair procedures should be more conservative than selecting repair procedures for nonfracture critical members. Repair-to-place methods may be used, but they should be used in a conservative manner. One method of attaining conservative results is to apply enough strengthening to achieve zero tension in precompressed tensile zones under all conditions of loading. Consideration should be given to inspecting these repairs at more frequent intervals than nonfracture critical members.

Primary Members

Nearly all prestressed members are primary members. The guidelines in this manual have been developed for primary members.

Secondary Members

Diaphragms in prestressed bridges are normally reinforced concrete. Damage should be repaired following common procedures used for the repair of reinforced concrete members. Prestressed diaphragms should be repaired in the same manner as primary members.

Minor Concrete Nicks, Spalls, and Scrapes

It is believed that minor nicks, spalls, and scrapes may deserve more attention than they usually get. This is particularly true in corrosive environments. Because of the effectiveness of concentrating strands near the bottom of prestressed girders, concrete cover is usually the minimum permitted by specifications. Reducing this cover and scraping away the concrete surface finish may permit the intrusion of corrosive elements to the strands. Strong consideration should be given to cleaning these surfaces and sealing with the two coats of a penetrating sealer. Other methods of repairing defects are described in "Guidelines for Repair of Damage."

Concrete Gouges

Two assessments are necessary, depending on how the girder was designed. If the girder was designed for zero tension bottom, a minor gouge can be assessed and repaired much the same as a minor nick, spall, or scrape. The gouge should be cleaned and sealed with two coats of a penetrating sealer. It may also be packed with epoxy, but it must be remembered that the epoxy has no precompression and may crack at the adjacent concrete, due to $LL + I$ stresses. It may be preferable to simply clean and seal minor gouges and leave them open for inspection. A gouge is considered minor, if in the opinion of the engineer it will not degrade durability if simply cleaned and sealed.

A gouge is assumed to have an approximate "V" or "U" shape. Girders designed to carry tension (as much as $6\sqrt{f'_c}$) pose an additional problem. A gouge across the bottom of a girder creates a stress raiser and the concrete may crack at a lower stress than $6\sqrt{f'_c}$. The crack could propagate upward to or above the bottom layer of strands. It is recommended that these girders be preloaded and then the gouge cleaned and inspected. If no crack propagation has occurred, the gouge can be packed with epoxy grout or filled in accordance with some other approved material. If a crack has propagated, it may be necessary to either remove concrete to the root of the crack and pack with epoxy grout or to inject epoxy resin in the crack and epoxy grout the remainder. The preload is removed after the patch has gained the required strength. The "Guidelines for Repair of Damage" contain a number of preload examples.

Concrete—Cracks

Cracks in prestressed members are repairable in place. Cracks are categorized as to direction (longitudinal, vertical or diagonal) and as to location, within the precompressed live load tensile zone or outside (usually above) the precompressed live load tensile zone. Even though vertical and diagonal cracks in the precompressed zone appear to be closed, it should not be assumed that the entire crack surface is in compression. If the entire crack surface was in compression, it would not be possible to inject the cracks. This means that epoxy resin injected into these areas will not be in compression when $LL + I$ loads are applied and will generally reappear when live loads are applied. The tensile stress due to live loads should be computed (do not deduct prestress); if the stress is greater than allowable, and it usually will be greater, then preload should be applied prior to injection. Longitudinal cracks or cracks outside the precompressed live load tensile zone do not require preload prior to

epoxy injection. Epoxy resin is capable of injection into and travel in a crack of only 0.002 in. (0.05 mm) wide. Cracks of lesser width are acceptable without injection. Epoxy injection is described in detail under "Guidelines for Repair of Damage."

Concrete—Loose or Shattered

There is no specific limit on the amount of loose and shattered concrete that can be removed and repaired in place. Loose and shattered concrete is usually accompanied with apparent strand damage, but this is not always the case. There have been incidents of the entire bottom flange and web being shattered with no apparent strand damage (see Fig. 6). The only structural limitation is restoring original strength and durability. The difficulty of replacing girders that have been made continuous for live load is mentioned elsewhere in this report. The cost and user inconvenience may lead to more use of repair-in-place methods. Extensive damage to either simple span or continuous for live load girders can be repaired in place. Preloading is one method of restoring compression in concrete. External post-tensioning is another. Metal sleeve splices can be used without restoring compression, provided the concrete stress at the top of the sleeve and at the sleeve ends is within the allowable tension for all loading conditions.

Strands or Other Prestressing Elements

A very high majority of the prestressed bridges in this country are prestressed with seven-wire strands, either 240 K or 270 K. This manual is, in the main, devoted to those bridges. Early bridges may have had other types of prestressing elements. High-strength bars or rods have been used. Single wires have had

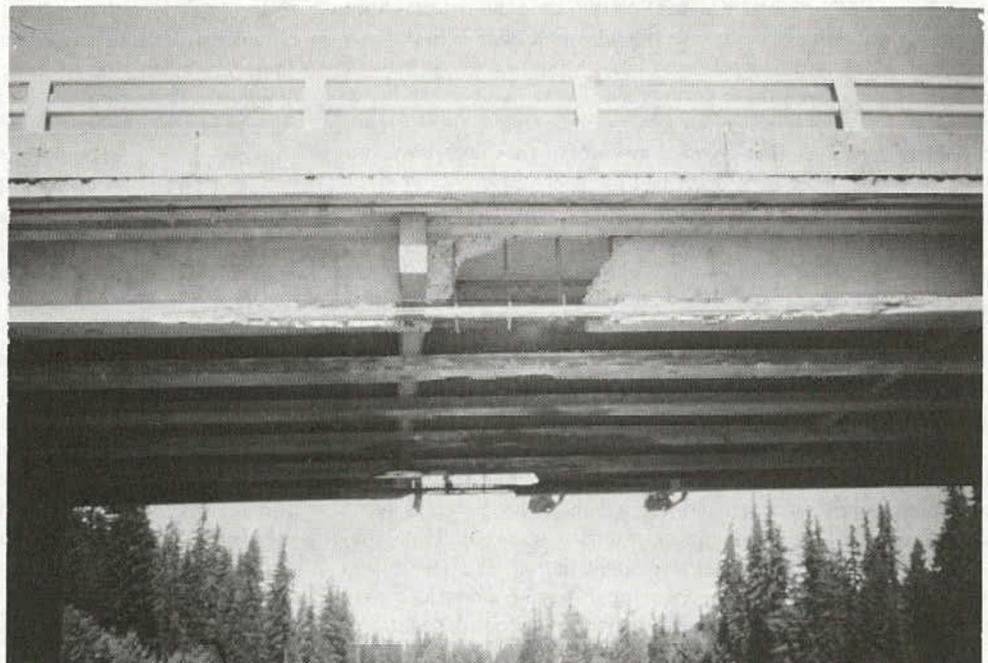


Figure 6. Major concrete damage with no apparent strand damage. (Courtesy of State of Washington DOT)

limited use. Post-tensioned tendons consisting of a group of wires or strands within a conduit have been used to prestress precast girders. They are presently used to prestress cast-in-place bridges. Combinations of post-tensioning and pretensioning have been used. Segmental bridges are a special category. Harped strands are generally bundled throughout the middle portion of girders. It is not possible, within the scope of this project, to provide specific guidelines for assessing damage to all of these diverse elements. However, the principles of assessment of strand damage and what to do about it will have applicability to other prestressing systems.

Exposed

Exposed strands should be carefully inspected for damage. When these strands are undamaged, they need to be protected for durability. Methods for protecting strands are given under "Guidelines for Repair of Damage." It should be remembered that concrete patches used to cover strands may crack because of live load unless preloading is used.

When a number of strands are exposed over long lengths (more than a few feet), they will usually be accompanied with loose and shattered concrete. After careful removal of the loose and shattered concrete, a close inspection may reveal that many or all of these strands have not suffered any apparent damage. A number of engineers may have reasonable doubt as to whether these strands have been significantly degraded. A question may also arise as to whether these strands have lost significant prestressing tension. As described under "Guidelines for Inspection of Damage," a length of one strand can be removed and tested. Also described is a method of measuring prestress.

Damaged

A nick in one wire of a seven-wire strand is not considered serious. Even if the wire eventually cracks because of fatigue, the failure will not cause propagation into the other wires. Other more serious damage of strands often occurs, such as flattening, severed wires, and sharp bends. For purpose of assessment of damage and repair, these strands should be assumed as being severed. Closed vertical cracks in precompressed areas are an excellent indicator that covered strands in that area have not yielded. The girder tests described in Appendix A required 22 loading cycles to a maximum moment of approximately 75 per cent ultimate moment. The maximum crack width varied from 0.016 in. (0.4 mm) to 0.024 in. (0.6 mm). When test loads were released, all cracks closed to hairline width (approximately 0.002 in. (0.05 mm)).

Severed

It is believed that nearly all state bridge engineers have some minimum number of severed strands that they judge as being permissible, without splicing or restoring strength. This judgment should be based on a stress calculation. It appears that fracture critical or nonfracture critical may be a consideration. More severed strands in a girder carrying sidewalk loading is permissible by some states. All or nearly all states will allow

one severed strand. Few, if any, will allow more than three severed strands.

There is no need for setting an arbitrary upper limit on the number of severed strands that can be spliced in place. The limits will be controlled by the splicing technique used. Each splicing technique has an upper limit to its ability to restore original strength and durability.

Web Reinforcement

Mild steel web reinforcement can normally be repaired in place. Straightening bent reinforcing bars is one method. Cutting and lap splicing with another bar is often possible. Welding bars together has been used successfully. When welding, it is imperative that the prestressing strands be fully protected from excessive heat or weld spatter. It may be done with asbestos sheets. Web reinforcement is critical for ultimate loading; therefore, fatigue is not a significant factor. Web reinforcement is generally not larger than No. 4 or No. 5 bars, which expedites repair in place.

Member Displacements

Sweeping lateral curvature is seldom caused by vehicle impacts. This type of curvature is invariably caused during manufacture, storage, or construction, and becomes locked into the bridge by placing diaphragms and roadway slab. When abrupt lateral offsets occur because of impact loading, they are cause for considerable concern. Enough concrete should be removed to determine the number of strands that also have an abrupt offset. If the strand offsets are abrupt and appear to be permanent, the conservative approach is to assume that those strands have excessively yielded and are no longer effective. The metal sleeve splice is capable of splicing a large number of strands and may be considered when the offset is small. The width of sleeve would be increased to fit the girder. Girder replacement would be a logical consideration.

Vertical deflection is rarely affected significantly by impact damages. When a large percentage of strands has been severed, downward deflection may occur. When a long length and large volume of concrete have been shattered with few, if any, severed strands, upward deflection may occur. Longitudinal profiles can be taken on the roadway surface over the damaged girder and adjacent girder or girders. After correcting for roadway slope and other geometric factors, the deflection curves may be of limited value. Downward deflection may simply confirm the fact that a large number of severed strands need to be spliced. Upward deflection may tend to confirm the fact that although a large volume of concrete has been lost, the strands are still in tension. As a general rule, vertical deflections are too small to be of significant value.

Permanent twist or rotation can be caused by vehicle impacts. Twist may also occur because of faulty manufacturing, storage, or construction. Twist due to impact loading will generally crack the girder longitudinally at the bottom of the top flange. It may also crack the girder at diaphragms. Permanent twist due to impact loading will be concentrated in the area of impact. Gentle increases in twist are generally caused by other factors. Permanent twist may be accompanied with abrupt lateral offsets

at the bottom of the girder. Rotations are not justifiable cause for girder replacement provided the rotations are uniform in nature. Cracks caused by rotation are repairable.

GUIDELINES FOR SELECTION OF REPAIR METHOD

The purpose of these guidelines is to arrive at the best repair solutions for any individual damage incident. A combination of repair methods may result in the best repair solution. The guidelines for assessment and repair of concrete nicks, spalls, scrapes, gouges, and cracks are contained in "Guidelines for Assessment of Damage" and "Guidelines for Repair of Damage" and, therefore, do not need coverage in this section of the manual. Damage assessment and repair of damage are interrelated. Anyone inspecting or assessing damage should thoroughly study the "Guidelines for Repair of Damage."

External Post-Tensioning

External post-tensioning can be used to restore strength and durability to damaged girders. It can also be used to add strength to existing bridges that are deficient in load capacity. This method should be suitable for splicing bundled strands or small tendons as well as a number of individual strands. The post-tensioning elements are normally high-strength rods, 150 K thread bar (ASTM-A722), or seven-wire strands. Concrete corbels cast against the girder are generally used as jacking points (detailed in Figs. 9, 10, and 11 for splice 1 and Fig. A-13 in Appendix A). The Washington State DOT uses a corbel similar to Figures 13 and 14. They may extend the nonprestressed cap to the girder end blocks for aesthetic reasons. A calculation should be made to determine if preload is required. If preload is required, it should remain in place during patching of concrete, injection of cracks, and until required strength of concrete has been attained. Preload should be removed prior to post-tensioning.

The strength of the jacking corbels will generally control the number of severed strands that can be spliced by post-tensioning. The test results shown in Appendix A prove that corbels with enough strength to splice four ½-in. diameter 270 K strands in an AASHTO Type III girder can be readily constructed. Corbel lengths can be increased; however, most of the load transfer occurs near the face of the corbel. Corbel size and amount of reinforcing could be increased, particularly for larger girders. If corbels similar to Figures 13 and 14 are used, increase in stress due to live load will be transferred by bond, and stress at the corbel face should not increase beyond the initial jacking load stress. It appears that splicing more than six to eight strands might be difficult.

Internal Splicing

The internal strand splices shown in this manual (Figs. 19, 20, 22, 23, and 24) and in the test results of Appendix A indicate that this type of splice is inexpensive and easy to install. This is particularly true of the single-strand splice used in Test 5. The splices may be torqued to the approximate tension in the adjacent nonsevered strands. Preload must be used in the spliced

area to restore compression in the concrete patch. The normal procedure would be to tension the strand splices, apply preload, inject epoxy resin into cracks, patch concrete, and remove preload after new concrete has gained required strength. Load Test 5 in Appendix A shows the methodology used. Load Test 5 proved the validity of restoring compression through the use of preload. The cracking load was not substantially less than the cracking load for the original uncracked girder (49.7 kips compared to 55.1 kips). The difference could easily be explained by not having injected the cracks with epoxy resin.

The amount of prestress that can be restored by preloading appears to be relatively large. Twenty-five percent of the strands were severed for Test 5 (4 out of 16). The preload required was only 26.4 kips. The basis for amount and allowable preload should be a stress calculation. Generally, enough compression can be restored to equal LL + I tension. During the tests described in Appendix A, the girder was test loaded to approximately twice LL + I. Cracks occurred that extended to within a few inches of the bottom of the slab. These cracks did close upon release of load. Preloading to the degree to preclude cracking under this amount of overload should not be necessary and is not desirable. Fatigue testing was not within the scope of this project. Internal strand splicing as presented in this manual uses commercial strand couplers. Studies made by Dr. Karl Frank of the University of Texas indicate that these couplers do reduce fatigue life. He has also made a study of methods to reduce the fatigue susceptibility of these couplers. The improved devices are larger in diameter and may require staggered splices. Ordinary splices should be used conservatively. Limiting splicing to 25 percent of the total number of strands may be a conservative figure. If all splices failed, the increase in stress would be 33 percent.

Metal Sleeve Splice

The metal sleeve splice detailed in this manual can be used to splice a large number of severed strands. It is also a good splice to use for repairing a girder with large volumes of loose or shattered concrete (shown with splice 3, Figs. 15, 16, and 17). This method may also be useful for splicing environmentally damaged concrete. It will restore strength and durability. The plates are made of galvanized A-36 steel, which can be painted "concrete gray" after installation, for aesthetic reasons. Bond between sleeve and concrete is furnished by injected epoxy resin. The sleeve should extend 3 ft 3 in. minimum beyond the damaged concrete area. For splicing six strands or less, the sleeve should lap the severed strands 5 ft 3 in. minimum (see Fig. A-50). For splicing more than six strands, the sleeve should lap the severed strands 160-strand diameters, minimum, as detailed in Figure 15. Preloading is not necessary, provided the stress at the top and end of the sleeve is within the allowable for all loading conditions. The sleeve could be extended upward for splicing web concrete. Bond stress could govern the allowable number of strands spliced.

Combining Splice Methods

Combining splice methods may be advantageous. External post-tensioning could be used to splice a tendon or a group of bundled strands in a girder. Internal strand splices could be

used to splice individual strands in the same girder. The metal sleeve splice could be adequate for nearly all damages. However, it could be combined with other methods.

Complete Replacement

Complete replacement of a member is normally the most expensive method of repair. Replacing a girder will require removing a portion of the roadway slab. Replacement of a girder made continuous for live load plus impact will be even more difficult and expensive. The Minnesota DOT uses a method of replacing simple span girders that requires a minimum of slab and diaphragm removal. See "Guidelines for Repair of Damage" for girder replacement methods.

Service Load Capacity

Calculations should be made to show that repair of damaged girders has essentially the same service load capacity as the original girder. Minor differences could be accepted if justified (see section under "Overload Capacity").

Ultimate Load Capacity

Calculations should be made to show that repair of damaged girders has essentially the same ultimate load capacity as the original girder. Minor differences could be accepted if justified (see section under "Overload Capacity").

Overload Capacity

The scope of this report is essentially limited to the repair of accidentally damaged prestressed concrete bridge members. The overload capability of the repaired members should not reduce

the allowable overload capacity of the bridge. One illustration of overload capacity for a repaired-in-place girder is shown in Figure 7 and accompanying calculations.

It is not within the scope of this report to study the adequacy of present bridge design procedures to carry the extra-legal loads that are operating under permits. A discussion of this subject is given in Cassano and LeBeau (2).

Some agencies consider the location of a damaged member in assessing overload capability. If a member is located behind a barrier curb or under a sidewalk, it may not receive as much overload as an interior girder. This is a logical consideration. For those agencies with Structural Design Language (STRUDL) computer programs available, it is fairly simple to determine the load carried by an individual girder. No justification has been found to add additional overload capacity to the repaired girder above that of adjacent girders.

Two methods shown in this report could be used to strengthen existing spans or bridges. One is the metal sleeve splice, which could be fabricated in segments and joined together by butt welding. The other is the adding of additional prestress, similar to Figures 9, 10, and 11 shown with splice 1, and tested in Load Test 2.

Overload Capacity Illustration

Permit loads are loads that exceed legal limits but are allowed to travel on the highway under a permit issued by an authorized agency. There is variation in load permit policies from state-to-state, and the details of these policies are not readily available. The State Transportation Department of California has published its permit policy (2) and the overload illustration (Fig. 7) uses this policy. California designs each new structure for both HS-20 loadings and a loading system that has axle weights approximately 1.5 times HS-20 loadings. The overload moments used in the illustration are based on California's loads and spacing of loads. The load distribution factors are assumed to

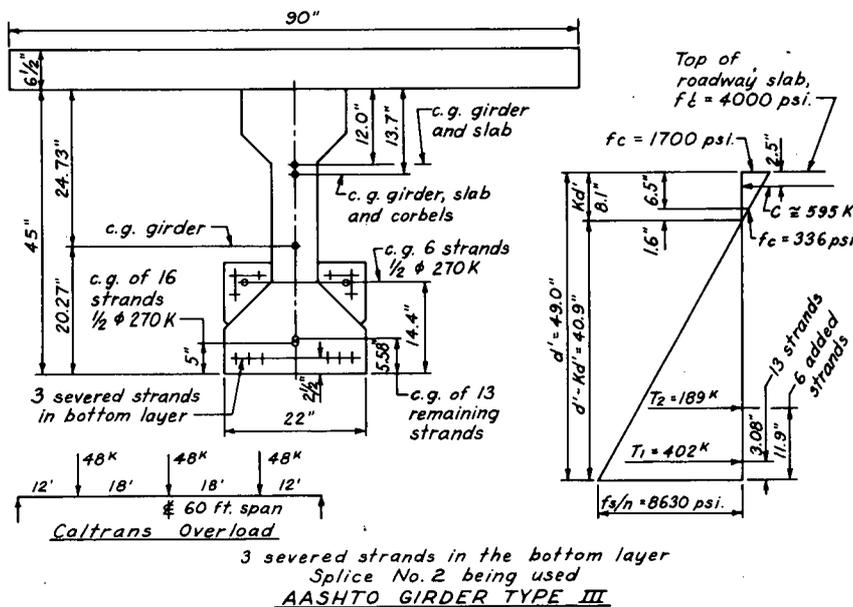


Figure 7. Overload capacity illustration.

be the same as AASHTO specifications. These factors may be conservative. The following calculations are intended to provide guidance in determining the effect of overload:

1. Given:

- f'_c girder = 5,000 psi. E_c slab assumed to be $0.8 \times E_c$ girder.
- For section properties and normal strand working loads, see splice 2 under "Post-Tensioning."
- Girder spacing (S) = 7 ft 6 in.
- Distribution = $S/5.5 = 1.36$.
- Impact = 27 percent.
- $M_{LL+I} = 1,120$ ft-kip (Overload).

2. Calculate overload stress for the original girder:

- DL girder = 683 lb/ft.
- DL slab = 650 lb/ft.
- Total DL = 1,233 lb/ft (Not including corbel wt).
- $M_{DL} = wL^2/8 = 555$ ft-kip.
- $f_{t,bot}$ (DL) = $M/S = 555(12)/6,190 = 1,076$ psi (t).
- $M_{LL+I} = 697$ ft-kip (HS loading).
- $f_{t,bot}$ (LL + I) = $697(12)/9,895 = 845$ psi (t).
- $f_{c,bot}$ (Prestress) = $P/A + M/S = 16(22 \text{ kips/strand})/559 + 16(22)(15.27)/6,190 = 1,498$ psi (c).
- $f_{t,bot}$ (HS-20) = $1,076 + 845 - 1,498 = 423$ psi (t).
- Assume allowable $f_t = 6\sqrt{f'_c} = 6\sqrt{5,000} = 424$ psi (t).
- $f_{t,bot}$ (Caltrans Overload) = $1,120(12)/9,895 = 1,358$ psi (t).
- $f_{t,bot}$ (Overload) = $1,076 + 1,358 - 1,498 = 936$ psi (t).
- Allowable cracking stress per AASHTO Article 1.6.6 = $7.5\sqrt{f'_c} = 530$ psi.
- Therefore, the stress due to Caltrans overload exceeds the allowable cracking stress by $(936 - 530)(100)/530 = 77$ percent for the girder as originally designed.

3. Calculate overload stress for the repaired girder:

a. Assume loss of concrete section to be negligible:

- $f_{c,bot}$ (Prestress) = $P/A + M/S = 13(22 \text{ kips per strand})/559 + 13(22)(14.7)/6,190 = 1,190$ psi (c).
- $f_{c,bot}$ (Added Post-Tensioning) = $P/A + M/S = 6(24.7 \text{ kips per strand})/1,129 + 6(24.7)(16.9)/11,550 = 350$ psi (c).
- f_t (DL) = 1,076 psi (t).
- f_t (Weight of Corbels) = $41.2 \text{ ft-kip}(12)/9,895 = 50$ psi (t).
- $f_{t,bot}$ (Caltrans Overload) = $1,120(12)/11,550 = 1,160$ psi (t).
- $f_{t,bot}$ (Repaired Girder) = $-1,190 - 350 + 1,076 + 50 + 1,160 = 746$ psi (t).
- Therefore, the stress due to Caltrans overload exceeds the allowable cracking stress by $(746 - 530)(100)/530 = 41$ percent for the repaired girder.

b. Assume 5-in. removal of shattered and loose concrete from the bottom flange (Preload No. 4). Assuming preloading is not used, the only prestress at the bottom of the girder in the damaged area will be that furnished by the six post-tensioned strands:

- $f_{c,bot}$ (Added Post-Tensioning) = 350 psi (c).
- $f_{t,bot}$ (Caltrans Overload) = 1,160 psi (t).
- $f_{t,bot}$ (Overload) = $-350 + 1,160 = 810$ psi (t).
- Therefore, the stress due to Caltrans overload exceeds the allowable cracking stress by $(810 - 530)(100)/$

530 = 53 percent for the repaired girder, at the point of damage.

4. Calculate ultimate strength capacity of original girder:

- f'_c roadway slab = 4,000 psi.
- $p^* = A_s^*/bd = 16(0.153)/90(46.5) = 0.000585$.
- $f_{su}^* f_t [1 - (0.5)(p^*)(f'_{su}/f'_c)] = 270 [1.0 - (0.5)(0.000585)(270)/4.0] = 265$ ksi.
- $M_u = A_s^* f_{su}^* d [1 - 0.6(p^*)(f_{su}^*/f'_c)]$.
- ϕ Assumed to be 1.0.
- $M_u = 2.45(265)(46.5) [1.0 - 0.6(0.000585)(265)/4.0]/12 = 2,460$ ft-kip.

5. Calculate ultimate strength capacity of repaired girder:

- $p^* = A_s^*/bd = 19(0.153)/90(43.2) = 0.000747$.
- $f_{su}^* = 270 [1.0 - 0.5(0.000747)(270)/(4.0)] = 263$ ksi.
- $M_u = 2.91(263)(43.2) [1 - 0.6(0.000747)(263)/4.0]/12 = 2,670$ ft-kip.
- Therefore, the ultimate strength of the repaired girder is greater than the ultimate strength of the original girder.

6. Calculate ultimate strength capacity required by 1977 AASHTO Art. 1.6.5 (Load Factors):

- $M_u = 1.3 [DL + 5/3 (LL + I)]$.
- $M_u = 1.3 [555 + 41.2 + (5/3)(697)] = 2,285$ ft-kip.
- Therefore, the ultimate strength of the original girder (2,460 ft-kip) and the ultimate strength of the repaired section (2,670 ft-kip) both exceed the ultimate strength required by AASHTO Art. 1.6.5.
- However, the AASHTO *Manual for Maintenance Inspection of Bridges 1978* also allows rating of bridges with a load factor method. The maximum allowable operating moment = $1.3(M_{DL} + M_{LL+I}) = 1.3(555 + 41.2 + 697) = 1,680$ ft-kip (assume $\phi = 1.0$ for prestressed girders).
- The maximum moment using Caltrans overloads = $555 + 41.1 + 1,120 = 1,716$ ft-kip. Therefore, the Caltrans overload moment exceeds the allowable operating rating moment by $(1,716 - 1,680)(100)/1,680 = 2$ percent.
- The *Manual for Maintenance Inspection of Bridges* also gives maximum allowable stresses for operating ratings. Article 5.4.6a (Prestressed Concrete) states that the Operating Rating shall result in moments not to exceed 75 percent of the ultimate moment capacity of the member. In situations with wide vertical dispersion of the tendons, the stresses shall not exceed 0.90 of the yield point stress in the prestressing steel in the layer of tendons nearest the extreme tension fiber of the member.
- 75 percent of the ultimate strength of the original girder = $0.75(2,460) = 1,845$ ft-kip. The maximum moment using Caltrans overload = 1,716 ft-kip which would be acceptable.
- The repaired girder does have vertical dispersion of strands, and the approximate capacity is calculated as follows: the allowable steel stress in the bottom layer = 0.9 yield point = $0.9(0.85)(270) = 207$ ksi.

7. Determine if section is cracked:

- The normal working stress in the strands after losses $\approx [0.7(270,000) - 45,000] = 144$ ksi. The change in strand stress ≈ 207 ksi - 144 ksi = 63 ksi; n for $f'_c = 5,000$ psi is 6.1.

- Therefore, the change in concrete stress = $63,000/n = 63,000/6.1 = 10,300$ psi. The concrete cannot resist this change in stress and will crack.
 - The allowable steel stress is below the yield point. Therefore, the strain due to the increase in tensile stress in the steel and the resulting compressive stress in the concrete will remain a plane. The working stresses in the strands will be added to the increases in tensile stress in the strands. Assume the bottom layer of strands to be $2\frac{1}{2}$ in. above the bottom of the girder. $d' = 51\frac{1}{2} - 2\frac{1}{2} = 49.0$ in.
 - For f'_c slab = 4,000 psi, $E_c = 33 (w^{3/2}) \sqrt{f'_c}$
 $w = 150$ lb/ft³
 $E_c = 3,834,000$ psi
 $E_s = 28,000,000$ psi
 $n = E_s/E_c = 7.3$
 $f_s/n = 63,000/7.3 = 8,630$ psi
 - $f_c/kd' = 8,630/(d' - kd')$
 - $f_c = 1,700$ psi (solution by trial and error).
 - $1,700/kd' = 8,630/(d' - kd')$
 - Therefore, $kd' = 8.1$ in.
 - f_c at bottom of roadway slab = $(1.6/8.1)(1,700) = 336$ psi.
 - $C = [(1.7 + 0.336)/2] [(90)(6.5)] \cong 595$ kips.
8. Calculate stresses in the 13 remaining strands (stress in top flange of girder neglected):
- f_s per strand = 144 ksi + $63(37.8)/40.9 = 202$ ksi.
 - Therefore, for 13 strands, $T_1 = 202(13)(0.153) = 402$ kips.
 - Working stress of six new post-tensioned strands = $0.6 f'_s = 0.6(270) = 162$ ksi.
 - f_s per strand = $162 + 63(29.0)/40.9 = 206$ ksi.
 - Therefore, for 6 strands, $T_2 = 206(6)(0.153) = 189$ kips.
 - $T_1 + T_2 = 402 + 189 = 591$ kips $\cong 595$ kips.
 - $M = 402(43.4)/12 + 189(34.6)/12 = 2,000$ ft-kip.
 - The maximum moment using Caltrans overload = 1,716 ft-kip, which would be acceptable.
 - For an approximate change in steel stress of 63,000 psi, $\Delta/l = f/E_s = 63,000/28,000,000 = 0.00225$ in./in. Assuming cracks spaced at 6 in., length of crack $\cong 6(0.00225) \cong 0.0135$ in.

Summary

With the exceptions discussed earlier in this section, the repair of damaged girders will have the same overload capacity as the original girder. This criterion does not appear to pose a problem. It appears that, in general, if repaired girders have service load capacity and ultimate load capacity equal to or greater than the original girder, the overload capacity will also be equal to or greater than the original girder.

The preceding illustration shows that the Caltrans overload gave very high tensile stresses in the girder. These stresses were high enough to cause cracking. This illustration does cast doubt on the ability of prestressed bridges designed in accordance with 1977 AASHTO Specifications to handle permit overloads without cracking. It is suggested that all design agencies review Caltrans' article (2). It is acknowledged that full-scale load tests of prestressed bridges normally show higher load-carrying capacities than calculations based on specifications.

Fatigue

In AASHTO Standard Specifications—1977, Article 1.6.17 states that the anchorages of unbonded tendons and the anchored tendon shall withstand a dynamic test as follows.

A representative specimen and tendon shall withstand, without failure, 500,000 cycles from 60 percent to 66 percent of its minimum specified ultimate strength, and also 50 cycles from 40 percent to 80 percent of its minimum specified ultimate strength. The period of each cycle involves the change from the lower stress level to the upper stress level and back to the lower. The specimen used for the second dynamic test need not be the same used for the first dynamic test.

Dynamic tests are not required on bonded tendons unless the anchorage is located or used in such manner that repeated load applications can be expected on the anchorage.

No other fatigue life criteria for prestressed concrete design are presented in AASHTO Standard Specifications—1977. It is believed that there is ample literature readily available that indicates that prestressed concrete beams do not fail because of concrete fatigue. Allowable current working stresses give adequate protection against concrete fatigue.

Existing tests show that prestressing strands in cracked prestressed concrete girders exhibit a lower fatigue life than fatigue tests on bare strands. When the fatigue loading on a prestressed girder is of sufficient magnitude to cause a flexural crack, stress concentrations are imposed on the prestressing elements. These stress concentrations give higher stress ranges for ensuing loads of the same magnitude and, therefore, a lower fatigue life. It is possible that girders subjected to accidental damage and cracking may exhibit an even lower fatigue life. Therefore, the fatigue life of damaged prestressed concrete bridge girders repaired in place deserves a review of previous fatigue tests.

Fatigue Life of Bare Strands

Fatigue tests of prestressing strands show that if strands are subjected to a maximum stress range of 0.10 times the ultimate strength, the strand has a fatigue life of approximately 2,000,000 cycles (3). For 250 K strands, this stress range would be 25 ksi, and 27 ksi for 270 K strands.

It is believed that nearly all grade separation structures would fall in this category. The following example (from Washington DOT computer printout) shows that the live load plus impact stress for a 57.33-ft span is 1.199 ksi. The 28-day concrete strength was 6.0 ksi. Using $E_c = 33 w^{3/2} \sqrt{f'_c}$, with $w = 150$ lb per cu ft, $E_c = 4,700,000$ psi. Using $E_s = 28,000,000$ psi, $n = 28,000,000/4,700,000 = 6.0$. The stress range is only $6.0(1.199) = 7.2$ ksi. It is apparent that the fatigue life of bare prestressing strand is entirely adequate. (Note in the example that a breakdown of stresses at the centerline of the span is given at the end.)

SBDG—Stress at bottom of the girder due to dead load of the girder only, on the girder section (K/in.²)

SBDS—Stress at bottom of the girder due to dead load of the slab only, acting on the girder section (K/in.²)

SBDC—Stress at bottom of the girder due to dead load of the curb and any weight of asphalt overlay acting on the composite section (Using Long Term SLAB E) (K/in.²)

SBXML—Stress at bottom of the girder due to live load plus impact only, acting on the composite section (K/in.²)

SBPRE—Stress at bottom of the girder due to prestress only, acting on the girder section ($K/in.^2$), after all losses.

INDENT NUMBER 1 DESIGN PROBLEM

PROBLEM 2, 6 HARPED STRANDS, 12 STRAIGHT STRANDS

HARPED STRANDS RAISED

SPAN 57.33, SERIES 6, 6. GIRDER AT 6.92, 7.00 SLAB

WEB INCR 0.0, 0.0 IN ASPH. OVERLAY

STRANDS 0.153, 270 K, A = 2.75

0.9 FT CURB S/5.5, AVFY = 16.00

FINAL DESIGN STRESS, BOT. CENTERLINE — 0.011
(ALLOWED 0.0)

FINAL DESIGN STRESS, TOP CENTERLINE — 1.563
(ALLOWED —2.400)

FINAL DESIGN STRESS, TOP AT D/2 — 0.132 (ALLOWED 0.232)

STRESS TOP AT HARP PT AT RELEASE 0.193 (ALLOWED 0.212)

STRESS BOT. AT HARP PT. AT RELEASE — 2.714 (ALLOWED —3,000)

STRESS TOP AT D/2 AT RELEASE — 0.064 (ALLOWED 0.212)

STRESS BOT. AT D/2 AT RELEASE — 1.496 (ALLOWED —3.000)

(INCLUDES E. SHORTNING AND RELAXATION)

GIR. CONCRETE STRENGTH AT STRAND RELEASE = 5.000, AT 28 DAYS = 6.000

WS = 0.719, WC = 0.139, E = 2,330, FCL = 10.000

RISE IN HARPED STRANDS 28.250, FO = 3.750

COMPOSITE SECTION I = 025842., Y BOT = 33.81

FIG W = 83.04, Q SLAB = 5043.

ULTIMATE MOMENT REQD 2300. AVAILABLE 2554.

ULTIMATE V AT END OF END BLOCK 148.8, V/BD = 0.682 (ALLOWED 0.731)

STIRRUP SPACING AT END OF END BLOCK = 14.39

ULTIMATE V AT CENTERLINE 49.7, STIRRUP SPACING = 116.12

GIR. D.L. END REACTION = 39.7, WITHOUT CURB OR ASPH = 35.6

BREAKDOWN OF STRESSES AT CENTERLINE SPAN

SBDG = 0.496; SBDS = 0.993; SBDC = 0.128; SBXML = 1.199, Live load plus impact stress in ksi; SBPRE = —2.827

Fatigue Life of Prestressed Concrete Beams

The fatigue life of prestressing elements in girders that are not subjected to cracking loads will be the same as bare strands subjected to the same stresses.

There are no known fatigue tests of beams that suffered concrete fractures because of overheight vehicle impacts. There have been fatigue tests of girders that have been cracked by test loads.

Rabbat et al. (4, 5) performed a series of tests primarily to determine the effect of repetitive loading on the behavior and strength of girders with draped and blanketed strands. "Crack formers" were placed at certain meaningful points. Cracks did occur at these points under service load tensile stresses equal to $6\sqrt{f'_c}$. The minimum fatigue life for loadings that produced a stress equivalent to $6\sqrt{f'_c}$ was about 3 million cycles. The minimum fatigue life for loads that gave zero tension in the concrete was 5,000,000 cycles.

Girder fatigue test series performed by Abeles and Barton (6) required static loading to be applied until the prestressing steel began to yield slightly. At this point, the load was removed and complete failure was avoided. Almost complete recovery was obtained with only slight or no visible damage in the compressive zone. The details of this test are too lengthy to describe here; however, the range and upper limits used indicate that girder strands can be stressed to the yield point, and the load can be released and subsequently withstand a large number of fatigue loading cycles.

In another fatigue test series of prestressed girders, Warner and Hulsbos (7) concluded that girder loadings causing flexural cracks to open should not shorten girder fatigue life provided the stresses induced in the strand reinforcement are smaller than the fatigue limit. A following test series (8) of prestressed concrete girders appears to verify the foregoing conclusion. This test series also gives an example solution of fatigue life for an underreinforced prestressed concrete girder.

Hanson et al. (9) carried out a fatigue test to obtain test information on the fatigue life of prestressed concrete I-beams that had been overloaded to cause flexural and inclined cracking prior to repeated loading. Details and complete results are too lengthy to describe here. However, each beam was subjected to an initial static loading of approximately 80 percent of the ultimate flexural capacity of the beam. This load was sufficient to cause significant inclined cracking in both shear spans. The beams were next subjected to repeated load cycles ranging from 19 percent to 45 percent of the ultimate flexural capacity of the specimen. Each beam sustained 2,000,000 cycles of this design loading. The equivalent tensile stress in the concrete was $6\sqrt{f'_c}$. The maximum flexural crack widths during this loading were less than 0.010 in. The cyclic loading was then stopped. Subsequent cyclic loadings were applied at 48 percent to 54 percent of the ultimate flexural capacity. These above-design loadings caused flexural fatigue failures at 458,000 to 1,201,000 cycles.

Summary

The damage caused by an overheight vehicle impacting a prestressed girder is complex to analyze. The effect on the fatigue life of the remaining strands, assuming that one or more strands have been broken, may defy the use of a highly theoretical analysis.

The failure of a strand because of impact by a dynamic load which also shatters surrounding concrete may be primarily a shear failure or a combination of shear and flexure. Some strands may not have failed, but a visual inspection after removing loose concrete may show permanent deformation. These strands should be deleted in computing girder strength. Individual wires showing abrasion or damage should also be deleted. It is reasonable that adjacent strands may not have been stressed beyond the yield point. One practical way of telling whether the strands have been stressed beyond the yield point is the following.

After loose concrete has been removed, a careful visual inspection should be made to determine whether other cracks in the compression flange have "closed." (Closure means a hairline crack.) If these cracks have closed, it is reasonable to assume that the remaining strands have not been stressed beyond the yield point. Although the research data available at this time are not entirely conclusive, present data indicate that these

strands will have a fatigue life equal to that of the original girder provided the following limitations are met:

1. Repair procedures as outlined in this report are followed.
2. If the original girder was designed for tension under working load stresses, the repair must be designed to at least withstand these same tensile stresses with the same factor of safety.
3. The maximum tensile stress in the concrete under working loads must not exceed $6\sqrt{f'_c}$. (AASHTO Article 1.6.6(B).)
4. The maximum allowable prestressing steel stress due to $LL + I$ must be 10 ksi for 7-wire prestressing strands, grades 240 K to 270 K.
5. The maximum tensile stress in the prestressing elements under working loads (after losses) must not exceed $0.6f'_s$.

No recommendation is made regarding the fatigue life of accidentally cracked girders that were prestressed with high-strength bars or rods. This is because there is sufficient fatigue test data of girders stressed with high-strength rods or bars. It can be concluded that damage that does not cause cracking completely through the precompressed area containing high-strength rods should not reduce the original fatigue life.

The foregoing criteria for strands are expected to give a minimum fatigue life of approximately 3,000,000 cycles. The criteria are practical for the most commonly encountered situations within the scope of this study. The criteria are not intended to cover extremely short-span girders or short-span slabs that have live load plus impact stresses that exceed 10 ksi.

See "Guidelines for Selection of Repair Methods" for comments regarding fatigue susceptibility of internal strand splices.

Durability

A major concern expressed by bridge engineers relates to the durability of repairs to damaged girders. Various state-of-the-art techniques used to improve durability are given in this manual. It has been recognized throughout this research that durability of repairs should be, as nearly as possible, equal to the durability of the original construction. The durability of structure components, such as roadway slab construction joints, appears to be accepted by bridge engineers in replacement projects. The durability of repair-in-place projects appears to be less widely accepted.

Present practice substantiates the fact that epoxy mortars and special concrete mixes are available that more than meet strength requirements of original material. It is recognized that a crack that has been caused by normal load cannot be structurally repaired by epoxy injection. Although the epoxy may not fail, a new crack will occur in the adjacent concrete if the member is subjected to the same load that caused the original crack. However, cracks in girders from collision are not caused by normal load stresses. Therefore, by applying preload prior to epoxy injection, and restoring prestress force, structural capacity can be restored which should restore durability. A study made by the Kansas Department of Transportation (10) shows effective use of epoxy to structurally repair stress cracks in normally reinforced concrete girders. Another study by the State Highway Commission of Kansas (11) contains valuable information relative to physical properties of epoxies related to epoxy injection.

The following guidelines should be considered in repairing

damaged prestressed concrete bridge girders (PCB) to achieve acceptable durability:

1. All unsound concrete should be removed and surface preparation should be such that new material placed will be compatible with existing material. New material should have equal or better strength characteristics than original.
2. Epoxy bonding, epoxy grout, and epoxy injection materials and systems should be fully tested and approved, and should be applied by trained personnel. Particular requirements concerning ambient temperatures must be observed.
3. Additional reinforcement to bond new material to existing surfaces should be considered.
4. Preloading should be used (if necessary) to ensure that the repair section will not be subject to greater tensile stress under live load than the original section.
5. Additional prestress force as required to ensure repaired stress levels are no greater than original design stress levels.
6. To further ensure durability, the repaired areas should be sealed with a proven water retardant.
7. Where repair design dictates, commitment should be made to perform periodic preventative maintenance.

Cost

The repair cost of minor damage, such as nicks, spalls, scrapes, gouges, cracks, and exposed strands, is relatively low. The repair can normally be accomplished by agency personnel. The cost of materials is relatively low. Most agencies have the necessary equipment needed to make these repairs. Some traffic control will be required. The estimate based on information and knowledge gained during this project is that the repair cost of minor and/or moderate damage (per girder) will not exceed 10 percent of the cost of replacing one girder.

The repair of severe damage, such as severed strands and major loss of concrete, will probably require the services of a contractor. The need for a contractor might depend on the type of repair (or splice) used. Splice 2 (the metal sleeve splice) and splices 4 and 5 (adding internal prestress) could all be accomplished by agency personnel. Most agencies do not have the equipment to add external prestress, although this equipment has been rented to make repairs.

Traffic control normally would be more extensive and costly for repairing severe damage than for repairing minor and moderate damage.

The number of severed strands may vary from one to as many as ten or more. The loss of concrete may vary from a small portion of concrete to most of the bottom flange. If more than one girder per span is damaged, the cost per girder will normally be less; however, increased traffic control may offset part or all of the savings. Because of the foregoing variables, it is not possible to give precise cost data for the repair of severe damage.

On the basis of information and knowledge gained during this project, it is estimated that the repair cost of severe damage (per girder) will vary from 15 percent to 50 percent of the cost of replacing one girder, depending primarily on the extent of damage.

Cost information received from agencies relative to replacing damaged girders varied as given in the following tabulation (costs are based on only one girder being replaced per bridge):

Agency	Cost per Girder
State of Washington	\$20,000 to \$40,000
State of Iowa	\$30,000
State of Pennsylvania	\$100,000 ^a
State of Illinois	\$40,000
State of Louisiana	\$50,000 to \$55,000
State of New Mexico	\$35,000 approx.
Province of Ontario, Canada	\$50,000
	\$48,000 average

^a May include higher traffic control cost.

Cost information relative to repairing girders in place is as follows:

Agency	Percent Cost of Repair in Place Replacement
State of Washington	50% (Major Repair)
State of Illinois	10% ^a
State of New York	50% (Major Repair)
State of South Carolina	20%
State of New Mexico	15%
	30% Average

^a Additional funds are deposited to provide for replacing epoxy patching at 10-year intervals.

Aesthetics

The emphasis placed on bridge aesthetics may vary between agencies, and it is acknowledged that opinions may differ as to what is poor, acceptable, or good bridge aesthetics. However, general aesthetic guidelines are helpful in determining the aesthetic acceptability of damage repairs.

Accidental damage that occurs during manufacture is normally completely restored architecturally, and these beams should be acceptable for their original intended use.

Certain accidental damage to girders in completed bridges is so severe that girder replacement is necessary. The types of girder replacements that are normally used give aesthetic results that are equal to the original bridge.

Most accidental bridge damages can be repaired in place. These repairs should be judged on the aesthetic acceptability of bridge details. The overall form or appearance is normally more important than minor bridge details. If bridges are viewed by pedestrians passing under the bridge, the aesthetics of details become more important. Pedestrians will have the visual time and ability to notice details.

All of the methods of restoring strength illustrated in this report are considered aesthetically acceptable details with the following possible exception.

Jacking corbels connected with post-tensioned rods may be viewed as being an undesirable aesthetic detail by pedestrians passing beneath the bridge. The motorist view of this detail would be of limited and short duration and, therefore, aesthetically acceptable. Rejection of this detail for aesthetic reasons should be considered when it will be seen in a near view by numerous pedestrians. However, jacking corbels can be extended for the full length of fascia girders and be aesthetically acceptable for any location.

Repair Method to Consider

Selection of the repair method should be based on an objective analysis. The selection of an appropriate repair method depends primarily on the extent and type of damage. Table 2 has been developed to help assess differences between repair methods used to repair serious damage. *Serious damage* is defined as damage requiring splicing of strands and may be accompanied by damage requiring restoring major loss of concrete.

GUIDELINES FOR REPAIR OF DAMAGE

Office Responsible for Repair Method

The best and most consistent repair procedures will result when the same office is responsible for assessment of damage and repair of damage. In many states, district or regional offices inspect damage and make minor repairs. The headquarters bridge office should always be consulted in case of severe or unusual damage. When responsibility is divided, close cooperation is needed to ensure consistently good repair procedures.

Accomplishment of Work

Many states do minor repair work such as repair of concrete nicks, spalls, scrapes, gouges, and injection of cracks, with state forces. Preload is used as appropriate. It is believed that state forces could do additional work such as removal of damaged concrete and installing internal strand splices, metal sleeve splices, and post-tensioning. Training would be required. Jacks required for post-tensioning could be loaned between districts. Internal strand splices could be made in advance. The foregoing work could also be accomplished by contract. Replacing girders would normally be performed by contract.

Table 2. Repair method to consider.

Damage Assessment Factor	Repair Method to Consider			
	Post-tensioning	Internal Splicing	Metal Sleeve Splice	Replacement
Service and Ultimate Load	Excellent	Excellent	Excellent	Excellent
Overload	Excellent	Excellent	Excellent	Excellent
Fatigue	Excellent	Limited	Excellent	Excellent
Adding Strength to Non-Damaged Girders	Excellent	N/A	Excellent	N/A
Combining Splice Methods	Excellent	Excellent	Excellent	N/A
Splicing Tendons or Bundled Strands	Limited	N/A	Excellent	Excellent
Number of Strands Spliced	Limited	Limited	Large	Unlimited
Preload Required	Perhaps	Yes	Possible	No
Restore Loss of Concrete	Excellent	Excellent	Excellent	Excellent
Speed of Repair	Good	Excellent	Good	Poor
Durability	Excellent	Excellent	Excellent	Excellent
Cost	Low	Very Low	Low	High
Aesthetics	Fair *	Excellent	Excellent	Excellent

N/A means not applicable.

* Can be improved to excellent by extending corbels on fascia girder.

Equipment Required

Specialized equipment is needed for each method of repair. However, there are general types of equipment that are usually needed for all repair work. Agencies that do much of their own repair work generally have a hydraulically operated platform mounted on a truck. The simple access provided by free-standing scaffolding may be entirely adequate. In determining access, the high and low work elevations and foundation condition under the work area are important access considerations. The least expensive means of good access should be used.

Concrete Nicks, Spalls, Scrapes, and Gouges

Review "Guidelines for Assessment of Damage" prior to repairing concrete nicks, spalls, scrapes, and gouges. As a minimum, all minor nicks, spalls, scrapes, and gouges should be cleaned and sealed with two coats of a penetrating sealer. This treatment is considered good practice to prevent the intrusion of moisture or other corrosive elements to the prestressing steel.

When a gouge has been assessed and found to require patching, preload should be applied as appropriate prior to patching. More than one type of patching has been successful. If an agency is satisfied with a particular method of patching, continue to use it. State Materials Laboratory approved epoxy mortar has been used successfully. Agency approved epoxy adhesives have been used to prime existing concrete and then filled with epoxy or concrete mortar. Gouge patches should attain required strength prior to removal of preload.

Concrete Cracks

Review "Guidelines for Assessment of Damage" for preload requirements. The following guidelines should lead to successful crack injection. Repair of structural cracks should be accomplished by a company that has been engaged in this type of work for at least 2 years. Certification should indicate that the personnel who will perform the repair are experienced in this work. The material for crack repair should be an epoxy resin adhesive capable of injection into and travel in a crack 0.002 in. (0.05 mm) wide. All cracks to be repaired should be routed to a V-shaped cross section $\frac{1}{2}$ in. deep along their entire length on both sides where cracks extend through. After completion of all routing, cracks should be blown clean with a high-velocity filtered-air jet. The injection ports should be drilled with hollow bits with vacuum attachment in order to remove the dust. Extreme care should be exercised to assure that cracks and drilled holes are open and not obstructed by dust or sand grains. The spacing of the entry ports for the epoxy resin adhesive should be selected so as to assure complete distribution of the material throughout the cracks. Six-in. minimum spacing and 16-in. maximum spacing are approximate guidelines. The entry ports may be pressure fittings, tubeless tire valves, short lengths of $\frac{1}{4}$ -in. copper tubing, or plastic injection ports, as approved by the agency. The entry ports should be sealed in place, and the V-grooves should be sealed to a level flush with the beam surface. All sealing epoxy should be applied with care to prevent leakage during subsequent pumping and should be allowed to cure in accordance with the manufacturer's directions. Injection should be accomplished with an injection pump that automatically

mixes components and provides heat at the nozzle. Pressures up to 150 psi should be sustainable. Pumping should begin at the lower end of the crack and continue until epoxy emerges from the corresponding valve stem on the other side. Low transfer pressure (approximately 15 psi) should be used in the beginning, and as confidence in the structural strength of the beam concrete and epoxy paste is developed, pressures can be increased as required. Other valves along the crack should be kept open (without valves cores or caps) to allow observation of the flow of fluid from the pumping station. When epoxy emerges from adjacent valves, the pumping station valve should be closed and the pump moved to the next valve indicated by the travel of the epoxy. This procedure must be repeated until the crack is completely filled. The epoxy should be allowed to cure without disturbance for 24 hours at 70 F to 90 F, or longer if the ambient temperature is lower. After the epoxy has been allowed to cure completely, the epoxy sealing paste and the valve stems should be ground flush with the original concrete surface. Supervision of this process by the manufacturer's representative should be required until the contractor is familiar with the products and the operation. After completion of repair, consider sealing the exterior face of the fascia girder with pigmented sealer.

Major Loss of Concrete

Major loss of concrete is defined as any loss that reduces clearance to prestressing steel or other reinforcing to the point that, in the opinion of the Engineer, patching is required to preserve durability. No specific upper limit is placed on the amount of concrete that can be repaired in place. Review "Guidelines for Assessment of Damage." Loss of concrete without loss of strands will generally require preloading prior to patching. When loss of strands has occurred, preloading may not be required, depending on the repair method used. Internal strand splicing must take place prior to patching and, therefore, preload will be required to restore compression in the patch. External post-tensioning may restore adequate compression in the patched area, which should be determined by a stress calculation. Use of the metal sleeve splice may not require preloading, provided the sleeve adequately covers the area that will be in tension. Typical calculations are included in this manual.

The construction of the patch itself does not depend on the preceding factors. The following guidelines are recommended. Remove all damaged and shattered concrete to sound concrete as directed by the Engineer. One-half in. deep saw cuts should be made in sound concrete to provide a minimum depth of patch. Extreme care must be used in concrete removal so that exposed prestress strands are not damaged. A chipping hammer may be used away from strands. A rivet gun fitted with a chipping tool may be used for fine work. The Engineer may require removal of concrete adjacent to strands with hand tools only. Chemically clean all exposed strands. Exposed reinforcing steel must be cleaned of heavy rust deposits by sand blasting. Restore bent and severed reinforcing steel. Prepared areas must be cleaned and sand blasted free of contaminants immediately prior to priming and patching with epoxy mortar. All loose materials must be removed by dry sweeping and blowing out with clean, dry compressed air at 90 psi. When the clean and dry areas have been approved by the Engineer, they must be primed with an approved epoxy resin binder forming a 10 to 15 mil thickness. Exposed strands and strand splices (if any)

must be coated with epoxy resin. The concrete surface to be patched must have a temperature of at least 50 F and rising. The epoxy surface should appear shiny and be tacky just before new concrete is placed against it. If the concrete has absorbed the adhesive, as evidenced by a dull appearance, apply another coat. Place approved epoxy mortar or concrete. Forms may be required, depending on amount of concrete to be placed; see Figure 8. Pneumatically placed mortar may be permitted subject to approval by the Engineer. Small expansion bolts ($\frac{3}{8}$ in.) may be used to help hold the patch in place, provided they can be located to avoid strands. Strands and reinforcement will be present to engage larger amounts of concrete.

Prestressing Steel and Concrete Repair

Prestressed concrete girders that have suffered severe damage will require restoring of strength. Splices have been divided into three types, external post-tensioning, internal splicing, and a metal sleeve splice. One of each of these types of splices has been tested and the results are shown in Appendix A. Methods of splicing can be combined. Complete girder replacement is another option.

The splices for repair have the capacity to restore strength of from one to ten $\frac{1}{2}$ -in. diameter 270K broken strands. At least one of these splices could be designed to splice even more than 10 strands. There is no need to restrict the number of strands that can be spliced.

The splices have been shown for standard AASHTO I-beam types. This shape of bridge girder is most commonly used. The splices can be readily modified for use with other shapes. In fact, some of the splices are more efficient in strengthening beams shaped more nearly like a wide flange steel shape.

The calculations in this manual have been kept as simple as possible. The splice calculations are based on no change in stress due to loss of damaged concrete. This provides a simple method of determining the prestress force required to restore strength. Prestress working loads based on simplified prestress loss calculations have been used. The user should refine these calculations, as required, for the particular damage incident. For examples showing the effect of loss of concrete and the possible need and advantages for preloading, see section under "Preloading" in this manual. The calculations shown for the test splices in Appendix A are more sophisticated.

The effect of strands broken on one side of a girder has not been taken into account. The combination of torsional and transverse flexural stress induced by strand eccentricities requires a very complex analysis. It is more practical to assess these stresses in the following manner. Measure the sweep (lateral curvature) of the bottom flange. If the sweep is within the standard tolerance for prestressed girders (generally $\frac{1}{8}$ in. per 10 ft of length), the lateral stresses can be ignored. Impact damages seldom cause sweeping lateral changes in curvature. Sweeping lateral curvature changes are normally caused by faulty fabrication, storage, or construction. Impact damages may cause abrupt lateral curvature. When the sweep is greater than allowable, consider the following approach: (1) Calculate the torsional and flexural stresses induced and use in making splice calculations. (2) Consider jacking the bottom flange into allowable alignment and holding with an added diaphragm. (3) If neither 1 nor 2 is practical, replace the girder.

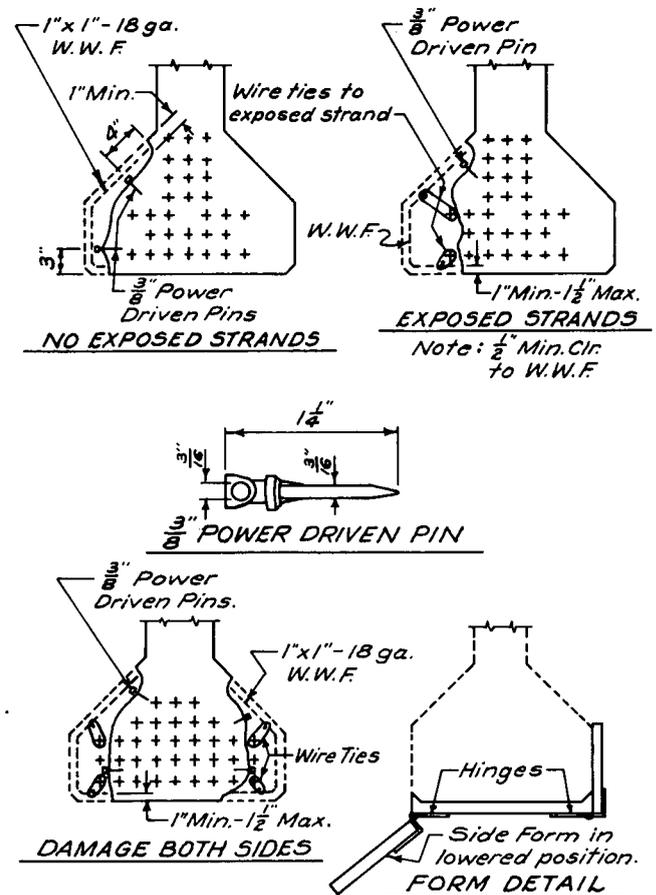


Figure 8. Girder flange patching details. (Courtesy of Illinois DOT)

Calculations are shown for the approximate ultimate strength of each splice. These strengths are compared to the required ultimate strength. Ultimate strength does not usually govern, but should always be calculated. The splices shown are based on the assumption that the broken strands were stressed to their full allowable capacity under service load conditions. Splices have adequate fatigue life provided the limitations shown under "Fatigue" are followed. The internal strand splice may be an exception. The damage is assumed to have occurred at or near the centerline of span (unless noted otherwise).

Nearly all of the splices shown extend above the bottom flange of the girder. For bridges with full-depth intermediate diaphragms, this factor does not necessarily pose a problem. Jacking corbels can be placed adjacent to the diaphragms and the diaphragms pierced and used for additional strength. For splices where the diaphragms are simply an obstruction, enough concrete can usually be removed or cored out from the diaphragm to allow passage of the splice. Depending on damage and diaphragm location, some diaphragms will not interfere. Any portion of the metal sleeve splice not used for strength can be slotted to fit around the diaphragm and preferably additional splice length added.

The minimum recommended concrete strength for cast-in-place splices is $f'_c = 5,000$ psi. Concrete in post-tensioning jacking areas must be well compacted. It should not be difficult to obtain 5,000-psi concrete for the small amounts needed.

The repair procedure for splices 1 through 3 is similar. Recut any broken ends of strands that are frayed. Use this new end of strand in computing strand development length. Tie broken strands in place. After removing loose concrete, epoxy grout the damaged area. Concrete repair areas (including the interface between the new repair and the existing concrete) must be capable of withstanding the same tensile stresses that the original girder was designed for. Strike off the external faces to the original shape. Repair other damage in accordance with guidelines in this report. Two of the splices require a roughened surface at the interface between beams and corbels. These surfaces should be cleaned and roughened to a minimum depth of 1/4 in., and loose particles should be removed. Holes in webs or flanges should be drilled and located to avoid existing strands or reinforcing. Proceed with repairs as shown for each splice.

Post-Tensioning

Splice 1

Description. Splice 1 (Figs. 9, 10, and 11) illustrates the use of two post-tensioned 1-in. diameter ASTM A722-75 smooth Grade 160 rods to restore the loss of prestress of four severed 1/2-in. 270 K strands in an AASHTO beam Type IV.

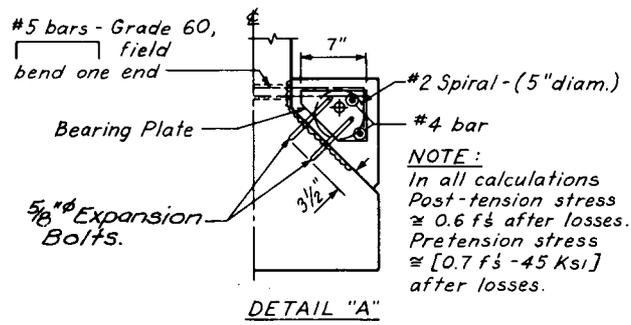
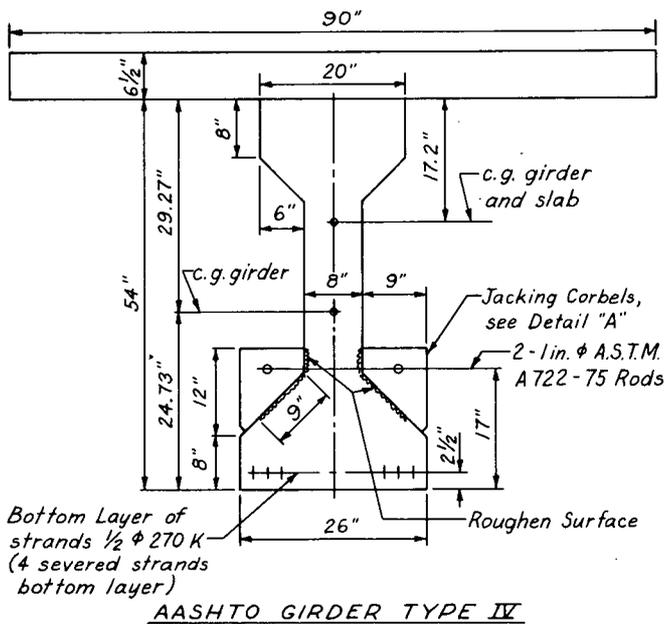


Figure 9. Post-tensioned splice 1.

This splice uses 4-ft-long jacking corbels located outside the damaged area. The jacking corbels should be located in an area where holes can be drilled through the web without interference with harped or draped strands.

Between corbels the high-strength rods are protected by one 1/2-in. minimum inside diameter rigid plastic conduits that are pressure grouted after post-tensioning. To protect exposed polyethylene conduits, coat with carbon black. Plastic tubes have been an accepted means of blanketing strands near the ends of prestressed girders for many years.

The defined yield point of A722-75 bars is at total strain of 0.7 percent and offset of 0.2 percent. These values are compatible with ASTM-A416 strand properties. The corbel length of 4 ft 0 in. was determined by using a minimum tie spacing of 7 1/2 in.

Construction Procedure.

1. Apply preload as required (see Preload No. 2 under "Preloading")
2. Repair concrete damage.
3. After concrete repair has gained required strength, remove preload.
4. Construct jacking corbels and post-tension.

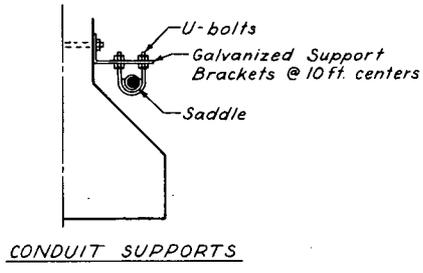
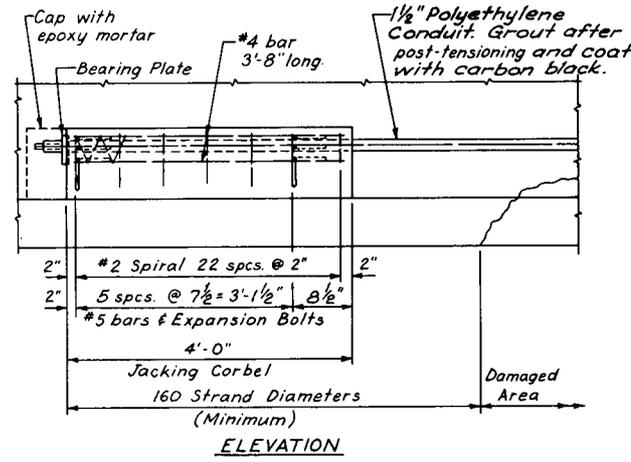
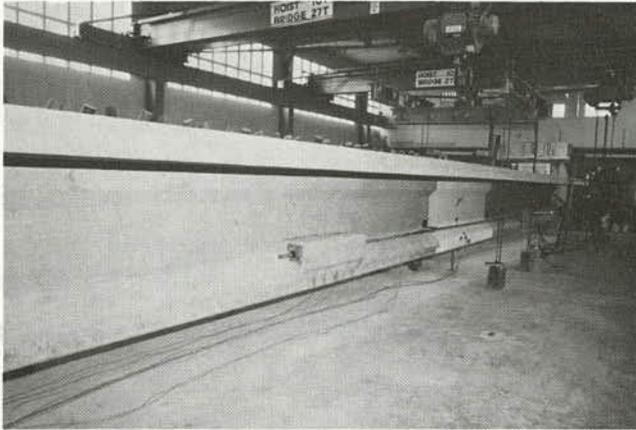
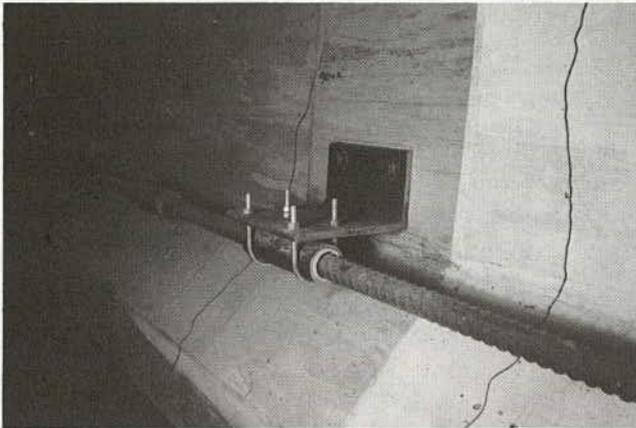


Figure 10. Corbel elevation and U-bolts for post-tensioned splice 1.



a. Post-Tensioned Rod and Corbel



b. U-Bolts and Support Bracket

Figure 11. Post-tensioned splice 1.

Splice 1 Calculations.

1. Given:

- f'_c girder = 5,000 psi. E_c slab assumed to be $0.8 \times E_c$ girder.
- Section properties of girder: $A = 789 \text{ in.}^2$, $I = 260,700 \text{ in.}^4$, $S_{\text{bot}} = 10,540 \text{ in.}^3$
- Section properties of girder and slab: $A = 1,257 \text{ in.}^2$, $I = 573,000 \text{ in.}^4$, $S_{\text{bot}} = 15,570 \text{ in.}^3$
- Live load: assume span length to be 85 ft.
- Girder spacing (S) = 7 ft 6 in.
- Moment one lane (AASHTO 1977) = 1,255 ft-kip (Without Impact).
- Distribution = $S/5.5 = 1.36$.
- $M_{\text{LL}+I} = 1,058 \text{ ft-kip (HS-20)}$.
- $f_{t,\text{bot}}$ (Composite Section), = $1,058 (12) (1,000)/15,570 = 815 \text{ psi (LL+I)}$.
- Assume four severed strands in bottom layer, working stress of severed strands = $[0.7(270,000) - 45,000] (4) (0.153) = 88 \text{ kips}$.

- $f_{\text{bot}} = P/A + M/S = 88,000/789 + 88,000 (22.23)/(10,540 = 111 + 186 = 297\text{-psi (Prestress Loss)}$.
- Added prestress = two 1-in. diameter ASTM A722-75 rods Grade 160 (Smooth).
- Use approximate working load = $0.6 f'_s = 0.6 (160 \text{ ksi}) = 96 \text{ ksi}$.
- Assume conduit supports at 10-ft centers, bending stress $\cong 5.5 \text{ ksi}$.
- Therefore, working load two rods = $(96 - 5.5) (0.785) (2) = 142 \text{ kips}$.
- $f_{\text{bot}} = P/A + M/S = 142/1,257 + 142(19.8)/15,570 = 113 + 180 = 293\text{-psi (Prestress Gain)}$.
- $293 \cong 297$; therefore, two 1-in. diameter Grade 160 rods will replace four severed $\frac{1}{2}$ -in. diameter 270 K strands.
- Strands are assumed to be in bottom layer.

2. Calculate approximate ultimate strength.

- a = depth of equivalent compression zone.
- $a = A_s \cdot f_y / (0.85 f'_c b)$, $f'_c = 4,000 \text{ psi}$, $b = 90 \text{ in.}$
- A_s^* two 1-in. diameter rods = 1.57 in.^2
- $A_s^* f_y = 1.57 (160) (0.85) = 213 \text{ kips}$.
- Assume 34 strands originally, with four severed.
- A_s^* strands = $30 (0.153) = 4.59 \text{ in.}^2$
- $A_s^* f_y = 4.59 (270) (0.85) = 1,053 \text{ kips}$.
- $a = (213 + 1,053) / [0.85 (4.0) (90)] = 4.0 \text{ in. Approx.}$
- Assume c.g. of 34 strands to be 6.0 in. above bottom of girder.
- A c.g. of 30 strands is 6.47 in. above bottom of girder.
- M_u rods = $213 (41.5) / 12 = 736 \text{ ft-kip}$.
- M_u strands = $1,053 (52) / 12 = 4,563 \text{ ft-kip}$.
- M_u (Total) = $736 + 4,563 = 5,299 \text{ ft-kip approx.}$
- DL girder = 822 lb/ft, DL slab = 650 lb/ft.
- $M_{\text{DL}} = 1,329 \text{ ft-kip}$.
- M_u required = $1.3 [\text{DL} + 5/3 (\text{LL} + \text{I})] = 1.3 [1,329 + 5/3 (1,058)] = 4,020 \text{ ft-kip}$.
- Therefore, ultimate strength of splice is adequate.

3. Calculate corbel reinforcing:

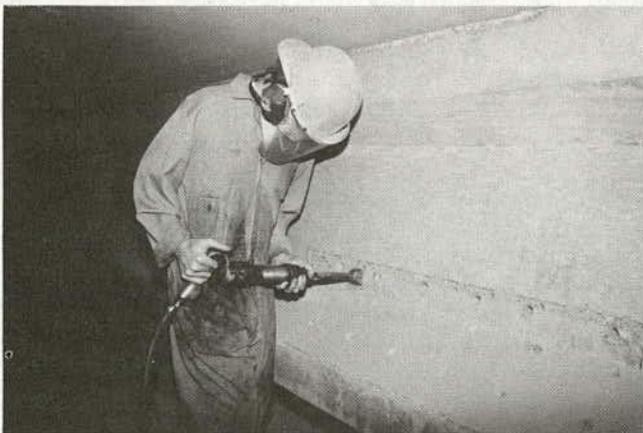
- Area bearing plate = 41 in.^2 , area corbel = 67 in.^2
- $P_u = 0.785 (160) (0.95) = 119 \text{ kips}$ (see AASHTO Article 1.6.17).
- Working load per corbel = $142/2 = 71 \text{ kips}$.
- f_c (Bearing) = $71/41 = 1,730 \text{ psi}$ under plate.
- f_c (Allowable) = 3,000 psi (see AASHTO Article 1.6.6).
- Bearing on corbel at $P_u = 119/67 = 1,780 \text{ psi (Safe)}$.

4. Calculate ties required:

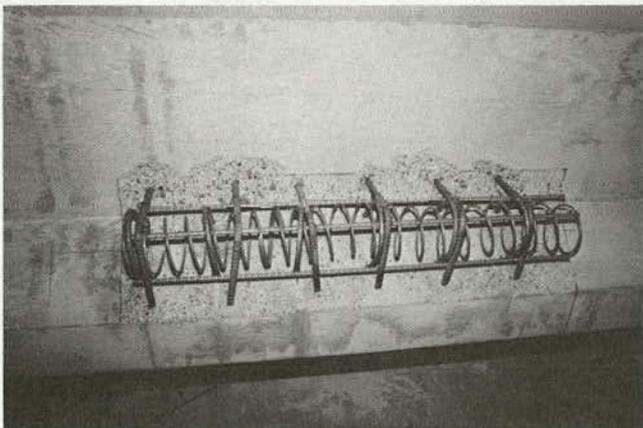
- $A_{v,t} = V_u / \phi f_y \mu$ (AASHTO Article 1.5.35).
- $\mu = 1.0$ concrete interface to be roughened.
- $A_{v,t} = 119,000 / 0.85 (40,000) (1.0) = 3.5 \text{ in.}^2$
- Strength required (at yield) = $3.5 (40,000) = 140 \text{ kips}$.
- Six No. 5 ties = $6 \times 0.31 \times 40 = 74.4 \text{ kips}$. Expansion bolts, 5/8-in. diameter, use 80 percent of allowable for reduced spacing in one direction = $12 \times 7,000 \text{ lb} \times 0.80 = 67.2 \text{ kips}$.
- Total each side = $74.4 + 67.2 = 141.6 \geq 140 \text{ kips}$.
- Use Grade 60 No. 5 ties for added strength across interface.
- Also check shear along interface per AASHTO Article 1.5.35E.
- v_u allowable = 350 psi.
- $140,000 / 12(48) = 243 \text{ psi} \leq 350 \text{ psi}$.

Load Test 8

Load Test 8 in Appendix A was similar to splice 1 above. Two 1-in. diameter 150 K thread bars (ASTM-A722) were post-tensioned to restore prestress lost by severing four 1/2-in. 270 K strands. All loose concrete was removed, primed with an approved bonding agent, and patched with concrete. Prior to patching, five 3/8-in. expansion bolts were installed at an upward angle away from the strands in each side of the girder to help hold the patches in place. The jacking corbels were cast in place against the girder (Appendix A, Figure A-13). No problems were encountered during fabrication, jacking, or testing (see Figs. 11 and 12). The jacking was incremental, 50 percent first side, 100 percent second side, and then 100 percent first side to preclude lateral tensile cracking of the girder flange. One of the corbels was later tested to ultimate load, and the results are shown under "Component Testing" in Appendix A. Preload was not required to restore compression in the patch. Crack injection with epoxy resin was not a part of the test procedure. If this repair had been an actual girder repair, cracks would have been epoxy injected. The first observed crack in the girder



a. Holes Drilled and Surface Being Roughened



b. Reinforcing Steel in Place

Figure 12. Corbel construction for post-tensioned rods.

patch occurred at a test load of 49.7 kips, which was not substantially below the 55.1-kip cracking load of the original uncracked girder. The difference could be explained by not injecting the cracks. The research indicated that post-tensioning is a very successful way of repairing girder damage.

Splice 2

Description. Splice 2 (Figs. 13 and 14) illustrates the use of six post-tensioned 1/2-in. 270 K strands to restore the loss of prestress of three severed 1/2-in. 270 K strands in an AASHTO beam Type III. The strands are post-tensioned individually and are protected from corrosion by a cast-in-place corbel which is continuous for the full length of the splice.

The Washington State Department of Transportation used a similar splice to restore loss of prestress to damaged I-beams in SR 5 Overcrossing No. 12/221 Bridge. Washington State's standard prestressed I-beam shapes differ in shape from AASHTO I-beam types.

Construction Procedure.

1. Apply preload if required (see Preload No. 4 under "Preloading").
2. Repair damaged concrete and preferably cast corbels at this stage.
3. After concrete has gained required strength, remove preload.
4. Post-tension.

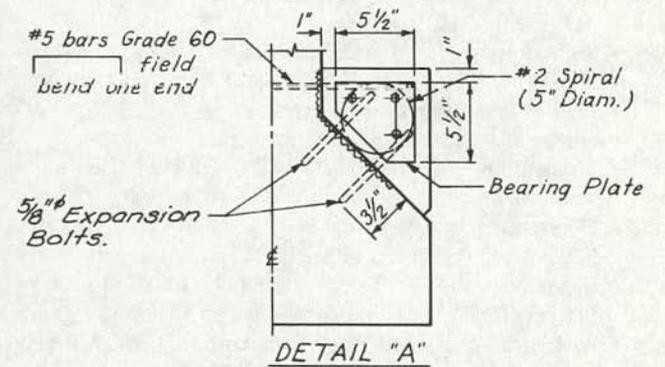
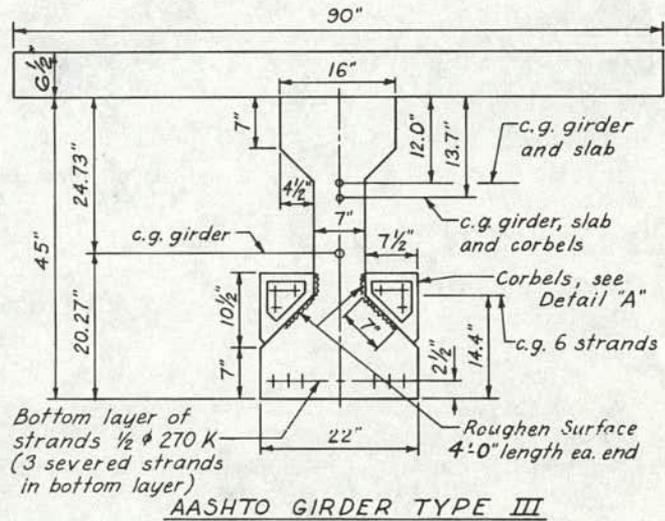
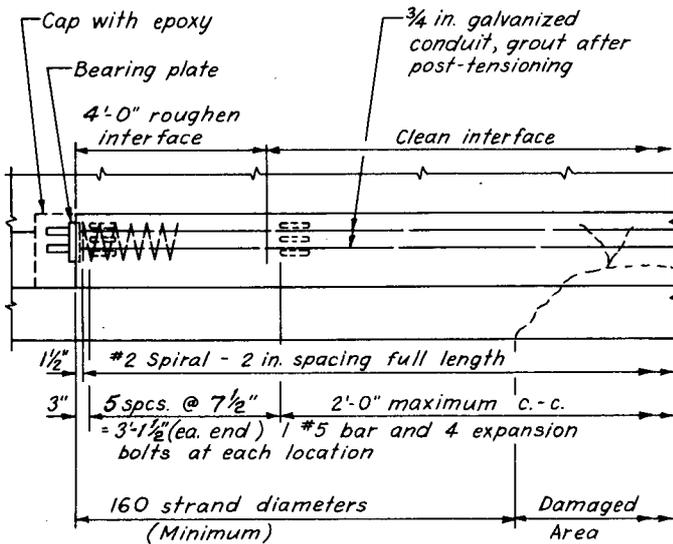


Figure 13. Post-tensioned splice 2.



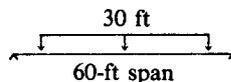
ELEVATION

Figure 14. Corbel elevation for post-tensioned splice 2.

Splice 2 Calculations.

1. Given:

- f'_c girder = 5,000 psi. E_c slab assumed to be $0.8 \times E_c$ girder.
- Section properties of girder: $A = 559.5 \text{ in.}^2$, $I = 125,400 \text{ in.}^4$, $S_{\text{bot}} = 6,190 \text{ in.}^3$
- Section properties of girder and slab: $A = 1,027 \text{ in.}^2$, $I = 326,000 \text{ in.}^4$, $S_{\text{bot}} = 9,895 \text{ in.}^3$
- Section properties of girder, slab and corbels: $A = 1,129 \text{ in.}^2$, $I = 361,500 \text{ in.}^4$, $S_{\text{bot}} = 11,550 \text{ in.}^3$
- Live load: assume span length to be 60 ft.
- Girder spacing (S) = 7 ft 6 in.
- Moment one lane (AASHTO 1977 = 807 ft-kip (Without Impact).
- Distribution = $S/5.5 = 1.36$.
- $M_{\text{LL}+I} = 697 \text{ ft-kip}$ (HS-20 Loading).
- $f_{t,\text{bot}}$ (Composite Section) = $697(12) (1,000)/9,895 = 845 \text{ psi}$ (LL + I).
- Assume three severed strands in bottom layer.
- Working strength of severed strands $\approx [0.7(270,000) - 45,000] 3(0.153) = 66 \text{ kips}$.
- $f_{\text{bot}} = P/A + M/S = 66,000/559 + 66,000(17.77)/6,190 = 118 \text{ psi} + 189 \text{ psi} = 307 \text{ psi}$ (Prestress Loss).
- Weight of corbels = 122 lb/ft.



- Assume corbel length = 30 ft.
- Corbel moment at centerline span = 41.2 ft-kip.
- $f_{t,\text{bot}}$ corbel = $41.2(12) (1,000)/9,895 = 50 \text{ psi}$.
- Added prestress = six 1/2-in. diameter 270 K strands.
- Working stress per strand after losses $\approx 0.6 f'_s = 0.6 \times 270 = 162 \text{ ksi}$, $162(0.153) = 24.7 \text{ kips}$, $24.7(6) = 148 \text{ kips}$.
- Working load = $(162) (0.153) (6) = 148 \text{ kips}$.
- $f_{\text{bot}} = P/A + M/S = 148/1,129 + 148(16.9)/11,550 = 131 + 217 = 348 \text{ psi}$.
- Prestress gain = 348 psi.

- Prestress loss = $50 + 307 = 357 \text{ psi}$.
- $348 \approx 357$; therefore, six 1/2-in. diameter 270 K post-tensioned strands will replace three 1/2-in. diameter 270 K severed strands in bottom layer.
- The ultimate strength of this splice has been calculated and found adequate. See "Overload Capacity Illustration," for calculations.

2. Calculate corbel reinforcing:

- Area bearing plate = 24 in.^2
- $P_u = 270(0.95) (3) (0.153) = 118 \text{ kips}$ (see AASHTO Article 1.6.17).
- Working load per corbel = $148/2 = 74 \text{ kips}$.
- f_c (Bearing) = $74/24 = 3,080 \text{ psi}$ under plate.
- f_c (Allowable) = 3,000 psi (AASHTO Article 1.6.6).
- Overstress = 2.7 percent, which is considered safe with spiral containment.
- Area corbel = 51 in.^2 Bearing on corbel at ultimate load = $118/51 = 2,300 \text{ psi} \leq 3,000 \text{ psi}$.

3. Calculate ties required:

- $A_{v,t} = V_u/\phi f_t \mu$ AASHTO Article 1.5.35.
- $\mu = 1.0$, concrete interface to be roughened.
- $A_{v,t} = 118,000/0.85(40,000) (1.0) = 3.47 \text{ in.}^2$
- Strength required at yield = $3.47(40,000) = 139 \text{ kips}$.
- Six No. 5 ties = $6 \times 0.31 \times 40 = 74.4 \text{ kips}$.
- Expansion bolts, 5/8 diameter, use 80 percent of allowable for reduced spacing in one direction. $12 \times 7,000 \text{ lb} \times 0.80 = 67.2 \text{ kips}$.
- Total each side = $74.4 + 67.2 = 141.6 \geq 139 \text{ kips}$.
- Use Grade 60 No. 5 ties for added strength across interface.
- Also check shear along interface per AASHTO Article 1.5.35E.
- v_u (Allowable) = 350 psi.
- $139,000/10 (48) = 290 \text{ psi} \leq 350 \text{ psi}$.

Load Test 2

Load Test 2 in Appendix A was performed to show the strengthening effect that could be gained by adding additional post-tensioning to a girder not having severed strands. This method could be used to strengthen a girder that was deficient in live load capacity. It might be used to strengthen a girder with environmental or fire damage. The test results of Appendix A show that a 50 percent increase in live load plus impact capacity could be gained by adding the post-tensioning used in Load Test 8. The same jacking corbels were used as in Load Test 8. Because no strands had been severed, incremental jacking was not required. No problems occurred during fabrication, jacking, or testing. Post-tensioning is a viable way of strengthening girders.

Metal Sleeve Splice

Splice 3

Description. Splice 3 (Figs. 15, 16, and 17) illustrates the use of a metal sleeve to splice approximately ten severed 1/2-in. 270 K strands in an AASHTO beam Type IV. Splice 3 is also a good

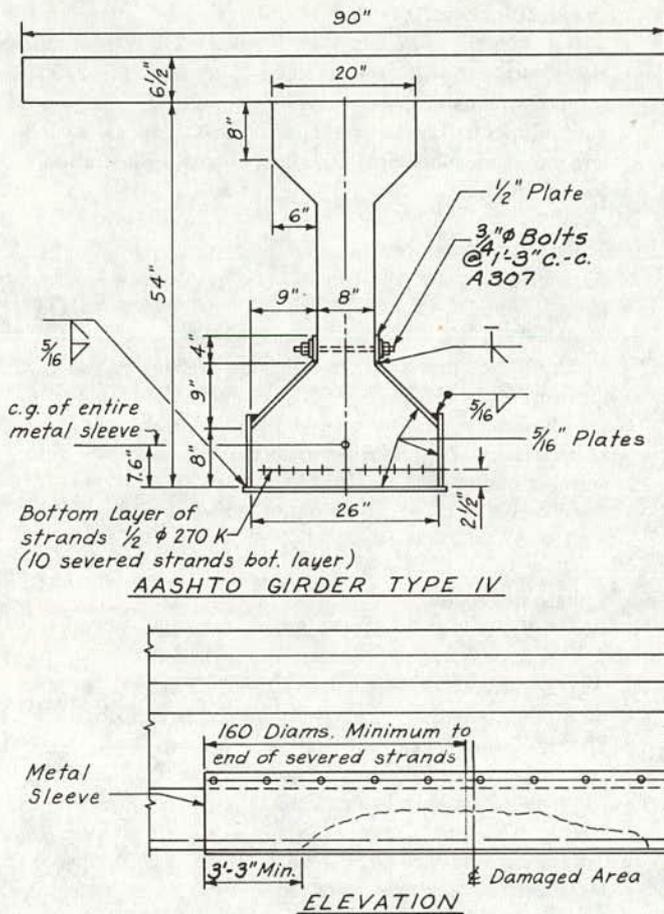


Figure 15. Metal sleeve splice.

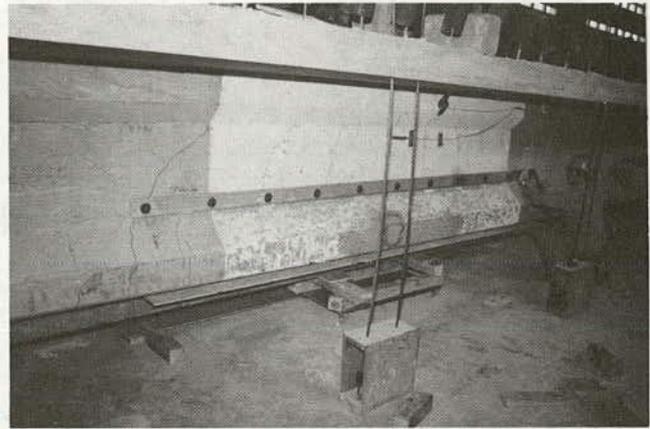
splice to use where loss of concrete occurs, but few if any strands have been damaged. It will restore strength and durability.

No need has been found to recommend a maximum number of strands to be spliced with this repair. The splice could be extended upward to cover the entire web and most of the top flange. Plate thicknesses and the splice development length could be increased. It is probable that this splice could be developed to the point where it would splice an entire portion of a damaged beam.

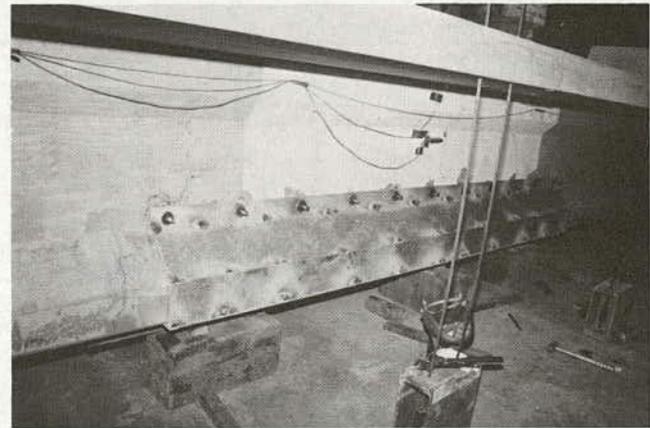
This splice does not restore prestress, although partial or full prestress may be restored by preloading. Preloading may not be necessary (see Preload No. 3). At the splice ends full original prestress will be restored. Any intermediate hairline cracks that may occur are covered by the splice and should not reduce structure integrity or durability.

Note that the minimum length of the splice may be determined by the bond length required to develop the broken strands. The bond length shown agrees closely with AASHTO Standard Specifications for Highway Bridges 1977. The strand is assumed to be bonded at the ends.

Bonding of the metal sleeve to the beam would be by pressure injection of epoxy resin. A clearance of 1/16 in. between the metal sleeve and the beam must be maintained by the use of small 1/16-



a. Top Plate Bolted and Bottom Plate Jacked into Position



b. Sleeve Splice Tack Welded

Figure 16. Metal sleeve splice construction.

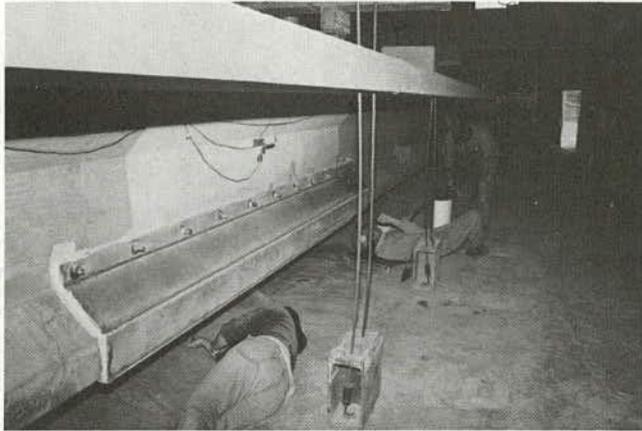
in. thick metal spacers attached by tack welding to the inside surface of the splice plates. The concrete interface must be clean.

The injection pressure will be nominal. No attempt has been made to reduce the splice metal thickness to less than 1/16 in. (see AASHTO Article 1.7.7). If bolts must be raised to avoid draped or harped strands, the top vertical plates and bolts may require redesign.

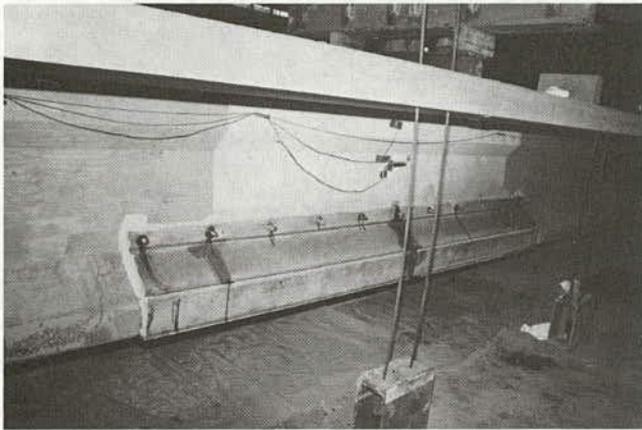
The plates are galvanized A-36 metal. The inside surfaces of the plates (contact surfaces) must be scored vertically by wire brushing prior to assembling. The brushing should be light enough that it scores the surface but removes relatively little of the zinc coating.

Field welds are to be painted with zinc rich paint. For exterior girders, the splice sleeves, in addition, should preferably receive one or more coats of "concrete gray" for aesthetic reasons. Even though this splice is galvanized, or galvanized and painted, it may require infrequent maintenance (depending on climatic conditions) to ensure that its life is equal to that of the original girder. Agencies may opt to use galvanized or painted "weathering steel" to prolong the life of this splice. In addition, 1/16-in. thickness of metal loss due to corrosion may be deducted in calculating the strength of this splice.

The plates may be either precut and welded or cut in the



a. Pumping Epoxy



b. Completed Splice

Figure 17. Metal sleeve splice construction.

field and field welded, depending on the accuracy of field measurements taken. In order to assemble the splice, a field weld will be required at the top outside corner of the bottom flange. If weather conditions are poor, a heat blanket may be wrapped around the sleeve and heat applied for the time required to cure the epoxy resin.

A metal sleeve splice for splicing prestressed concrete piles has been used by Concrete Technology of Tacoma, Washington. A tension test for this splice gave the following results.

Two pile segments were spliced with a 3-ft long, $\frac{3}{16}$ -in. thick sleeve. The space between the sleeve and pile segments was pumped full of epoxy gout. The specimen was loaded in axial tension. The specimen failed by concrete cracking just outside the sleeve and strand slippage at a total load of 210 kips. (The development length of the $\frac{1}{2}$ -in. strands was only 18 in.) The bond stress on the metal sleeve was 213 psi. The stress in the strands at failure was 125,000 psi.

Construction Procedure.

1. Apply preload as appropriate (see Preload No. 3).
2. Repair concrete damage.
3. After concrete repair has gained required strength, remove preload.
4. Proceed with installation of metal sleeve.

Splice 3 Calculations.

1. Given:

- Assume 10 severed strands in bottom layer.
- Working strength per strand = $[0.7(270,000) - 45,000] 0.153 = 22.0$ kips/strand.
- Ultimate strength per strand = $270 \text{ kips} \times 0.153 = 41.3$ kips.

2. Calculate the strength of the metal-sleeve. (Note: the following calculation is approximate; for a more detailed analysis, see Preload No. 3.)

- Use only that portion of the sleeve with c.g. at or below c.g. of severed strands.

$$\begin{array}{l}
 \begin{array}{c}
 1 \text{ in.} \\
 \diagdown \quad \diagup \\
 \frac{5}{16} \\
 \diagup \quad \diagdown \\
 1 \text{ in.} \\
 8 \text{ in.} \quad \quad \quad 8 \text{ in.} \\
 \quad \quad \quad \downarrow \\
 \quad \quad \quad 26 \text{ in.}
 \end{array}
 \quad
 \begin{array}{l}
 \text{Area} = 26(5/16) + 2(8)(5/16) \\
 \quad + 2(1)(5/16) = 13.7 \text{ in.}^2 \\
 \text{Working strength} = 13.7 (20) = 274 \\
 \text{kips.}
 \end{array}
 \end{array}$$

- Yield strength = $13.7 (36) = 493$ kips.
- Allowable number of broken stands = $274/22.0 = 12 +$ or $493/41.13 = 11 +$.
- Bond on sleeve at ultimate strength = $10(41.3)/(26 + 16 + 2) (39) = 241$ psi (350 psi allowable, AASHTO Article 1.5.35E).
- Bond length for strands; $l_d = (f_{su}^* - 2/3 f_{se}) D = [256 - 2/3(144)] \frac{1}{2} = 80$ in. AASHTO Article 1.6.18.
- 160 Diameters = $160 \times \frac{1}{2} = 80$ in.

3. Calculate approximate ultimate strength:

- a = depth of equivalent compression zone.
- $a A_s f_y (0.85 f'_c b) f'_c = 4,000$ psi and $6 = 90$ in.
- A_s entire splice sleeve = 25.4 in.^2
- $A_s f_y = 25.4 \times 36 = 914$ kips.
- Assume 34 strands originally, with ten severed.
- A_s^* strands = $24 (0.153) = 3.67 \text{ in.}^2$
- $A_s^* f_y = 3.67(270) (0.85) = 842$ kips.
- $a = (914 + 842)/0.85(4.0) (90) \cong 5.7$ in., $a/2 = 2.85$ in.
- Assume c.g. of 34 strands to be 6.0 in. above bottom of beam.
- c.g. of 24 strands is 7.46 in. above bottom of girder.
- M_u sleeve = $914(50.0)/12 = 3,810$ ft-kip.
- M_u strands = $842(50.2)/12 = 3,520$ ft-kip.
- M_u (Total) = $3,810 + 3,520 = 7,330$ ft-kip.
- DL girder = 822 lb/ft, DL slab = 650 lb/ft.
- Span length = 85 ft.
- $M_{DL} = 1,329$ ft-kip (neglecting weight of steel splice).
- Beam spacing = 7 ft 6 in. (HS-20 Loading).
- $M_{LL+I} = 1,058$ ft-kip.
- M_u required = $1.3 [DL + 5/3 (LL + I)] = 1.3 [1,329 + 5/3(1,058)] = 4,020$ ft-kip.
- $7,350 \text{ ft-kip} \geq 4,020$
- Therefore, ultimate strength of splice is adequate.

Load Test 9

Load Test 9 in Appendix A was similar to splice 3 above (see Fig. 15). A 13-ft-long metal sleeve was used to splice six severed strands. The strands were anchored 3 ft 3 in. into solid concrete. Loose concrete was removed and patched. The sleeve lapped the strands 5 ft 3 in. minimum (see Fig. A-50). The splice was

tested to ultimate load during Load Test 10 and found to be adequate. When splicing more than six strands, the lap should be 160-strand diameters, as shown in Figure 15. The bottom plate could be held in place with clamps, as shown in Figure 18 bottom. This method would be used with the bottom plate being field welded. The bottom plate and vertical flange plates could be held in place with clamps as shown in Figure 18 top. This method would be used with the vertical and diagonal plates being field welded together. Other clamping devices could be used. Field welding should be kept to a minimum to reduce its effect on galvanizing. The steel in Load Test 9 was galvanized, and the inside faces were scored vertically. A spacing of $\frac{1}{16}$ in. to the concrete was maintained by depositing $\frac{1}{16}$ -in. thick weld deposits. Long-term fatigue susceptibility should be considered when making any tack welds or weld deposits. Spacing plates could be attached with adhesive. Six hours were required to install and weld the plates together.

Bonding of the sleeve to the concrete was accomplished by injecting epoxy resin between sleeve and girder. Sikastik 350 resin was used with two parts Sikadur HI-MOD LV-A and one part Sikadur HI-MOD LV-B. Four staggered injection ports were used in the bottom plate and three each side near the top of the bottom flange. The ends and top of the sleeve were sealed with epoxy mortar; however, inspection openings were left at intervals. When resin reached these levels, they were closed. Injection started near one bottom end of the sleeve; as resin started flowing out of successive ports, fittings were installed and injection followed the flow of the epoxy. The entire space was filled to the top of the sleeve; however, the resin did leak down $1\frac{1}{2}$ in. to the level of the transverse bolts. This leakage could have been prevented by placing mortar around the bolts heads and washers. The time required to mix and inject the resin was 46 min. Seven gallons of resin were used. The resin flowed readily with the aid of a hand pump. It appeared that the only pressure was the nominal head from bottom to top of sleeve. Thirteen $\frac{7}{8}$ -in. transverse bolts were used. Because of the nominal injection pressure, $\frac{3}{4}$ -in. bolts at 1 ft 6 in. centers are recommended. No problems occurred during fabrication, injection, or testing. The sleeve splice is an excellent way of repairing a large volume of damaged concrete, or splicing a large number of severed strands.

Internal Strand Splices

Splice 4

Description. Splice 4 (Figs. 19 and 20), illustrates a method of splicing a single $\frac{1}{2}$ -in. 270 K strand. A number of strands could be spliced in one girder. One advantage of this splice is that it restores strength internally. Combined with preloading, it should completely restore the beam to its original condition. The strand spacing is assumed to be 2 in. both vertically and horizontally. This spacing is the predominant spacing used for $\frac{1}{2}$ -in. strands. It is possible that this method could be used for $1\frac{3}{4}$ -in. strand spacing also.

The stressing is accomplished by torquing the splice to approximately the working strength of adjacent strands. This load may be approximately 22,000 lb for $\frac{1}{2}$ -in. 270 K strands. The severed strands must be accessible. If several strands must be spliced, splicing would start with the innermost strand first.

The strand grip shown in Figure 19 does not require any

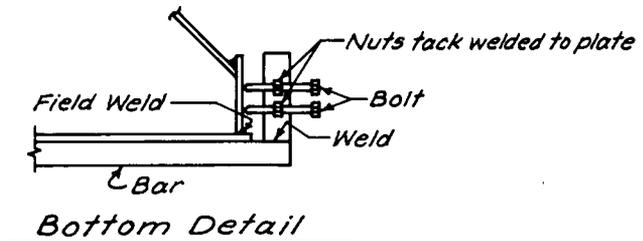
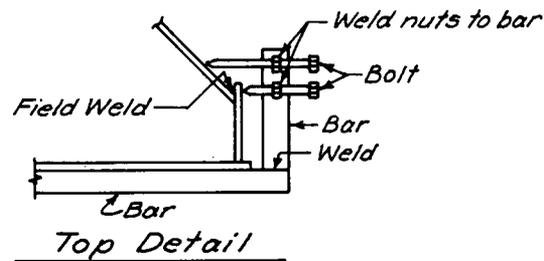


Figure 18. Metal sleeve splice installation details.

modification. The transition from strand to rod is accomplished by using two additional steel splices.

Construction Procedure.

1. Determine preload requirements (see section under "Preloading" for guidance). If stresses permit, it is preferable to apply preload after stressing the splices.
2. Assemble splice, locating splice sleeves and strand grips to allow seating of the strand grips and sufficient thread length in the splice sleeves.
3. Torque lubricated splice sleeve to approximately 22,000 lb (the working strength of one strand). The strand grips must be prevented from rotating during torquing.
4. Repeat steps 2 and 3 for other severed strands.
5. Apply preload.
6. Repair concrete.
7. After concrete repair has gained required strength, remove preload.

Splice 4 Calculations.

1. Given:
 - Working strength per strand = $[(0.7)(270,000 - 45,000)](0.153) = 22$ kips/strand.
 - Splicing rod = 1-in. diameter threaded rod ASTM-A722 Grade 150, $f'_s = 150$ psi.
 - $A_{net} = 0.551$ in.²
 - f_s (Allowable) = $0.6(150,000) = 90,000$ psi.
 - f_s (@ Working Load) = $22,000/0.551 = 40,000$ psi (40,000 = 90,000).
2. Calculate ultimate strengths:
 - Ultimate strength of $\frac{1}{2}$ -in. round 270 K strand = $270,000(0.153) = 41,300$ lb.
 - Ultimate strength of splicing rod = $150,000(0.551) = 82,600$ lb.

To Assemble and Tension Splice

1. Place barrels of splice chucks on strands.
2. Screw one steel splice onto threaded coupling.
3. Screw one rod into threaded coupling to bottom out.
4. Screw other steel splice approximately 2 1/2" onto other rod.
5. Then screw assembly 4, one inch into turnbuckle.
6. Then back steel splice off rod while screwing onto the other threaded coupling.
7. Tension splice by torquing the turnbuckle.

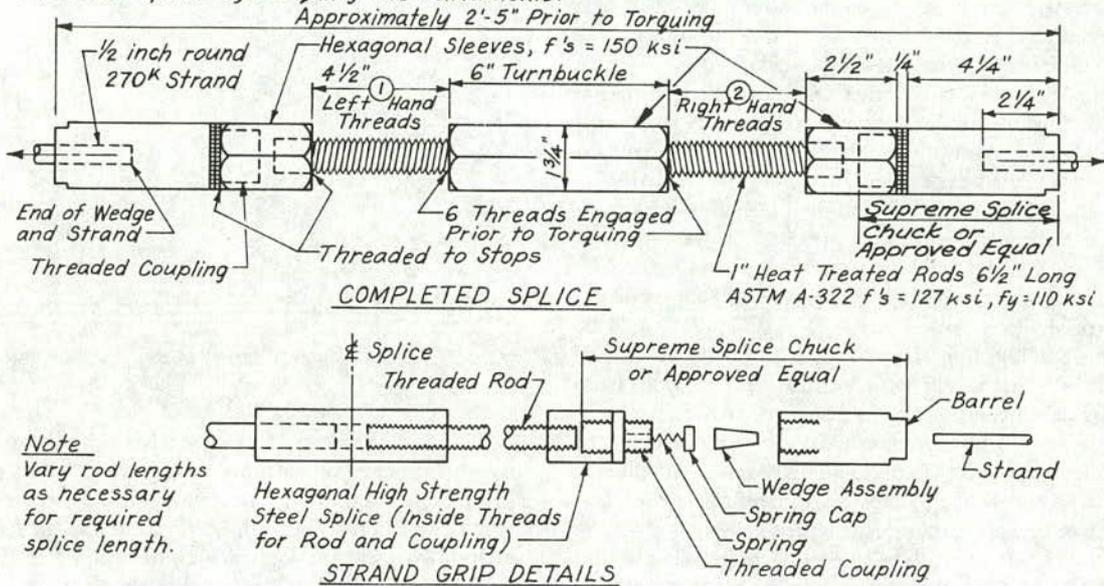
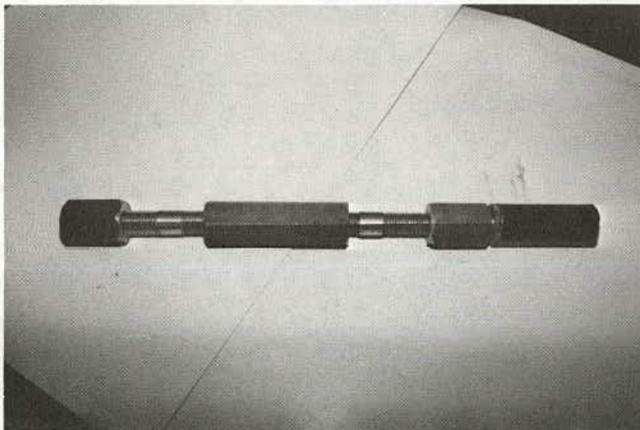
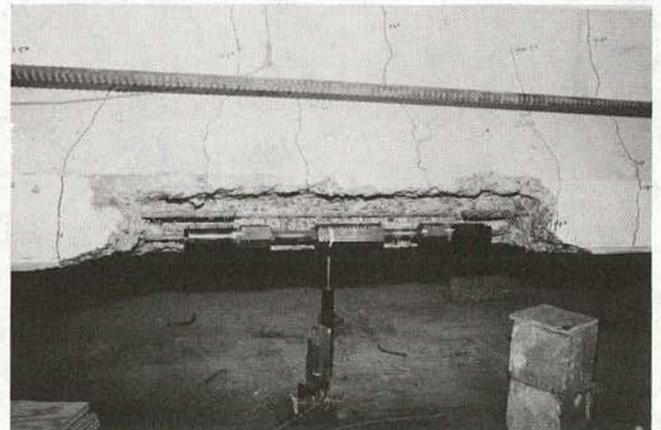


Figure 19. Single-strand internal splice.



a. Single-Strand Splice



b. Single-Strand Splice Installed

Figure 20. Single-strand splice detailed in Figure 19 and used in load test No. 5.

- f_s (at Ultimate Load) = $41,300/0.551 = 75,000$ psi (75,000 psi \leq 150,000 psi).

Load Test 5

Load Test 5 in Appendix A was similar to splice 4 above. Four single-strand splices were used to splice and restore pre-stress lost by severing four 1/2-in. 270 K strands. Concrete was

removed as necessary. During Load Test 4, enough concrete was removed to sever four strands with an electric sander. During Load Test 5, enough additional concrete was removed to install the four single strand splices. Concrete was removed with a chipping hammer, and for careful removal near strands, a chipping tool attached to a rivet gun was used. The strand splices were installed, as shown in Figure 20. Threads were lubricated with a spray-on graphite-based lubricating material. All four strands were tensioned to 24.1 kips per strand, which was ap-

proximately equal to the computed force of 24.3 kips in each of the nonsevered strands. The amount of torque required (433 ft-lb) was read from Figure 21. Torque was applied with a 500-ft-lb torque wrench attached to the special wrench shown in Figure 22. The special wrench was designed with a gap at the end to place over the threaded rods and then slide longitudinally to engage the steel turnbuckle. It could be removed in reverse order. After each fractional rotation of the turnbuckle, the wrench was disengaged from the turnbuckle and a new grip obtained. This process was repeated until the required torque was attained. Cutting the four strands to the correct gap, installing the splices, and torquing to the required tension required 2½ hours. A calculated preload of 30.5 kips was applied. The girder was patched with concrete and the patch allowed to gain required strength prior to removal of preload. The first observed crack in the south girder patch occurred at the test load of 49.7 kips, which was not substantially below the 55.1-kip cracking load of the original uncracked girder. The difference could be explained by not injecting the cracks. The ability to restore compression with preloading was confirmed. It is obvious from the foregoing discussion that this is a splice that is very easy to install and tension. The only machine shop work required to make the splice is the ability to drill and make inside and outside threads. The special wrench adapter also requires routine machine shop work; it could be much lighter in weight than the one shown in Figure 22. It is believed that most states would have the ability to fabricate and install this splice with their own shops and forces. Because of possible fatigue susceptibility

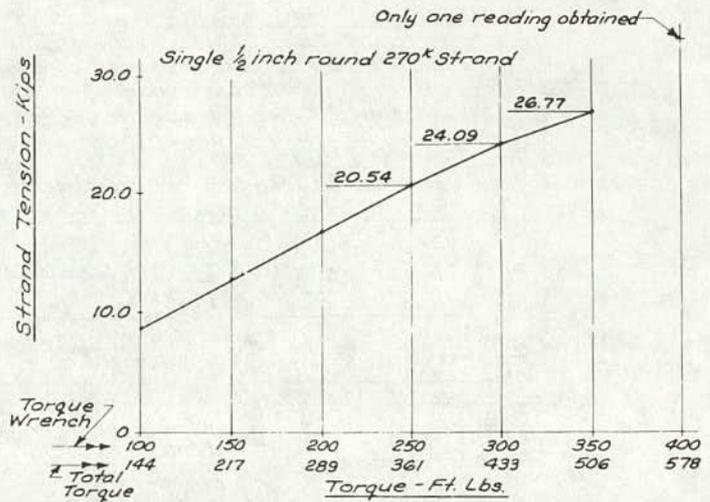
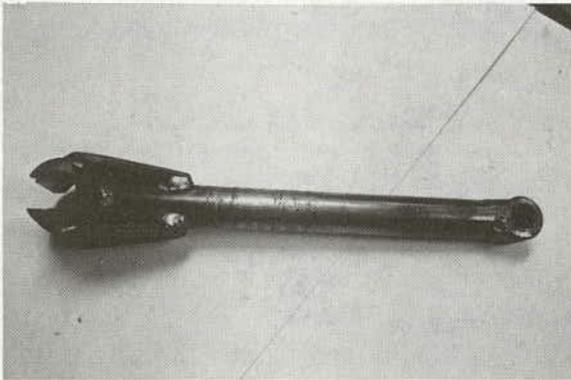
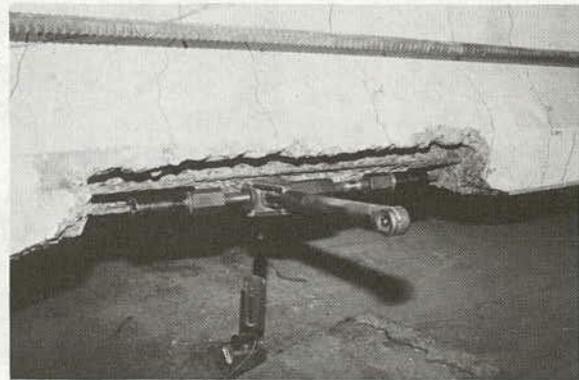


Figure 21. Strand tension versus torque for load test No. 5.

as previously discussed, it may be wise to limit the number of splices used at any one location in a girder. The suggested limit is 25 percent of the total number of strands. Also during the normal inspection of the bridge, this portion of the girder should be carefully inspected for cracking or spalling in the repaired area. From test results, it is believed that a splice rupture will cause local distress.



a. Special Wrench



b. Special Wrench in Position to Torque

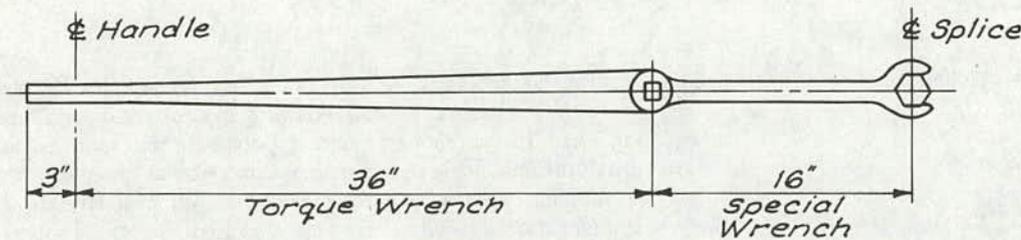


Figure 22. Special Wrench for torquing strands in load test No. 5.

Splice 5

Description. Splice 5 (Figs. 23 and 24) illustrates the use of one 1-in. diameter high-strength bar to directly splice a pair of 1/2-in. diameter 270 K strands. Several pairs of strands could be spliced in one girder. One advantage of this splice is that it restores strength internally. Combined with preloading, it should completely restore the beam to its original condition. Either horizontal or vertical pairs of bars could be spliced. The strand spacing is assumed to be 2 in. both vertically and horizontally. This spacing is the predominant spacing used for 1/2-in. strands.

The stressing is accomplished by torquing the splice sleeve to approximately 44,000 lb, the working strength of two 1/2-in. 270 K strands. The strands must be accessible.

These short length strands, with small diameter swage fittings, were spliced with ordinary "Supreme" strand splices, or equal, as shown in Figure 23. The materials used in the splice are not unusual. The 1-in. round rod or bolt should be ASTM A722 Grade 160 or better. The splice sleeve, transfer plate, and even the rod grip nuts can be fabricated from ASTM-A517 steel. Some machine work is necessary, but not impractical.

Construction Procedure.

1. Determine preload requirements (see section under "Preloading" for guidance). If stresses permit, it is preferable to apply preload after stressing the splices.
2. Insert rod or bolt through transfer plates and connect sleeve, leaving ample take-up in sleeve.
3. Insert strands with swage fittings through transfer plates and connect strand splices.
4. Torque lubricated splice sleeve to approximately 44,000 lb (the working strength of two strands). A groove has been provided to prevent the grip nuts or bolt head from turning when torque is applied. The transfer plates must be prevented from rotating.
5. Repeat steps 2 through 5 with other pairs of severed strands.
6. Apply preload.
7. Repair concrete.
8. After concrete repair has gained required strength, remove preload.

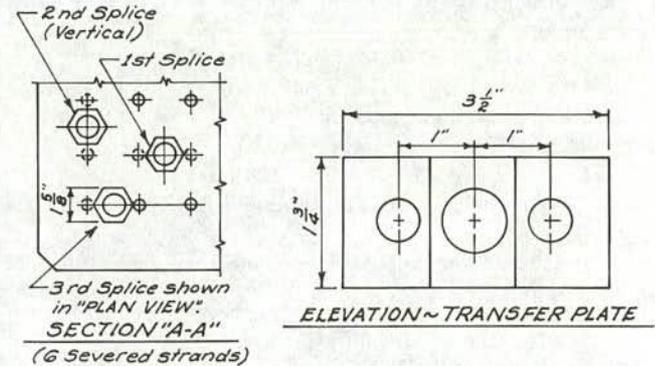
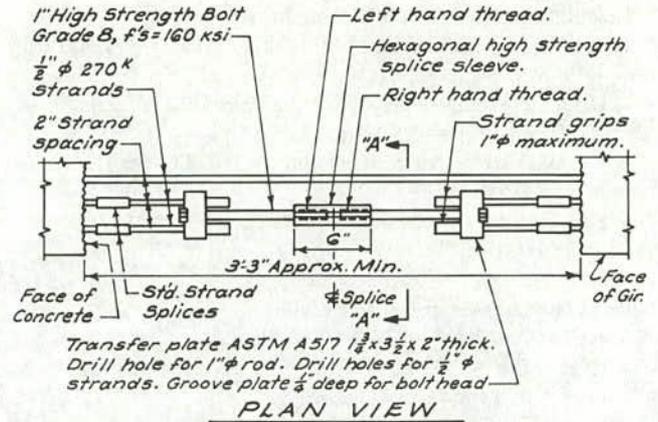
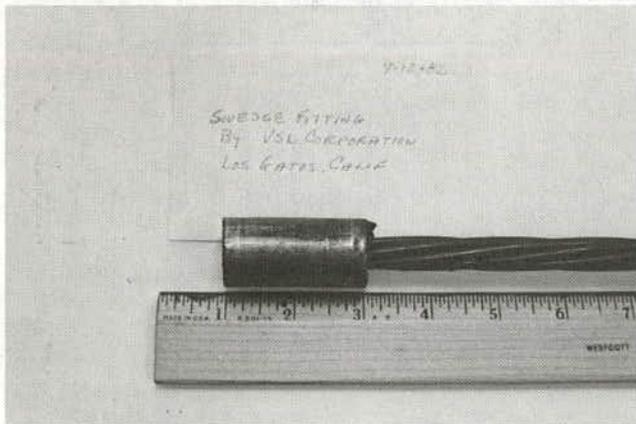


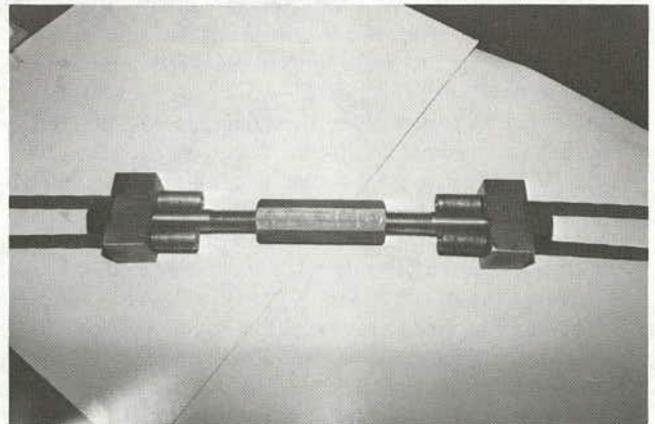
Figure 23. Two-strand internal splice detail.

Splice 5 Calculations.

1. Given:
 - Working strength per 1/2-in. diameter 270 K strand = $[0.7(270,000) - 45,000] 0.153 = 22$ kips/strand.
 - Working stress per splicing rods, ASTM Grade 160 = $0.6(160,000) = 96,000$ psi.
 - One 1-in. diameter threaded rod, $A_{net} = 0.551$ in.²



a. Swage Fitting



b. Splice Assembled

Figure 24. Two-strand internal splice.

2. Calculate stresses in splice components:

- Splicing two strands $f_s = 22,000(2)/0.551 = 79,900$ psi $\leq 96,000$ psi.
- Ultimate strength per $\frac{1}{2}$ in. diameter 270 K strand = $270,000(0.153) = 41.3$ kips/strand.
- Ultimate stress per splicing rod = 160,000 psi.
- $A_{net} = 0.551$ in.²
- Splicing two strands $f_s = 41,300(2)/0.551 = 149,900$ psi $\leq 160,000$ psi.

3. Calculate stresses in transfer plate:

- ASTM-A517 $1\frac{3}{4} \times 3\frac{1}{2} \times 2$ -in. thick.
- (Net width = $1\frac{3}{4}$ in. - 1 in. = $\frac{3}{4}$ in., net thickness = 2 in. - $\frac{1}{4}$ in. groove = $1\frac{3}{4}$ in.).
- $S = bd^2/6 = \frac{3}{4}(1 - \frac{3}{4})^2/6 = 0.383$ in.³
- Working strength moment = $P\ell/4 = 22(2)(2)/4 = 22$ in.-kip
- $f_s = M/S = 22/0.383 = 57.4$ ksi.
- f_s (Allowable AASHTO Table 1.7.1A) = 55 ksi (O.K.)
- Ultimate strength moment = $P_u\ell/4 = 41.3(2)(2)/4 \leq 41.3$ in.-kip.
- $f_s = M_u/S = 41.3/0.383 = 108$ ksi.
- f_s (Allowable AASHTO Table 1.7.1A), $F_y = 100$ ksi (O.K.)
- Because of dimensions, plate moment will be less than 22 in.-kip.

Two-Strand Internal Splice

This splice, as described above, was not tested as a part of the girder tests. However, it was tested as a component test. The results are shown in Appendix A. A plot of splice tension versus torque is shown in Figure A-11. For the reasons given in Appendix A, the single-strand splice is regarded as being superior. Unless clearances mandate the use of the two-strand splice, the single-strand splice is recommended.

Preloading

Preload is simply the application of a temporary vertical load during the repair-in-place of a damaged prestressed member.

There have been damage incidents where a significant portion of girder concrete has been lost, but few, if any, of the prestressing elements severed. Preloading can be used in many instances to restore the girder to its original condition without adding prestress.

The section properties of the remaining cross section of the girder should be computed. If the maximum allowable compressive stress in the reduced bottom flange is exceeded, enough preload during repair must be applied to bring this stress within allowable limits. Assuming the beam is a simple span, the maximum compressive stress would occur under dead load (live load would reduce the stress).

Preload may be used to restore partial or full prestress to the repaired area, which may be needed to reduce or eliminate tension under live load plus impact. The restoring of prestress is also very valuable from a durability standpoint. The need for preload should be calculated for any girder that is repaired in place and has a significant loss of concrete. Preloading may also improve the repair of girders with severed strands. Care should

be taken not to apply excessive preload to the point where cracking of the remaining girder section may occur. Examples of preload are shown in Figures 27 through 30.

Dead-load weight of curb and rail base is not included in the samples. For actual bridge repair designs, any portion of curb and rail base weight carried by the beam being repaired should be included in the calculations. Normally this weight acts on the composite section.

Preload may be applied either by means of a loaded vehicle or by vertical jacking. One means of providing preload by jacking is shown in Figures 25 and 26. Providing preload this way requires less roadway width, which could be an advantage. If preload is provided by a single jack, consideration should be given to the temporary stresses induced in other bridge elements. Part of the jacking load will be transferred through the slab and diaphragms to adjacent girders. If a conclusion is reached that this transfer of stress will cause excessive stress in the slab and diaphragms, more than one jack should be used, with reasonable longitudinal spacing (14 ft is suggested).

Measuring flexural elongation with a strain gage or similar device is probably the most accurate way of determining the actual amount of preload being carried by the damaged beam. If preload is applied with a loaded truck (or trucks) having weight distribution similar to AASHTO HS loadings, AASHTO distribution factors are considered to be sufficiently accurate to determine the preload carried by a particular girder.

The transportation departments of Florida, Washington, Texas, Illinois, Montana, New Mexico, New York, and the province of Ontario, Canada, have reported that they use preload to improve damage repairs.

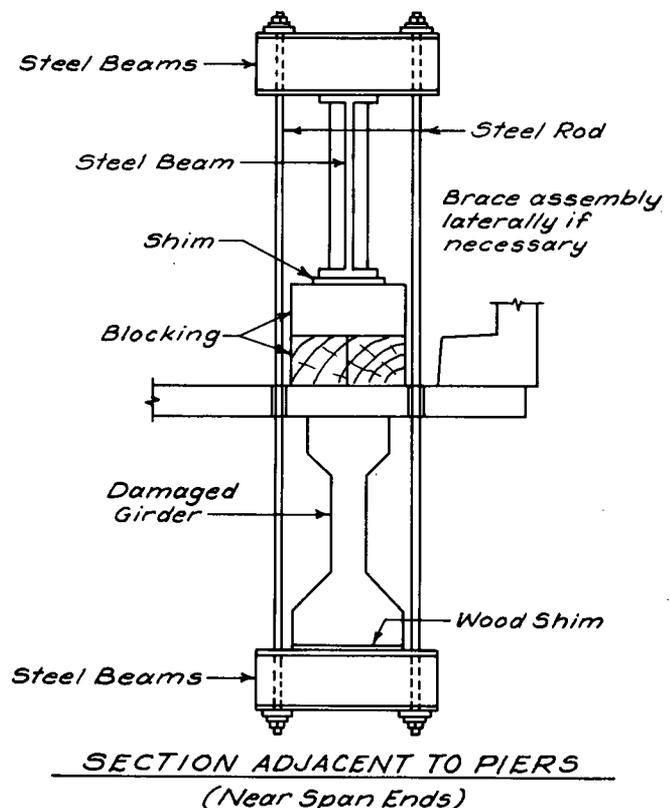


Figure 25. Preload by vertical jacking.

Preload No. 1 Calculations (Fig. 27)

- Given:
 - f'_c girder = 5,000 psi. E_c slab assumed to be $0.8 \times E_c$ girder.
 - Section properties of girder: $A = 789 \text{ in.}^2$, $I = 260,700 \text{ in.}^4$, $S_{\text{bot}} = 10,540 \text{ in.}^3$
 - Section properties of girder and slab: $A = 1,257 \text{ in.}^2$, $I = 573,000 \text{ in.}^4$, $S_{\text{bot}} = 15,570 \text{ in.}^3$
 - Section properties of damaged girder: $A = 659 \text{ in.}^2$, $I = 184,000 \text{ in.}^4$, S (Bottom of Damaged Section) = $7,610 \text{ in.}^3$
 - Section properties of girder and slab minus damaged area: $A = 1,127 \text{ in.}^2$, $I = 402,000 \text{ in.}^4$, S (Bottom of Damaged Section) = $11,230 \text{ in.}^3$
 - Live load: assume span length to be 85 ft.
 - Girder spacing = 7 ft 6 in.
 - Distribution = $S/5.5 = 1.36$.
 - $M_{LL+I} = 1,058 \text{ ft-kip}$ (HS-20).

- Calculate maximum stresses for governing conditions:
 - $f_{\text{bot}} (\text{LL} + \text{I})$, girder and slab = $1,058(12)/15,570 = 815 \text{ psi (t)}$.
 - $f_{\text{bot}} (\text{LL} + \text{I})$, damaged girder and slab = $1,058(12)/11,230 = 1,130 \text{ psi (t)}$.
 - DL girder = 822 lb/ft, DL slab = 650 lb/ft.
 - $M_{\text{DL}} = 1,329 \text{ ft-kip}$.
 - $f_{\text{bot}} (\text{DL Acting on Undamaged Girder}) = 1,329(12)/10,540 = 1,513 \text{ psi (t)}$.
 - $f_{\text{bot}} (\text{Prestress Acting on Undamaged Girder}) = P/A + M/S = 34(22 \text{ kips/strand})/789 + 748(18.73)/10,540 = 948 + 1,330 = 2,278 \text{ psi (c)}$.
 - $f_{\text{bot}} (\text{DL} + \text{LL} + \text{I} + \text{Prestress Acting on the Undamaged Girder}) = 1,513 \text{ psi} + 815 \text{ psi} - 2,278 \text{ psi} = 50 \text{ psi (t)}$.

- Depending on the length of damaged area and number of exposed strands, stress redistribution upward through the roadway slab may or may not occur. If in doubt, the following alternate calculations are recommended, assuming (1) that prestress plus dead load of girder and slab act on the damaged girder section alone; and (2) that prestress plus dead load of girder and slab act on the composite damaged girder and slab:
 - Under assumption (1):
 - $f_{c, \text{bot}} (\text{Prestress}) = 748/659 + 748(23.1)/7,610 = 1,135 + 2,270 = 3,405 \text{ psi (c)}$.
 - $f_{t, \text{bot}} (\text{Dead Load}) = 1,329(12)/7,610 = 2,096 \text{ psi (t)}$.
 - 3,405 psi compression minus 2,096 psi tension = 1,309 psi compression.
 - Under assumption (2):
 - $f_{c, \text{bot}} (\text{Prestress}) = 748/1,127 + 748(34.8)/11,230 = 664 + 2,320 = 2,984 \text{ psi (c)}$.
 - $f_{t, \text{bot}} (\text{Dead Load}) = 1,329(12)/11,230 = 1,420 \text{ psi (t)}$.
 - 2,984 psi compression minus 1,420 psi tension = 1,564 psi compression.
 - Allowable $f_c = 0.4 f'_c = 0.4(5,000) = 2,000 \text{ psi}$.
 - Therefore, preloading is not needed in this case to reduce compressive stresses at the bottom of the damaged section.

- Under assumption (1):
 - $f_{c, \text{bot}} (\text{Prestress}) = 748/659 + 748(23.1)/7,610 = 1,135 + 2,270 = 3,405 \text{ psi (c)}$.
 - $f_{t, \text{bot}} (\text{Dead Load}) = 1,329(12)/7,610 = 2,096 \text{ psi (t)}$.
 - 3,405 psi compression minus 2,096 psi tension = 1,309 psi compression.
- Under assumption (2):
 - $f_{c, \text{bot}} (\text{Prestress}) = 748/1,127 + 748(34.8)/11,230 = 664 + 2,320 = 2,984 \text{ psi (c)}$.
 - $f_{t, \text{bot}} (\text{Dead Load}) = 1,329(12)/11,230 = 1,420 \text{ psi (t)}$.
 - 2,984 psi compression minus 1,420 psi tension = 1,564 psi compression.
 - Allowable $f_c = 0.4 f'_c = 0.4(5,000) = 2,000 \text{ psi}$.
 - Therefore, preloading is not needed in this case to reduce compressive stresses at the bottom of the damaged section.

- Determine the amount of prestress that could be restored by preloading:
 - $f_{c, \text{bot}}$ of damaged section due to dead load plus prestress = 1,309 psi minimum (c).

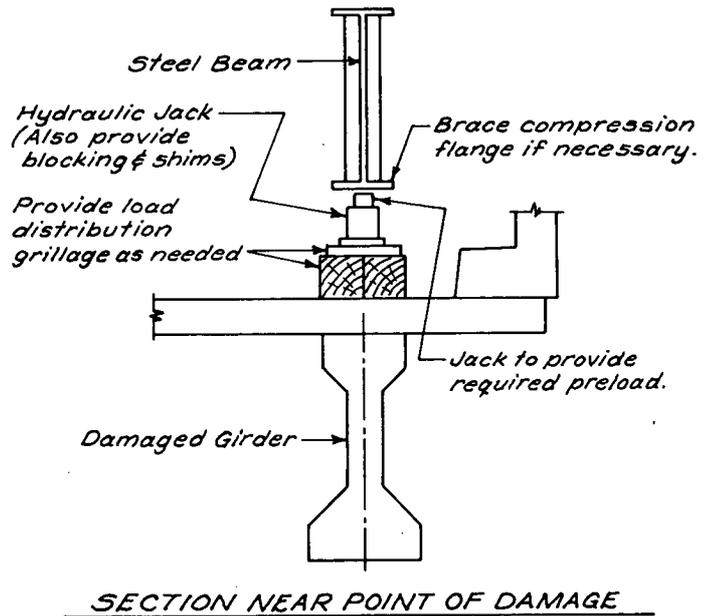


Figure 26. Preload by vertical jacking.

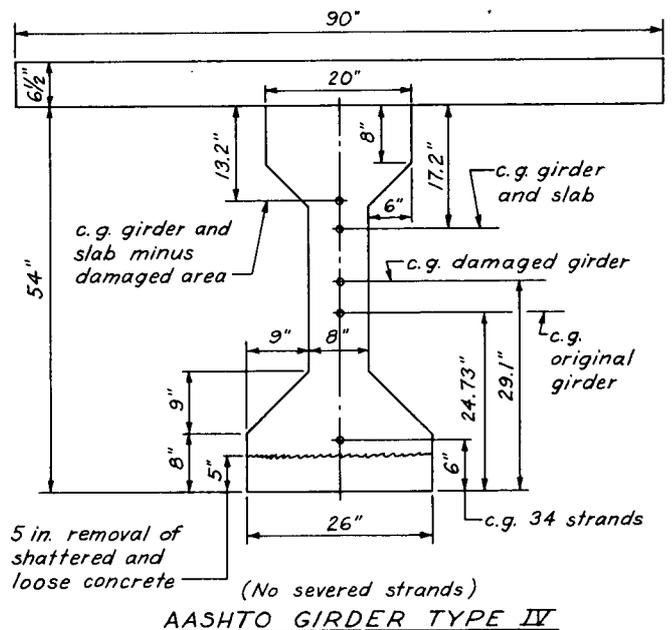


Figure 27. Preload No. 1.

- $f_{t, \text{bot}}$ of damaged section due to full LL + I = 1,130 psi (t) (Load Acting on Composite Damaged Section).
- 1,309 psi compression minus 1,130 psi tension = 179 psi compression.
- Therefore, the girder could be preloaded for full LL + I, and original prestress restored at bottom of patched area.
- 49.8 kips preload per girder
- $P = 4 M_{\text{LL+I}}/\ell = 4(1,058)/85 = 49.8 \text{ kips}$

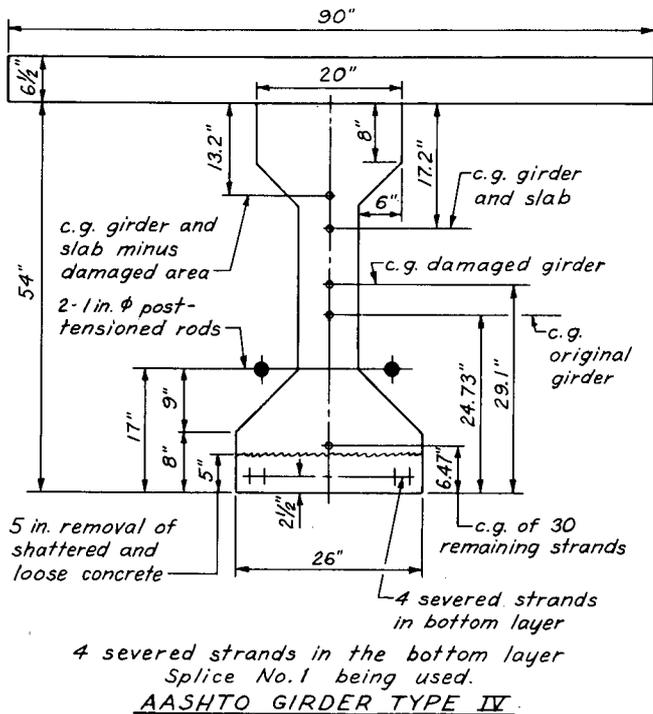


Figure 28. Preload No. 2.

- If preloading is not used, the tensile stress in the repaired area at the bottom of the girder would be 815 psi, the tensile stress due to LL + I. The normal allowable tension = $6\sqrt{f'_c}$ (AASHTO Article 1.6.6(B): $6\sqrt{f'_c} = 6\sqrt{5,000} = 424$ psi. 815 psi - 424 psi = 391 psi greater than allowable. Or 815 psi tension $\cong 11.5\sqrt{f'_c}$, which is exceedingly high (see AASHTO Article 1.6.6B), and would likely crack the repaired area.
- Overloads would increase the probability of cracking. Therefore, preloading is necessary and should be placed on the girder prior to replacing the lost concrete. The preload should remain on the girder until the repair has gained required compressive strength.

Preload No. 2 Calculations (Fig. 28)

1. Given:

- f'_c girder = 5,000 psi. E_c slab assumed to be $0.8 \times E_c$ girder.
- Note: girder, span length, girder spacing, loading and section properties are the same as Preload No. 1.
- $M_{LL+I} = 1,058$ ft-kip.
- $M_{DL} = 1,329$ ft-kip.
- Compression at the bottom of the damaged girder will not be excessive. The loads and damaged area are the same as Preload No. 1, and the remaining prestress is less.

2. Determine if preloading is required:

- The only prestress at the bottom of the girder in the damaged area will be that furnished by the two 1-in. diameter post-tensioned rods.
- f_c (Post-Tensioning) = $P/A + M/S = 142/1,257 + 142(19.8)/15,570 = 293$ psi (c).
- f_t (LL + I) = $M/S = 1,058(12)/15,570 = 815$ psi (t).

- 815 psi tension minus 293 psi compression = 522 psi total tension.
- Assume allowable tension = $6\sqrt{f'_c} = 6\sqrt{5,000} = 424$ psi.
- Net tension in excess of 424 psi = $522 - 424 = 98$ psi.

3. Determine if preloading plus the added post-tensioning can reduce the LL + I tensile stress to 50 psi, the same as the original girder design:

- Assume that prestress plus dead load of girder and slab act on the damaged girder section alone.
- $f_{c, \text{bot}}$ (Prestress) = $30(22)/659 + 660(22.63)/7,610 = 1,000$ psi + 1,960 psi = 2,960 psi (c).
- $f_{t, \text{bot}}$ (Dead Load) = $1,329(12)/7,610 = 2,100$ psi (t).
- 2,960 psi compression minus 2,100 psi tension = 860 psi compression.
- Assume that prestress plus dead load of girder and slab act on the composite damaged girder and slab.
- $f_{c, \text{bot}}$ (Prestress) = $30(22)/1,127 + 660(34.33)/11,230 = 586$ psi + 2,018 psi = 2,604 psi (c).
- $f_{t, \text{bot}}$ (Dead Load) = $1,329(12)/11,230 = 1,420$ psi (t).
- 2,604 psi compression minus 1,420 psi tension = 1,184 psi tension.
- Additional stress needed to reduce tension to 50 psi under LL + I = 815 psi - 293 psi - 50 psi = 472 psi.
- $M = fS = 0.472(15,570)/12 = 612$ ft-kip.
- $P = 4M/\ell = 4(614)/85 = 28.9$ kips.

↓ 28.9 kips preload per girder

- f_t preload at bottom of damaged section = $M/S = 612(12)/11,230 = 654$ psi (t).
- Minimum compression at bottom of damaged section = 860 psi compression minus 654 psi tension = 206 psi compression (with 28.9 kips preload).

4. Review result:

- f at bot. of repaired girder = f_t (LL + I) + f_c (Post-Tensioning) + f_c (Preload) = 815 psi - 293 psi - 472 psi = 50 psi (t).
- Therefore, post-tensioning plus preload restores the full loss of prestress due to four severed strands plus loss of concrete section.
- Procedure: Place preload prior to replacing the lost concrete. Keep preload on the girder until the repair has gained required compressive strength. Remove preload and proceed with post-tensioning.

Preload No. 3 Calculations (Fig. 29)

1. Given:

- f'_c girder = 5,000 psi. E_c slab assumed to be $0.8 \times E_c$ girder.
- Note: girder, span length, girder spacing, loading and section properties (prior to placing metal sleeve) are the same as Preload No. 1.
- $M_{LL+I} = 1,058$ ft-kip.
- $M_{DL} = 1,329$ ft-kip.
- Compression at the bottom of the damaged girder will not be excessive. The loads and damaged area are the same as Preload No. 1, and the remaining prestress is less.
- Assuming preloading is not used, there will be no prestress at the bottom of the girder in the damaged area.

- Section properties of entire composite girder including metal sleeve: A_s (Sleeve) = 25.4 in.², $n = 6.8$ for $f'_c = 5,000$, A_c (Sleeve) = 6.8(25.4) = 173 in.², $A_{total} = 1,257 + 173 = 1,430$ in.², $I_{total} = 710,000$ in.⁴, S_{bot} (Girder) = 21,310 in.³, S_{bot} (Metal Sleeve) = 21,100 in.³
- f_t (LL + I) = 1,058(12)/21,310 = 596 psi (t).

2. Determine if preloading can reduce the LL + I tensile stress to 50 psi, the same as the original design:

a. Assume that prestress plus dead load of girder and slab act on the damaged girder section alone:

- $f_{c,bot}$ (Prestress) = 24(22)/659 + 528(21.6)/7,610 = 800 psi + 1,500 psi = 2,300 psi (c).
- $f_{t,bot}$ (Dead Load) = 1,329(12)/7,610 = 2,095 psi (t).
- 2,300 psi compression minus 2,095 psi tension = 205 psi compression.

b. Assume that prestress plus dead load of girder and slab act on the composite damaged girder and slab:

- $f_{c,bot}$ (Prestress) = 24(22)/1,127 + 528(33.34)/11,230 = 468 + 1,567 = 2,035 psi (c).
- $f_{t,bot}$ (Dead Load) = 1,329(12)/11,230 = 1,420 psi (t).
- 2,035 psi compression minus 1,420 psi tension = 615 psi compression.
- Additional stress needed to reduce tension to 50 psi under LL + I = 596 - 50 = 546 psi.
- $M = fS = 0.546(15,570)/12 = 708$ ft-kip.
- f_t at bottom of damaged section due to $M = 708$ ft-kip = 708(12)/11,230 = 757 psi (t).
- Total tension = 757 - 205 = 552 psi (t).
- This tension is considered to be too high. If the concrete is sound and no cracks are present at the top of the damaged area, a temporary tension of $6\sqrt{f'_c} = 424$ psi should be acceptable.
- Allowable $f_t = 424 + 205 = 629$ psi. Allowable Moment = 629/757(708) = 588 ft-kip. Allowable pre-load: $P = 4M/\ell = 4(588)/85 = 27.7$ kips.

↓ 27.7 kips preload per girder.
↑ 85-ft span ↑

3. Review result:

- Amount of prestress restored = 588(12)/15,570 = 453 psi.
- Final stress at bottom of girder after installing splice sleeve = f_t LL + I - f_c preload = 596 (t) - 453 (c) = 143 psi (t).
- 143 - 50 = 93 psi more tension than the original girder design.
- Therefore, preload restores 93 psi less than full prestress.

4. Assume preloading is not used. Since no compression is induced by preloading, the tensile stress will simply be f_t LL + I = 596 psi (t) at the bottom of the repaired girder. The stress at the bottom of the metal sleeve = f_t LL + I = $M(n)/S = 1,058(6.8)(12)/21,110 = 4.1$ ksi.

- The stress in the concrete at the top of the metal sleeve assuming dead load plus prestress acting on the damaged section is:
- f_t LL + I = $M_y/I = 1,058(12)(12.3)/710,000 = 220$ psi (t).
- f_t DL = $M_y/I = 1,329(12)(8.1)/184,000 = 702$ psi (t).
- f_c prestress = $P/A + M_y/I = 528/659 + 528(21.64)(8.1)/184,000 = 1,304$ psi (c).

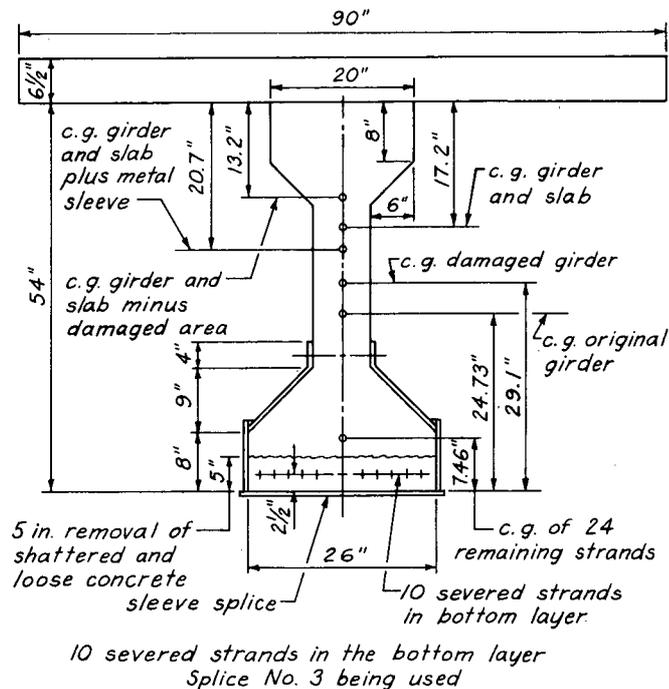


Figure 29. Preload No. 3.

- Total compression = 1,304 - 922 = 382 psi.
- The tension of 596 psi at the bottom of the girder is higher than the normal allowable stress of $6\sqrt{f'_c} = 424$ psi. However, the damaged area is encased by the metal sleeve. There is a compressive stress at the top of the sleeve of 382 psi. The stress in the metal sleeve is 4.1 ksi. Any potential cracking will be confined within the area of the sleeve.
- Based on the preceding calculations and the results of load Test 9, shown in Appendix A, preload is not required.

Preload No. 4 Calculations (Fig. 30)

1. Given:

- f'_c girder = 5,000 psi. E_c slab assumed to be $0.8 \times E_c$ girder.
- Section properties of girder: $A = 559.5$ in.², $I = 125,400$ in.⁴, $S_{bot} = 6,190$ in.³
- Section properties of girder and slab: $A = 1,027$ in.², $I = 326,600$ in.⁴, $S_{bot} = 9,895$ in.³
- Section properties girder, slab, and corbels: $A = 1,129$ in.², $I = 361,500$ in.⁴, $S_{bot} = 11,550$ in.³
- Section properties of damaged girder: $A = 449$ in.², $I = 81,000$ in.⁴, $S_{bot} = 4,170$ in.³
- Section properties of girder and slab minus damaged area: $A = 917$ in.², $I = 212,000$ in.⁴, S_{bot} (of damaged girder) = 6,680 in.³
- Live load: Assume span length to be 60 ft.
- Girder spacing (S) = 7 ft 6 in.
- Distribution = $S/5.5 = 1.36$.
- $M_{LL+I} = 697$ ft-kip (HS = 20).

2. Calculate stresses in undamaged girder:

- f_{bot} (LL + I), girder and slab = 697(12)/9,895 = 845 psi (t).

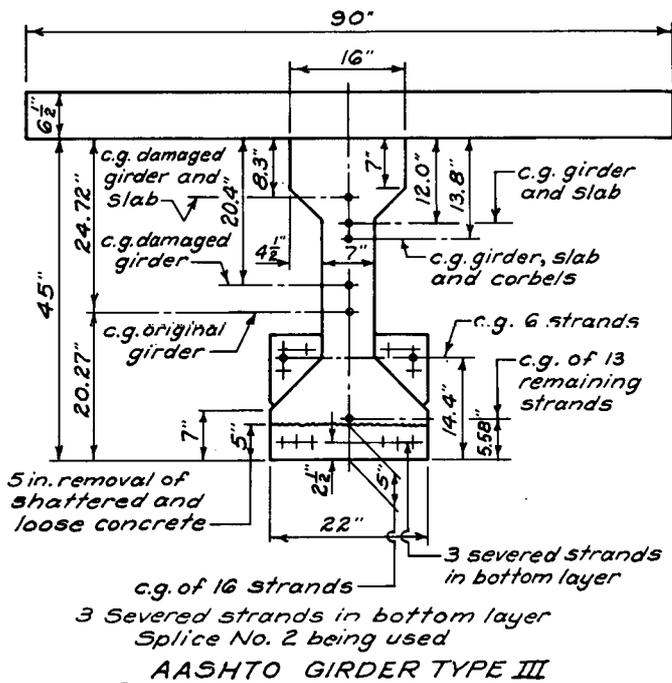


Figure 30. Preload No. 4.

- f_{bot} (LL + I), acting on girder, slab, and corbels = $697(12)/11,550 = 724$ psi (t).
- DL girder = 583 lb/ft, DL slab = 650 lb/ft.
- $M_{DL} = 555$ ft-kip, $f_t = 555(12)/6,190 = 1,076$ psi (t).
- f_{bot} , original prestress (16 strands) acting on undamaged girder = $P/A + M/S = 16(22)/559 + 16(22)(15.27)/6,190 = 1,498$ psi (c).
- f_{bot} DL + LL + I + Prestress (prior to damage) = $1,076 + 845 - 1,498 = 423$ psi tension.
- Assume allowable = $6\sqrt{f'_c} = 6\sqrt{5,000} = 424$ psi (t).

3. Calculate stresses in damaged girder:

- Assume that prestress plus dead load of girder and slab act on the damaged girder section alone:
 - $f_{c,bot}$ (Prestress) = $13(22)/499 + 13(22)(19.0)/4,170 = 1,940$ psi (c).
 - $f_{t,bot}$ (Girder) = $555(12)/4,170 = 1,597$ psi (t) (DL).
 - $f_{total} = 1,597$ (t) + $1,940$ (c) = 343 psi (c).
- Assume that prestress plus dead load of girder and slab act on the composite damaged beam:
 - $f_{c,bot}$ (Prestress) = $13(22)/917 + 13(22)(31.1)/6,680 = 1,643$ psi (c).
 - $f_{t,bot}$ (Girder) = $555(12)/6,680 = 997$ psi (t) (DL).
 - $f_{total} = 997$ (t) + $1,643$ (c) = 646 psi (c). (Note: the weight of the corbels is assumed to act on the composite damaged section and will reduce the compressive stress in the damaged portion from 343 (c) to 269 (c) or 646 (c) to 572 (c) (psi). f_{bot} corbel DL = $(41.2)(12)/6,680 = 74$ psi (t).)
 - Allowable $f_c = 0.4 f'_c = 0.4(5,000) = 2,000$ psi (c). Therefore, preloading is not needed in this case to reduce compressive stresses.

4. Calculate stresses assuming preloading is not used. The only prestress at the bottom of the girder in the patch area will be that furnished by the six post-tensioned strands:

- $f_{c,bot}$ (Girder Prestress) = $148/1,129 + 148(16.9)/11,550 = 348$ psi (c).
- f_t (LL + I) = $697(12)/11,550 = 724$ psi (t).
- $f_{total} = 724$ (t) - 348 (c) = 376 psi (t).
- Assume allowable tension = $6\sqrt{f'_c} = 424$ psi. Therefore, the girder has been restored to its original condition and preloading is not needed.

5. If for any reason the corbels are not cast prior to casting the girder concrete patch, then the weight of the corbels will also produce tension in the concrete patch.

- f_t (DL corbels) = $M/S = 41.2(12)/9,895 = 50$ psi (t).
- The total tension at the bottom of the girder in the damaged area would be $376 + 50 = 426 \approx 424$ psi (t) allowable.

6. Calculate stresses at a section adjacent to the damaged area after repair:

- f_t (DL) = $M/S = 555(12)/6,190 = 1,076$ psi (t).
- f_t (DL corbels) = 50 psi (t).
- f_t (LL + I) = $M/S = 697(12)/11,550 = 724$ psi (t).
- $f_{c,bot}$ (13 Strands) = $P/A + M/S = 13(22)/559 + 13(22)(14.7)/6,190 = 1,190$ psi (c).
- $f_{c,bot}$ (Added Prestress) = $P/A + M/S = 148/1,129 + 148(16.9)/11,550 = 348$ psi (c).
- f_{bot} (Repaired) = $1,076$ (t) + 50 (t) + 724 - $1,190$ (c) - 348 (c) = 312 psi (t).
- Assumed allowable = 424 psi (t).

Girder Replacement

The authors believe that nearly all accidental damage can be repaired in place. A combination of repair-in-place methods should be adaptable to a wide spectrum of damages. The most difficult damages to repair may be those that occur in areas of harped pretensioned strands or draped post-tensioned tendons. A sleeve splice could be extended upward to splice these areas. The difficulty of replacing girders made continuous for live load may lead to greater acceptance of innovative repair-in-place methods.

Most state agencies prepare replacement plans and specifications for public advertising, with the work to be performed by bridge contractors. Some replacement projects have been accomplished with transportation departments working jointly with the girder manufacturer, on a negotiated agreement basis. Some departments of transportation have established agreements with girder manufacturers by which they are able to achieve early delivery of replacement girders. This procedure is normally used only when conditions dictate immediate action, and removal operations would be performed during the same time as girder replacement manufacture. State agencies have developed satisfactory methods of replacing girders. Therefore, guidelines in this report are limited to those that may improve present practices or lead to consideration of other replacement techniques.

Most Common Replacement Method

The most common method of replacing a damaged girder is to remove portions of the roadway slab and diaphragms, allow-

ing the girder to be lifted up and out of the structure. Sufficient length of reinforcing steel is required to be left in place, extending from the slab and diaphragms, to provide lap requirements. Prior to concrete removal, saw cuts are made in the slab to obtain good break lines in sound concrete. Although some agencies make the cut line at the centerline of the girder that is left in place, most agencies cut the slab at the quarter point of the slab away from the girder that is left in place. The authors have a preference for the latter location because lower stresses should be expected at this point. Removal at this location does not involve the girder stirrups. When damage requires that removal be accomplished in two stages, Iowa DOT designs falsework bents to facilitate traffic movement, as shown in Figure 31.

Other Replacement Methods

Minnesota. The Minnesota DOT has replaced damaged girders by removal and replacement from underneath the roadway slab. They have used this technique for fascia and interior girders. Figures 32 and 33 show details for accomplishing the work. Referring to Figure 32, two steel beams supported the damaged beam and overhanging curb and rail section. The highway under this structure was closed to traffic approximately 45 min for placement of the new girder. Referring to Figure 33, the new girder required end modification for placement. Conventional falsework was placed on the curb side of the damaged girder. A transverse steel beam on the bridge deck, and post supports at diaphragms, supported the damaged beam during removal. Repairs were completed in 12 working days. Replacement of damaged girders by this method results in less concrete replacement and, with the exception of a minimum amount of bottom transverse slab steel, leaves deck steel in place. To restore composite action, steel plates were cast into the top flange and shear studs were field welded. Additional details and photographs are available in Shanafelt and Horn (12).

Florida. A fascia girder was replaced with a precast section consisting of the girder, roadway slab, and curb being cast prior to erection. An 18-in. gap was provided for the cast-in-place closure pour. Camber requirements were carefully controlled. This innovative method of replacement was used for speed of repairs and thus reduced inconvenience to users of the facility.

Worthy Considerations

During this research the authors became aware of a number of worthy considerations that may apply to girder replacements. Plans and specifications should make reference to traffic requirements, protection to the structure, safety considerations, and location of concrete removal in the performance of work. Specific details may be included or parameters may be provided that are to be followed by the contractor and approved by the agency. Consider including the applicable original construction plans for information. Consider detailing bracing requirements for new girders during erection and construction. The use of conventional falsework is generally required (see Fig. 31). The falsework should be required to be designed and constructed to safely carry the imposed loads, and where not specifically detailed on the plans, the contractor's plans should be subject to approval by the agency. Plans and specifications generally include provisions for protection to the public from the work.

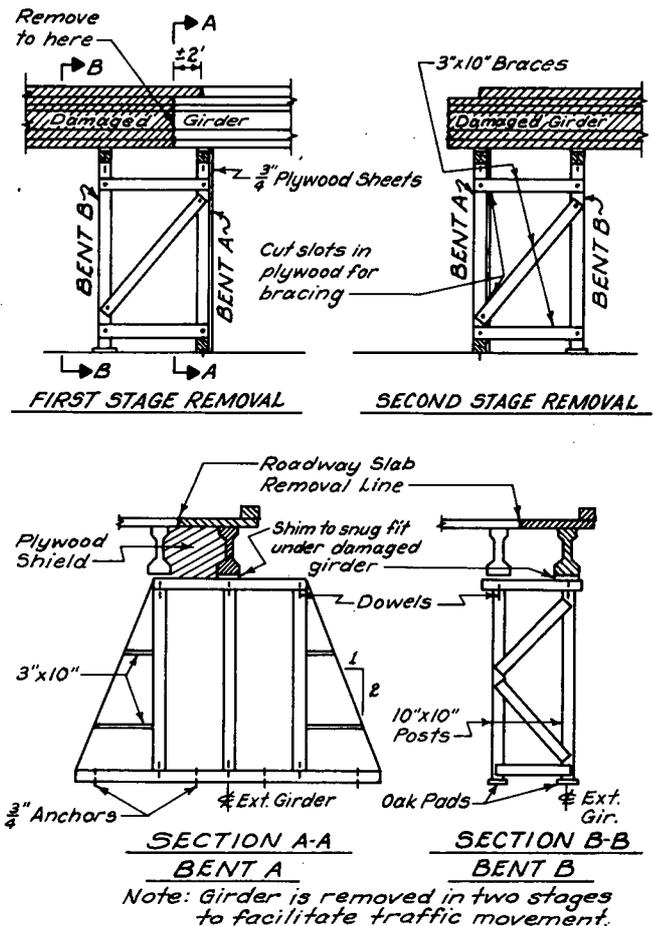


Figure 31. Staged falsework to facilitate traffic. (Courtesy of Iowa DOT)

These provisions include netting, tarpaulins, and wood and metal coverings to ensure that debris will not come in contact with any traveled way. Temporary closure of the highway below should be required during removal and replacement of the girders. Extreme care should be exercised in removal of the girder because of the high internal forces. Explosives must not be used in the removal process. Falsework support should be required prior to cutting slab when girder is damaged beyond load-carrying capacity. Girders should be aged 4 weeks prior to placing roadway slab to prevent undue differential camber (and stress) between new and existing girders. Consider closure of traffic on the structure until slab concrete has attained a strength of 2,000 psi, then restrict traffic to a distance of 15 ft until a strength of 4,000 psi has been attained.

Fire Damage

Damage caused by fire occurs on rare occasions. The assessment and repair of fire-damaged areas should be similar to collision-damaged areas. Fire damage and repair have been discussed in Shanafelt and Horn (12). The probable need for preloading is worth repeating. When a patched area is not preloaded, the tensile stress in the patch will be equal to the

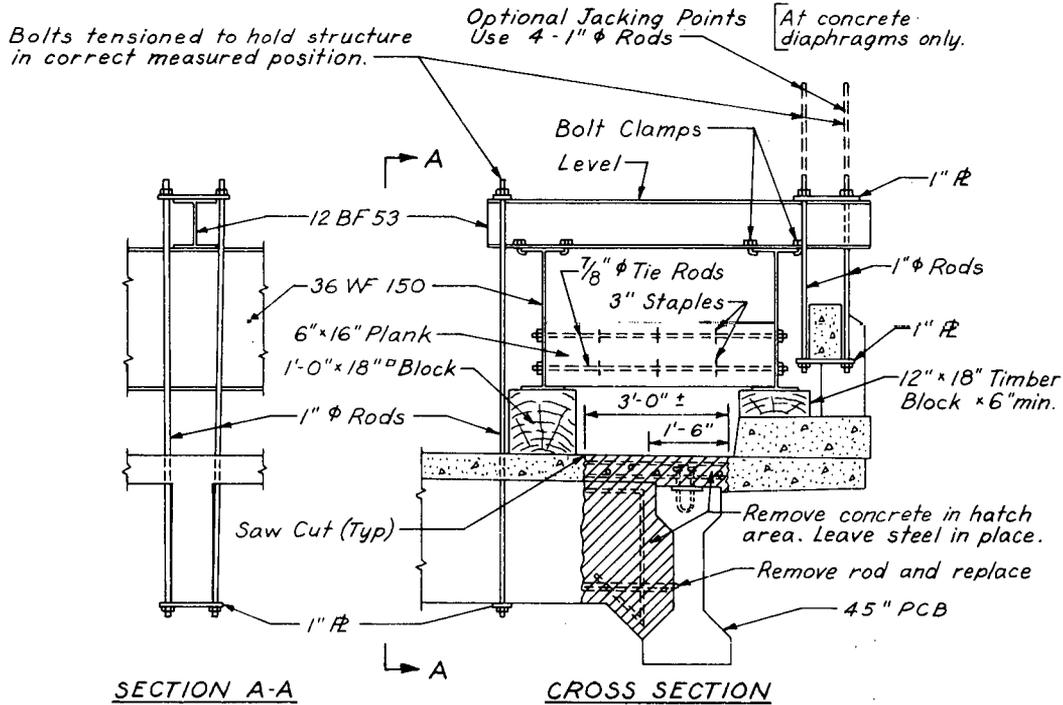


Figure 32. Support system for removing fascia girder from below. (Courtesy of Minnesota DOT)

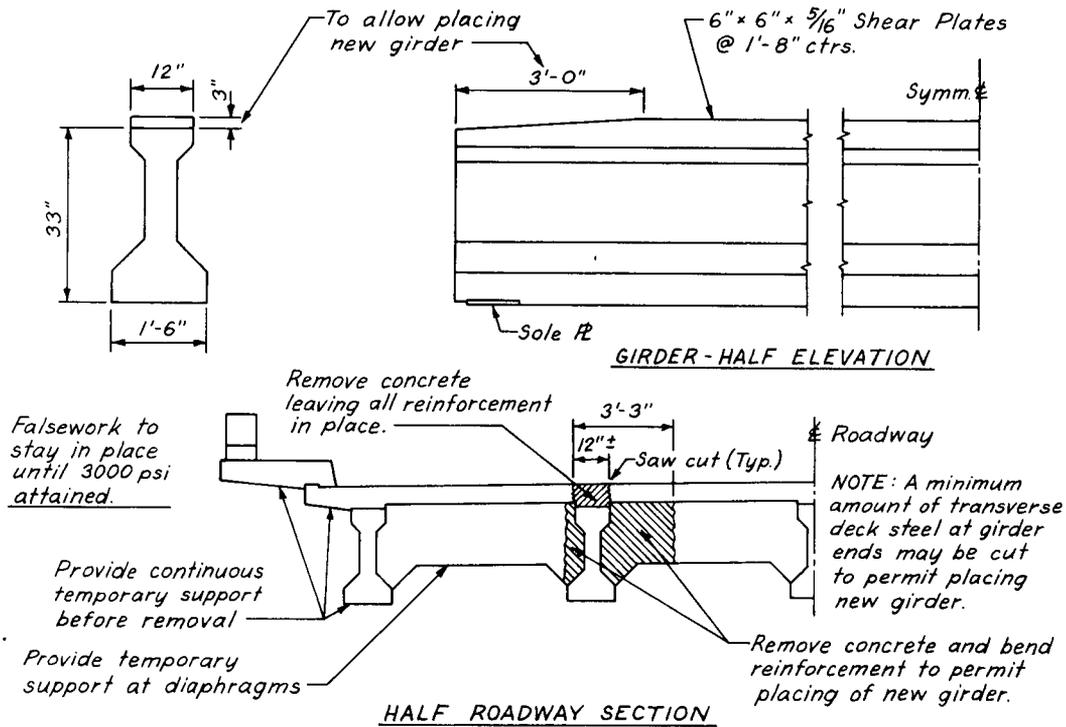


Figure 33. Removing interior girder from below. (Courtesy of Minnesota DOT)

full live load plus impact stress. This tensile stress may be high enough to cause cracking and thus reduce durability. The need for preload should be established by a stress calculation. Additional reinforcement should be installed, as shown in Figure 34, where significant amount of new concrete is required. Bonding agents and epoxy injection should be used to improve durability.

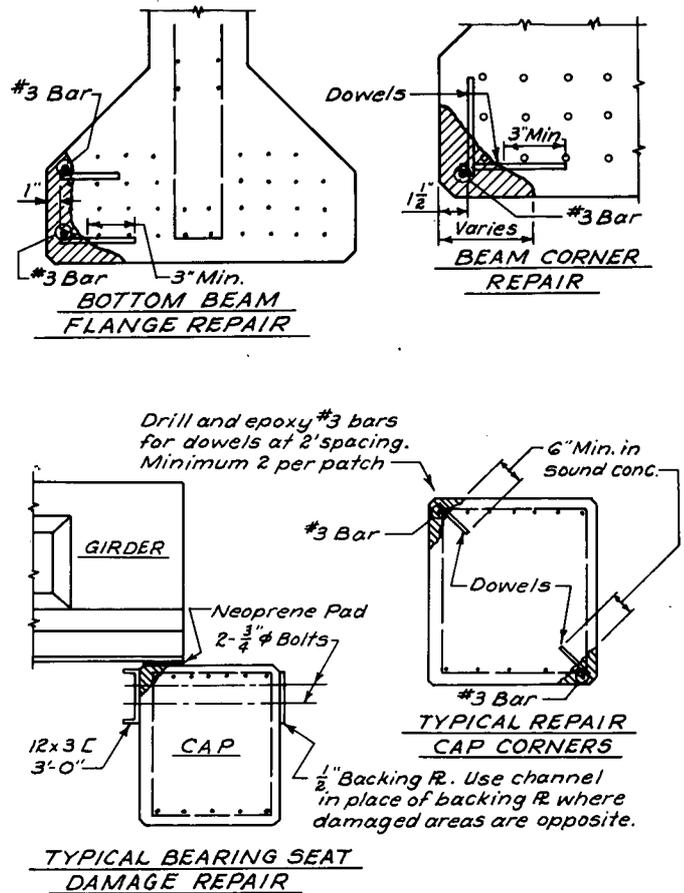


Figure 34. Repair of fire damage. (Courtesy of Texas DOT)

APPENDIX A

LOAD TESTS

SUMMARY

The full-scale tests have been described under tasks 2 and 3 in the "Background" discussion of Chapter One. All full-scale load tests were successful. Loading cycles for each test were repeated until corroboration was attained for successive cycles. Tests were also compared to previous similar tests. Test data were verified with calculated data; the methodology used is shown for Tests 1, 5, 8, and 9. The girder became a cracked section near centerline of span during the first cycle of Test 1. Dead load redistribution did occur and the entire load, dead

load plus test load, was carried by the cracked section. The full-scale load test results are summarized in Tables A-1 through A-6 as follows:

1. Table A-1 shows the applied test load for each test. Figure A-1 shows the girder elevation.
2. Table 1 in the "Commentary" (Chapter One) shows the ratio of the applied test load moment to the AASHTO specification live load moment.
3. Table A-2 shows the ratio of measured deflections due to the maximum test loads to the computed deflections from crack

Table A-1. Applied test loads.

	Load Test Number									
	1	2	3	4	5	6	7	8	9	10
	16 Strands (Cir. as cast)	16 Strands plus 2 150K Thread Bars	16 Strands	12 Strands (4 severed)	12 Strands plus 4 Internal splices	Test No. 5 plus 2 150K Thread Bars	12 Strands (4 severed)	Test No. 7 plus 2 150K Thread Bars	10 strands plus sleeve splice	10 Strands plus sleeve splice
Partial ⁽¹⁾ Cycle	30.5	31.9		19.8	30.5			30.5	26.7	
1st Cycle Cracking Load	55.1				49.7			49.7		
1st Cycle Maximum	90.4	114.7	85.7	57.9	83.7	113.4	57.9	89.7	92.0	
2nd Cycle ⁽²⁾ Maximum	90.4				82.7			89.7	92.0	
Last Cycle Maximum	90.4	114.7	85.4	57.9	82.3	112.7	57.9	89.7	92.0	166.8

Loads are in kips.

(1) A partial "working in" cycle was used for all new tests.

(2) Three cycles were used for all tests that had concrete patching prior to testing in order to obtain corroboration between the 2nd and last (3rd) cycles.

Table A-2. Ratio of measured deflections to computed deflections.

	Test Numbers									
	1	2	3	4	5	6	7	8	9	10
Measured Deflection (in.)	1.18	1.18	1.13	0.78	1.06	1.15	0.76	0.89	0.64	5.28
Computed Deflection (in.)	1.01	0.96	0.90	0.65	0.76	0.93	Δ	0.72	0.73	*
Ratio	1.17	1.23	1.26	1.20	1.39	1.24		1.24	0.88	

Last Cycle, all tests.

Δ Crack widths not representative due to bursting of concrete at severed internal strand splices.

* No attempt made to calculate deflections. Residual deflection after testing was 1.48 in.

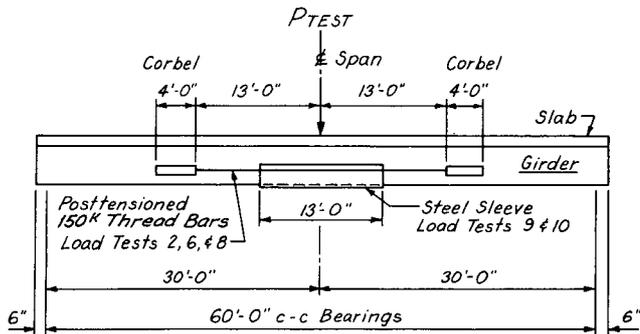


Figure A-1. Girder elevation showing corbels and sleeve.

widths plus elastic deflections. The computed deflections are approximate. They were used during testing to assure that potentiometer readings were giving reasonable results.

4. Table A-3 shows the maximum crack width for each test. Cracks were measured in millimeters with a 10X scope.

5. Table A-4 shows the ratio of measured moments developed from strain data to applied moments, at centerline of span. The report explains why these ratios are approximate.

6. Table A-5 gives the governing factor or factors for each test; an asterisk denotes the factor. For example, the governing factor for Test 1 was 0.75 times the ultimate moment. The governing factor for Test 2 was maximum deflection not to exceed Test 1 deflection.

7. Table A-6 shows the maximum steel stress at centerline of span for each test. Maximum stress in strands was developed from three methods: (1) setting horizontal forces equal to zero at centerline of span, (2) from strand strain gage 110, and (3) from strand strain gage 111. Stresses in the 150 K bars and in

Table A-3. Maximum crack widths.

	Test Number									
	1	2	3	4	5	6	7	8	9	10
Crack Widths in mm.	0.5	0.4	0.5	0.6	0.7	0.7	*	0.6	0.4	6.4
Crack Widths in inches.	0.020	0.016	0.020	0.024	0.028	0.028	*	0.024	0.016	0.252
	16 strand Gir. as cast	16 strands plus 2 thread bars	16 strands	12 Strands (4 severed)	12 strands plus 4 internal splices	Test No. 5 plus 2 thread bars	12 strands (4 severed)	Test No. 7 plus 2 thread bars	10 strands plus sleeve splice	10 strands plus sleeve splice

*Crack width not representative due to bursting of concrete at severed internal strand splice.

the steel sleeve were developed from strain gages attached to the bars and sleeve.

Load tests of splice components were accomplished as described under task 2 in the "Background" discussion of Chapter One and under "Component Testing" in this appendix. The results are summarized as follows:

1. A very successful single-strand splice was developed. This splice could be used to splice any number of strands provided the strand was accessible for torquing. Normally strands are damaged from the exterior strands inward to the interior strands. The splices would be installed working from interior strands toward exterior strands. A broken interior strand surrounded by unbroken strands would not have enough access to permit torquing. Test results are given in Table A-7, in "Component Testing." Strand tension versus torque is shown in Figure 21, and the installed splice is shown in Figures 20 and 22 in Chapter Two.

2. A successful two-strand splice was developed. This splice could be used for splicing any number of pairs of strands provided each pair was accessible for torquing. This splice required the use of a 1,000-ft-lb torque wrench to attain 49,400 lb of tension in a pair of strands. This tension is approximately equal to the working load. A 500-ft-lb torque wrench attained a tension of 40,750 lb. This splice could be used where clearances would mandate the use of the two-strand splice. Test results are given in Table A-8. Strand tension in a pair of strands versus torque is shown in Figure A-11, in "Component Testing." Two-strand splice details are shown in Figures 24 and 25, in Chapter Two.

3. Load tests were performed on three concrete corbels attached to the girder, with differing methods of attachment. Two corbels were proven to be satisfactory. Details and test results are given under "Concrete Corbels."

TEST GIRDER DESIGN

The test girder has been described under task 3 in the "Background" discussion of Chapter One. The girder was designed in accordance with AASHTO specifications. Sixteen straight ½-in. diameter, 270 K strands were required for the 60-ft span,

Table A-4. Ratio of measured moments to applied moments at centerline span.

	First Load Cycle		Last Load Cycle	
	(1)	(2)	(1)	(2)
Test No. 1	1.00	1.00	1.05	1.05
Test No. 2	0.96	0.93	0.97	0.92
Test No. 3	1.10	1.06	1.11	1.07
Test No. 4	1.15	1.10	1.16	1.09
Test No. 5	1.11	1.05	1.10	1.03
Test No. 6	1.04	0.98	1.05	0.98
Test No. 7	1.28	1.20	1.26	1.16
Test No. 8	1.14	1.02	1.14	1.03
Test No. 9	0.98	0.88	1.00	0.88
Test No. 10 (one cycle only)	1.12	1.05		

Measured Moments have been developed from the recorded strains at ¼ span.

(1) Using maximum strains, initial strains in top slab are not modified for creep.

(2) Same as (1) except initial strains in slab are modified for creep.

$$\text{Applied Moments} = \frac{P_{\text{test}}(60')}{4} + 554 \text{ kip ft. (DLM).}$$

Ratio should be = 1.0

Ratios = Measured M/Applied M

Type III AASHTO girder. Mild steel reinforcing was required at the top of the girder near the span ends. No cracks were observable at any location in the girder prior to load testing. The girder was supported on neoprene bearing pads. Figure A-2 shows the composite girder section. Figure A-3 shows test girder stress diagrams. Figures A-4 through A-7 show girder details. The following design calculations are included:

1. Given:

a. Section properties of test girder:

- A = 559.5 in.²
- I = 125,400 in.⁴
- S bottom = 6,190 in.³
- S top = 5,072 in.³

Table A-5. Governing factors.

	Load Test Number									
	1	2	3	4	5	6	7	8	9	10
Calculated Ult. Moment	2511	3242	2511	1887	2511	3242	1887	2511	2511	
0.75 Calc. Ult. Moment (1)	1883*	2432	1883*	1415*	1883*	2432	1415*	1883*	1883*	
Test Moment (2)	1910	2275	1840	1423	1809	2255	1423	1900	1934	3056
Ratio (2)/(1)	1.01	0.94	0.98	1.01	0.96	0.93	1.01	1.01	1.03	
Maximum Test Deflection	1.18	1.18*	1.15	0.79	1.10	1.16*	0.79	0.89	0.64	5.28
Ratio of Test 1 Deflection		1.00	0.97	0.67	0.93	0.98	0.67	0.75	0.54	4.47
0.9 Fy (3) 150K Thread Bars		114.8				114.8*		114.8*		
Test Stress(4) 150K Thread Bars		112.0				113.5		115.5		
Ratio (4)/(3)		0.98				0.99		1.01		
Steel Sleeve Calc. Stress									11.1	
Test - Sleeve Stress									14.8	31.3
Calc. Ult. Moment at End of Sleeve										2511*
Applied Moment at End of Sleeve										2488

* Governing Factor or Factors

(2) Moments include dead load moment

Moments in Kip ft., Deflections in in., Stresses in ksi.

Table A-6. Maximum steel stresses at centerline span (ksi).

	Test No.1	Test No.2	Test No. 3	Test No. 4	Test No. 5	Test No.6	Test No. 7	Test No.8	Test No.9	Test No.10.
	Cycle	Cycle	Cycle	Cycle	Cycle	Cycle	Cycle	Cycle	Cycle	Cycle
	First Last	First Last	First Last	First Last	First Last	First Last	First Last	First Last	First Last	One Cycle Only
Strands H = 0 (1)	209 216	179 179	218 218	235 237	217 212	195 195	263 259	234 235	159 154	193
% of f's f's = 270	77% 80%	66% 66%	81% 81%	87% 88%	80% 79%	72% 73%	97% 96%	87% 87%	59% 57%	71%
Strands H = 0 (2)	209 216	172 166	210 210	224 223	205 197	182 179	246 239	202 203	123 111	155
% of f's f's = 270	77% 80%	64% 61%	78% 78%	83% 83%	76% 73%	67% 66%	91% 89%	75% 75%	46% 41%	57%
Strands Gage 110	198 200	194 194	196 196	200 201	193 192	192 192	199 201	199 199	Strain Gage "Over Range"	
% of f's f's = 270	73% 74%	72% 72%	73% 73%	74% 74%	72% 71%	71% 71%	74% 74%	74% 74%		
Strands Gage 111	207 209	202 203	205 205	211 213	201 200	200 200	212 212	209 209	170 170	181
% of f's f's = 270	77% 77%	75% 75%	76% 76%	78% 79%	74% 74%	74% 74%	79% 79%	77% 77%	63% 63%	67%
Thread Bars		(3) (3) 112 112				(3) (3) 114 114		(3) (3) 115.5 115.5		
% of Fy Fy = 127.5		(4) (4) 88% 88%				(4) (4) 89% 89%		(4) (4) 91% 91%		
Steel Sleeve									14.2 14.0	31.3
% of Fy Fy = 36									39% 41%	87%

(1) Inelastic strains not deducted in top slab. Based on summation of forces = zero (for comparison).

(2) Same as (1) except creep portion of strains deducted in top slab. Generally closer to 75% f's than (1).

(3) 5.5 ksi bending stress added.

(4) Percentage should not exceed 90%.

b. Section properties of girder and slab:

- A = 1,027 in.²
- I = 326,600 in.⁴
- S bottom = 9,895 in.³
- S top of girder = $\frac{326,600}{12} = 27,217$ in.³
- S top of slab = $\frac{326,600}{18.5} = 17,654$ in.³

c. Dead load:

- Use 155 lb/cu ft including steel.
- DL girder + DL slab = 602 + 630 = 1,232 lb/ft.
- DL moment = $\frac{(1.23)(60^2)}{8} = 554$ ft-kip.
- f_t bottom = (554)(12)/6,190 = 1,074 psi (t) at centerline span.

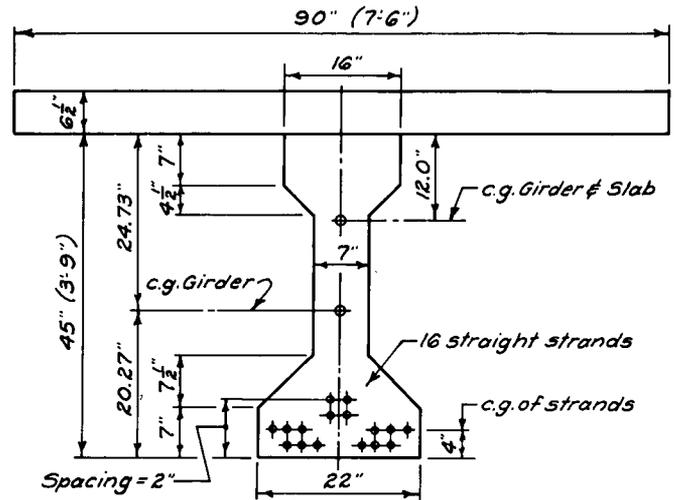
2. Calculate maximum stresses:

a. Stresses from DL and HS-20 LL + I:

- DL girder moment = $\frac{(0.602)(60)^2}{8} = 270.9 \text{ ft-kip.}$
- f_t bottom of girder = $\frac{(270.9)(12)}{6,190} = 525 \text{ psi (t).}$
- f_c top of girder = $\frac{(270.9)(12)}{5,072} = 641 \text{ psi (c).}$
- DL slab moment = $\frac{(0.630)(60)^2}{8} = 283.5 \text{ ft-kip.}$
- f_t bottom of girder = $\frac{(283.5)(12)}{6,190} = 550 \text{ psi (t).}$
- f_c top of girder = $\frac{(283.5)(12)}{5,072} = 671 \text{ psi (c).}$
- HS-20 moment = 697 ft-kip (girder spacing (S) = 7 ft 6 in., distance = S/5.5).
- f_t bottom of girder = $\frac{(697)(12)}{9,895} = 845 \text{ psi (t).}$
- f_c top of girder = $\frac{(697)(12)}{27,217} = 307 \text{ psi (c).}$
- f_c top of slab = $\frac{(697)(12)}{17,654} = 474 \text{ psi (c).}$

b. Strands tensioned to 0.7 f'_s , $A_s = (0.7)(270)(0.153) = 28.9 \text{ kips:}$

- Total DL + LL stress = 1,074 + 845 = 1,919 psi (t).
- Assume strand loss = 45,000 psi.



Span Length = 60 ft.
 f'_c girder = 5000 psi
 f'_c L girder = 4000 psi
 f'_c slab = 4000 psi
 $\frac{1}{2}$ " ϕ 270K Strands, $A_s = 0.153/\text{strand}$
 E_c slab assumed to be 0.8 E_c girder
 MDL Girder = 270.9K'
 MDL Slab = 283.5K' } 554.4
 M HS 20 LL+I = 697K'

TYPE III AASHTO GIRDER

Figure A-2. Test girder design.

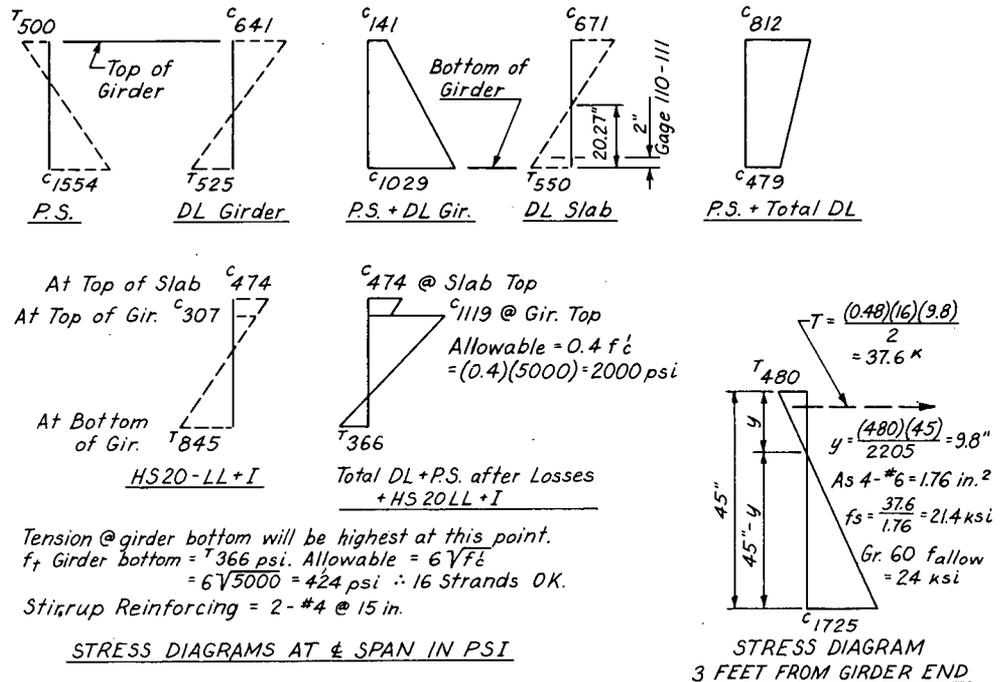


Figure A-3. Test girder stress diagrams.

- Working stress of strands (after all losses) = $(0.7)(270,000) - 45,000 = 144,000$ psi.
- Working load = $(144)(0.153) = 22$ kips per strand.
- f_c bottom prestress after losses = $P/A + M/S_{bot} = 1,554$ psi (c).
- f_t top prestress after losses = $P/A - M/S_{top} = 500$ psi (t).

3. Check section at 3 ft from end of girder (see Fig. A-3). Tension at top of girder and compression at bottom of girder at release of prestress will be highest at this point (assuming strands developed in 3 ft).

- a. Deduct losses due to elastic shortening and steel relaxation only:

- $f_s = (0.7)(f'_s) - 21.7$ ksi = 167 ksi.
- $P/\text{strand} = (167)(0.153) = 25.6$ kips.
- f prestress top = $P/A - M/S = (16)(25.6)/559.5 - (16.27)(409.6)/5,072 = 582$ psi (t).
- f prestress bottom = $P/A + M/S = 732 + (16.27)(409.6)/6,190 = 1,809$ psi (c).

- b. Girder extends 6 in. past bearing centerline:

- Dead load moment 3 ft from girder end = 43.2 ft-kip.
- f_c DL top of girder = $(43.2)(12)/5,072 = 102$ psi (c).
- f_c DL bottom of girder = $(43.2)(12)/6,190 = 84$ psi (t).
- f_t total top = 582 tension - 102 compression = 480 psi (t).
- f_c total bottom = 1,809 compression - 84 tension = 1,725 psi (c).

- c. Check allowable stresses with reinforcing steel:

- f_t at girder top = $7.5 \sqrt{f'_{ci}} = 7.5 \sqrt{4,000} = 474$ psi \approx 480 psi (t).
- f_c at girder bottom = $0.6 f'_{ci} = (0.6)(4,000) = 2,400$ psi.

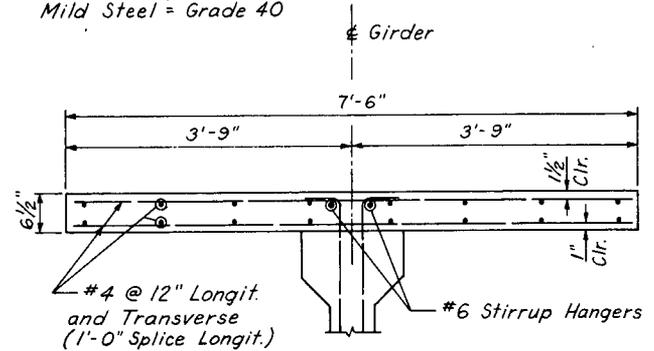
4. Determine length of two No. 6 bars:

- a. At 10 ft from end of girder:
- R girder DL = 18.36 kips.
 - $M = (18.36)(10 - 0.5) - (0.602)(10)^2/2 = 144$ ft-kip.
 - f DL top = $(144)(12)/5,072 = 341$ psi (c).
 - f prestress top = 582 (t).
 - f total = 582 - 341 = 241 psi (t) (190 psi allowable).
- b. Therefore run two No. 6 bars full length.
- c. Add two additional No. 6 bars, 10 ft long at each end.

5. Review prestress losses:

- a. Total loss for prestress strands assumed to be 45,000 psi.
- b. From AASHTO (12th Edition), Article 1.6.7: $\Delta f_s = SH + ES + CR_c + CR_s$, in which: Δf_s = total prestress loss excluding friction, SH = shrinkage loss, ES = elastic shortening loss, CR_c = creep of concrete in psi, and CR_s = steel relaxation loss.
- SH = 17,000 - 150 RH, in which RH equals mean ambient relative humidity in percent (see Fig. 1.6.7 in AASHTO specifications).
 - SH = 17,000 - (150)(80) = 5,000 psi.
 - ES = $(E_s/E_{ci})(f_{cir})$, in which: $E_s = 28,000,000$ psi, $E_{ci} = E$ concrete at transfer of prestress, $E_{ci} = 33 w^{3/2} \sqrt{f'_{ci}} = (33)(155)^{3/2} \sqrt{4,000} = 4,028,000$ psi, and f_{cir} computed on basis of $0.63 f'_s = 170$ ksi.

$f'_c = 4000$ psi
Mild Steel = Grade 40



SECTION THRU SLAB

Figure A-7. Girder section through slab.

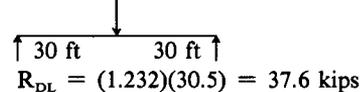
- $f_{cir} = P/A + M_y/I - f_{DL}$ [$P = (16)(0.153)(170) = 416$ kips].
- $f_{cir} = 416/559.5 + (416)(20.27 - 4)(16.27)/125,400 - f_{DL}$
= 744 (c) + 878 (c) - M_y/I (DL girder)
= 1,622 - $(270.9)(12)(16.27)/125,400 = 1,200$ psi (c).
- $E_s = (28,000)(1,200 (c))/4,028 = 8,340$ psi.
- $CR_c = (12)(f_{cir}) - (7)(f_{cds})$, in which: $f_{cds} = M_y/I, M = (0.630)(60)^2/8 = 283.5$ ft-kip, and $f_{cds} = (283.5)(12)(16.27)/125,400 = 441$ psi.
- $CR_c = (12)(1,200) - (7)(441) = 11,310$ psi.
- $CR_s = 20,000 - 0.4 ES - 0.2 (SH + CR_c)$
= 20,000 - (0.4)(8,340) - (0.2)(5,000 + 11,310) = 13,400 psi.
- $\Delta f_s = 5,000 + 8,340 + 11,310 + 13,400 = 38,050$ psi.
- The nominal figure of 45,000 psi was used for design.
- Losses at release = ES + $CR_s = 8,340 + 13,400 = 21,740$ psi.

6. Calculate ultimate strength capacity of girder:

- a. Design f'_c roadway slab = 4,000 psi.
- b. $d = 51.5 - 4.0 = 47.5$ in.
- $p^* = A_s^*/bd = (16)(0.153)/(90)(47.5) = 0.000573$.
 - $f_{su}^* = f'_s [1.0 - (0.5)(p^*)(f'_s/f'_c)]$
= $(270)[1.0 - (0.5)(0.000573)(270/4)] = 265$ ksi.
 - $M_u = A_s^* f_{su}^* d [1 - (0.6)(p^*)(f_{su}^*)/f'_c]$, $\phi = 1.0$
= $(2.45)(265)(47.5) [1 - (0.6)(0.000573)(265)/(4)]/12$.
 - $M_u = 2,511$ ft-kip (f'_c roadway slab = 4,000 psi).
- c. Ultimate moment capacity required per AASHTO:
- $M_u = (1.3)(DL + 5/3 HS-20 LL + I)$.
 - $M_u = (1.3)(554 + 5/3)(697) = 2,230$ ft-kip.
 - Test load to produce cracking ≈ 56 kips.

7. Design neoprene bearing pads:

- a. 89 kips Test 1 (Other loads assumed to be overloads)



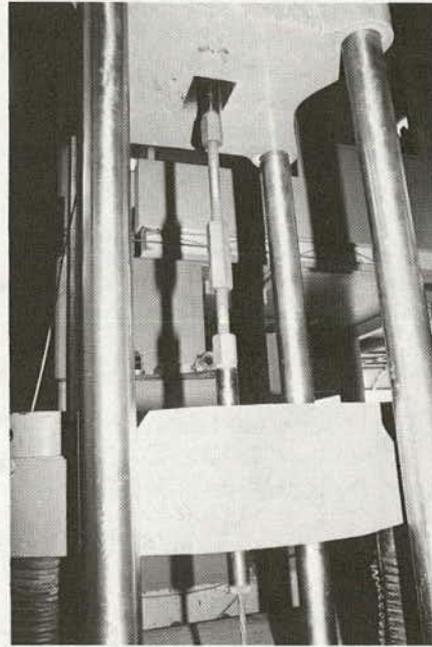
$$R \text{ Test 1} = 89/2 = \frac{44.5}{82.1} \text{ kips}$$

- b. Use 21-in. wide pad for stability.
 - c. Use $\frac{3}{4}$ -in. thickness.
 - d. Use 7-in. length (5 t is specified minimum).
 - e. Durometer hardness = 60
 - f. DL bearing pressure = $37.6/(21)(7) = 256$ psi (allowed bearing pressure = 500 psi).
 - g. DL plus test load pressure = $82.1/(21)(7) = 558$ psi (allowed bearing pressure = 800 psi).
 - h. Check deflection:
 - Shape factor $s = (L)(W)/(2t)(L + W) = (21)(7)/(2)(0.75)(21 + 7) = 3.5$.
 - $\Delta = 7$ percent t (allowable = 7 percent t).
 - $\Delta \text{ Test 1} = (0.07)(0.75) = 0.0525$ in. ($\frac{1}{16}$ in.).
 - $R \text{ Test 2} = 37.6 + 115/2 = 95.1$ kips.
 - DL + test load pressure = $95.1/(21)(7) = 647$ psi.
 - $\Delta = 8$ percent $t = (0.08)(0.75) = 0.06$ in. ($\frac{1}{16}$ in.).
 - $R \text{ Test 10} = 37.6 + 130/2 = 102.6$ kips.
 - DL + test load pressure = $102.6/(21)(7) = 698$ psi.
 - $\Delta = 8.5$ percent $t = (0.085)(0.75) = 0.06375$ in. ($\frac{1}{16}$ in.).
8. Slab Design:
- a. Span length (s) = (7 ft. 6 in.) - (1 ft 4 in.) = 6 ft 2 in.
 - b. $M = [(s + 2)/32] 16,000 = (8.17/32)(16,000) = 4,085$ ft-lb.
 - c. Assume continuous, $M = (0.8)(4,085) = 3,269$ ft-lb.
 - d. $d = 6\frac{1}{2}$ in. - $1\frac{1}{2}$ in. - $\frac{3}{8}$ in. = 4.63 in.
 - $A_s = M/f_s j d = (3,268)(12)/(20,000)(0.875)(4.63) = 0.48$ in.² Would require No. 5 at $7\frac{1}{2}$ -in. centers.
 - Distribution steel = $100/\sqrt{S} = 100/\sqrt{6.17} = 40.3$ percent.
 - $A_s = (0.48)(0.403) = 0.19$ in.²/ft. Use No. 4 at 12-in. centers, top and bottom both ways.
9. For corbel design see Corbel 3 in "Component Testing."

COMPONENT TESTING

Single-Strand Internal Splice

Four component load test cycles were conducted on the single-strand splice shown in Figures 19 and 20. This internal splice was used to connect four severed strands of girder Test 5. The parts for this splice were readily obtainable and required ordinary machine shop work to thread the rods and splices. A "Supreme" splice chuck including its threaded coupling was used to connect the $\frac{1}{2}$ in. round strand to each $2\frac{1}{2}$ -in. long steel splice. A 6-in. long steel sleeve was used as a turnbuckle to tension the splice. Threads were lubricated with a spray-on graphite-based lubricant. Tensile loads were recorded from the Tinius-Olsen Dial Gage machine and from a load cell wired to the SY 76 data logger. Torque was applied with a 500-ft-lb, 36-in. torque wrench. A 16-in. long special wrench was used to connect the torque wrench to the turnbuckle sleeve (see Fig. 22). Figure A-8 shows the single-strand splice being tested and the special wrench. In addition to recording torque and tensile load, the number of turnbuckle turns, and dimensions (1) and (2) (see Fig. 19) were recorded. The rod threads were eight per inch. The strand was tested to failure after the last test and



a. Splice Being Tensioned



b. Special Wrench

Figure A-8. Component test of single-strand splice.

broke at the splice chuck at a load of 36.6 kips equal to 239.2 ksi. The results in Table A-7 indicate that the most reliable measure of tension is torque. Figure 21 is a plot of strand tension versus torque using the average of all test data. Figure A-9 shows the strand break and some of the equipment used in this component test.

Two-Strand Internal Splice

Three component test load cycles were conducted on a two-strand splice shown in Figures 23 and 24. This splice could be used to splice pairs of $\frac{1}{2}$ -in. 270 K strands. Several pairs of strands could be spliced in one girder. The parts for this splice were readily obtainable and required ordinary machine shop work. The strands with swage fittings were obtained from VSL Corporation of Los Gatos, California. A single-strand with swage fitting was tested and the strand broke at the strand grips

Table A-7. Single-strand splice test.

Test Number	1	2	3	4
Max. Torque - Wrench - ft. lb.	300	350	350	400
Max. Torque - Total - ft. lb.	433	506	506	578
Max. Tension in strand - lbs.	24,300	26,050	26,300	31,100
Tension lbs. for 433 ft. lb. of Torque	24,300	23,400	23,450	25,200
Tension - lbs. per ft. lb. Torque	56.1	54.0	54.2	58.2
Turns for 433 ft. lbs. Torque	3 2/3	3 1/4	2 2/3	2 1/3*
Tension per Turn - lb.	6,630	7,200	8,790	-
Splice Shortening in inches for 433 ft. lbs. Torque	7/8	13/16	5/16**	9/16
Tension in lbs. per in. of Shortening	27,800	28,800	48,600**	44,800

Strands are $\frac{1}{2}$ inch round 270^K.

Area = 0.153 square inches

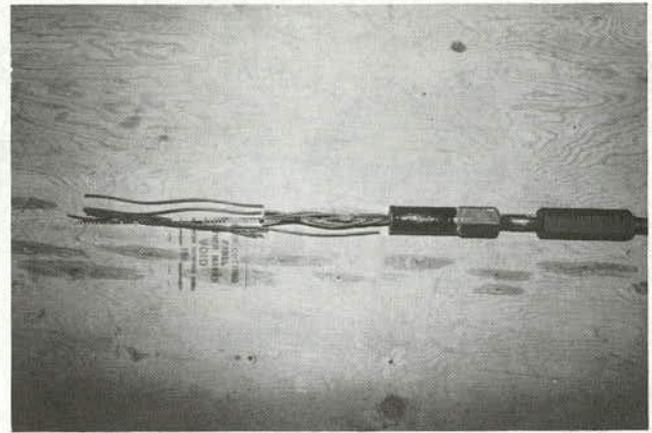
*Turns not applicable - did not loosen chucks in test machine.

**No zero readings. Calculated from dimensions obtained at 8250 lbs. tension.

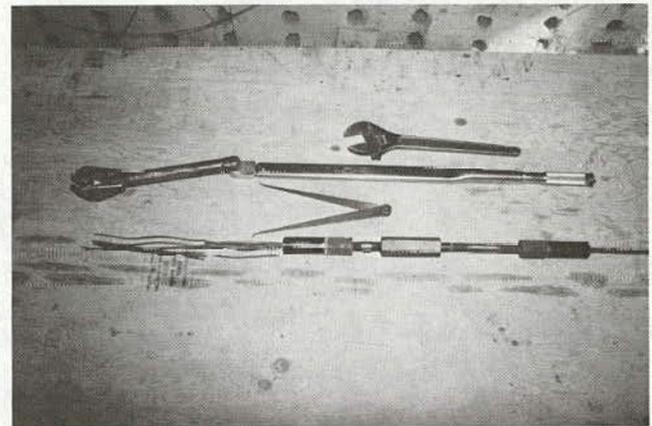
at a load of 40.1 kips, equal to 262 ksi. No evidence of distress occurred at the swage fitting.

A 6-in. long steel sleeve was used as a turnbuckle to tension the two-strand splice. Threads were lubricated with a spray-on graphite-based lubricant. The first test was a preliminary cycle using a 500-ft-lb torque wrench. Torque was then applied with a 1,000 ft-lb, 64-in. torque wrench. The same 16-in. long special wrench used for the single-strand splice was used to connect the torque wrench to the turnbuckle sleeve, shown in Figure 22.

The same test data, as obtained for the single-strand splice, were recorded for the two-strand splice. The splice was tested to failure after the last test cycle, and failed at 81 kips with the load in each strand being 40.5 kips. This load equates to 265 ksi in each strand. Failure mode occurred by breaking one strand near its midpoint after one wire failed in a swage fitting allowing that strand to slip. Figure A-10 shows the two-strand splice being tested and the failure mode. An examination of the test results shown in Table A-8 indicates a large decrease in tension per ft-lb of torque between the first and last cycles. This decrease may have been partially due to increased abrasion of threads. During the second and third cycles the load per strand varied—14 percent deviation for the second cycle at 49.4-kips maximum load, and 29 percent deviation for the third cycle at 30.25-kips maximum load. This deviation caused some misalignment which may have also decreased the tension per ft-lb of torque. Tension in pounds per inch of splice shortening is quite consistent; however, this consistency may be misleading. The amount of seating strands is approximately 49 kips. It is unlikely that a pair of broken strands should be prestressed to a higher load. This load was attained during the second test cycle but not during the third cycle. The recess in one transfer plate became damaged to the point where it would not keep the rod nut from turning. A deeper recess, or use of a square nut, may have prevented



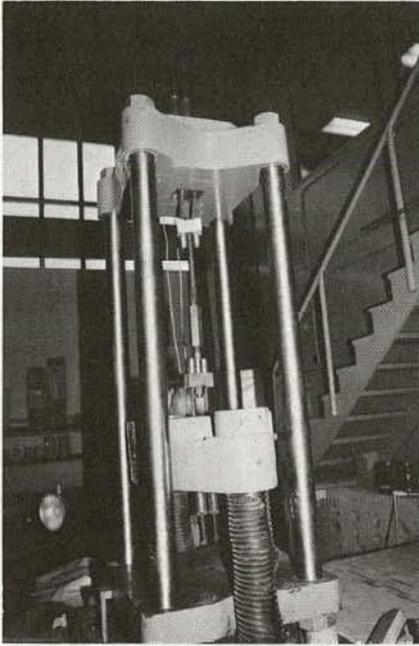
a. Strand Break



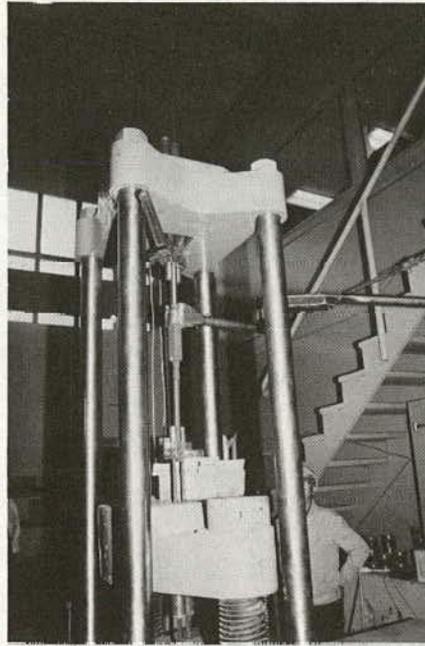
b. Sampling of Equipment Used

Figure A-9. Strand break and equipment for single-strand splice.**Table A-8. Two-strand splice test.**

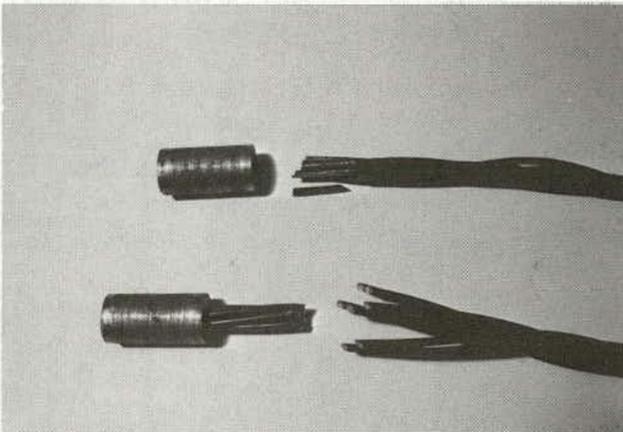
	Test Cycle		
	1st	2nd	3rd
Maximum Torque - Wrench - ft. lb.	500	1000	700
Maximum Torque - Total - ft. lb.	722	1227	859
Maximum Tension in Two Strands - lbs.	40,750	49,400	30,250
Tension - lbs. for 722 ft. lb. of Torque	40,750	34,300	26,700
Tension - lbs. per ft. lb. of Torque	56.4	47.5	37.0
Turns for 722 ft. lb. of Torque	3.50	2.22	2.22
Tension per turn in lbs.	11,640	15,450	12,030
Splice Shortening in inches for 722 lbs. of Torque	0.719	0.584	0.521
Tension in lbs. per inch of Shortening	56,700	58,700	51,200



a. Preliminary Cycle



b. Full Test Cycles



c. Failure Mode

Figure A-10. Component test of two-strand splice.

this occurrence. Figure A-11 is a plot of splice tension versus torque using the second test cycle data. Figure A-12 shows the torque and special wrench used in the second and third cycle tests of the two-strand splice.

Although this splice tested reasonably well, it is believed that the single-strand splice is superior. The test results were more consistent, the physical details of the splice are simple, and ease of obtaining required torque and tension is better. Unless clearances mandate the use of the two-strand splice, the single-strand splice is recommended.

Concrete Corbels

Load tests were made on three concrete corbels attached to the test girder. One corbel was one of four identical corbels used in girder Tests 2, 6, and 8. This corbel is identified as corbel 3 (see Fig. A-13). Two other corbels, 1 and 2 (see Fig. A-14), were cast in place against the girder. The three corbels differed only in the method of attachment to the girder fillet. This area is regarded as being critical because heavily loaded girders have strands near the face of the fillet, and it is difficult to avoid these strands in making attachments. Corbel 1 was attached with twelve $\frac{1}{2}$ -in. diameter, round expansion bolts that penetrated the fillet $2\frac{1}{2}$ in. Corbel 2 was attached with twelve $\frac{1}{2}$ in. diameter, round expansion bolts that penetrated the fillet $1\frac{1}{2}$ in. and the holes were filled with epoxy resin prior to installing the bolts. The face of the fillet was coated with epoxy resin to supplement the strength of the expansion bolts.

The $\frac{1}{2}$ -in. depth expansion bolts without epoxy resin could be loosened easily by hand. It appeared to be impractical to prevent this depth of hole from becoming oversize or out of round. Corbel 3 was attached to the fillet with six Grade 60, No. 4 hairpin reinforcing bars which were inserted into 6 in. deep $\frac{3}{4}$ -in. diameter holes filled with epoxy resin. All corbels were designed to withstand the ultimate strength of a 1-in. diameter 150 Grade Dywidag bar. $P_u = 150 (0.85) = 127.5$ kips.

Strain gages were attached to the top surface of the corbels as shown in Figure A-15. Three potentiometers were used to measure displacements. Figure A-16 shows construction of the corbels and the jacking arrangement for the corbel tests. The ultimate strength of the concrete in corbels 1 and 2 was 7,040 psi during load testing. The ultimate strength of the concrete in corbel 3 was 8,710 psi during load testing. Load was applied by jacking a 100-ton ram placed between corbels. Corbels 1 and 3 were subjected to a partial load cycle with maximum load of

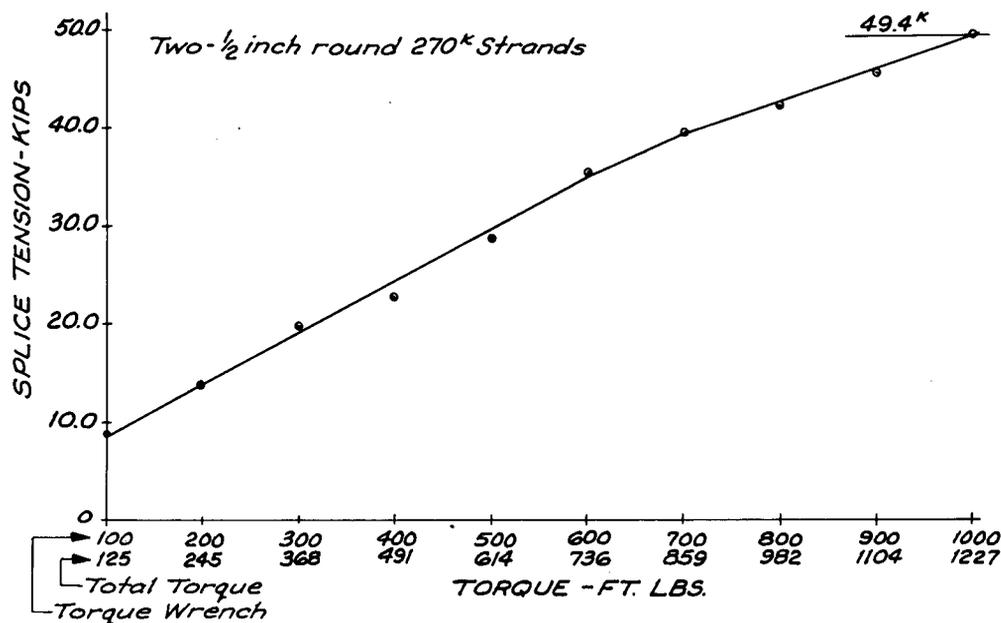


Figure A-11. Two-strand splice tension versus torque.

36.9 kips, three load cycles with a maximum load of 127.8 kips, and a last (fourth) load cycle with a maximum load of 186 kips. Corbel 2 was subjected to the same loading with the exception of two intermediate load cycles to a maximum load of 127.8 kips. The approximate capacity of the 100-ton ram used was 186 kips. The partial cycle was used as a working-in cycle and to check uniform bearing at the corbel faces.

Corbel 1 developed an observable crack of 0.1 mm at the fillet interface when the load reached 98 kips (equaling $0.77 f'_c$ or $0.9 f_y$). This crack opened to 0.2 mm at 127.8 kips, as observed by a 10X lens. The crack remained the same width at 186 kips. This load exceeded the ultimate design load of 127.5 kips by 46 percent. A diagonal crack of 0.1 mm width across an inside corner of the top surface of the corbel occurred at 128 kips. There was no observable change in this crack width at higher load. A second diagonal crack of 0.1 mm width developed on the top surface at 186 kips (see Fig. A-15). Potentiometer 171 indicated a longitudinal displacement of 0.002 in. during the first two load cycles at the ultimate strength design load of 127.8 kips. This displacement reduced to 0.001 in. when the load was reduced to 8.55 kips. During the third load cycle the displacement was 0.004 in. at load of 127.8 kips. This displacement returned to 0.001 in. when the load was removed. During the last (fourth) load cycle, the displacement was 0.006 in. at load of 186 kips. This displacement also returned to 0.001 in. when the load was removed. Corbel 1 is believed to have ample strength to anchor a 1-in. diameter Grade 150 K thread bar (ASTM-A722).

Corbel 2 developed an observable crack of 0.1 mm at the fillet interface when the load reached 75 kips (equaling $0.59 f'_c$ or $0.69 f_y$). This crack opened to 0.2 mm at 127.8 kips, and opened to 0.3 mm at 186 kips. A diagonal crack of 0.1 mm width across an inside corner of the top surface of the corbel occurred at a load of 99 kips (equal to $0.91 f_y$). This crack extended along the top corner of the corbel (see Fig. A-15) at

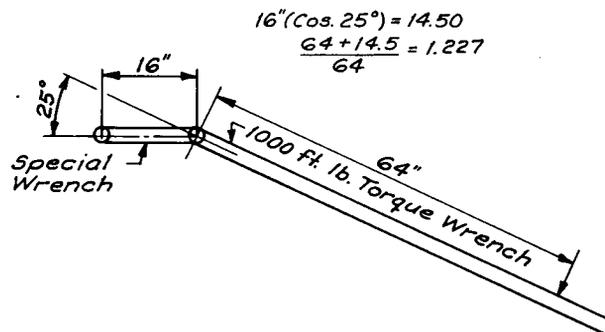


Figure A-12. Torque and special wrench for two-strand splice.

a load of 120 kips. A crack developed at the bottom interface at a load of 145 kips. Potentiometer 172 indicated a longitudinal displacement of 0.010 in. during the first load cycle; at ultimate strength design load of 127.8 kips this displacement reduced to 0.005 in. when the load was reduced to 8.55 kips. During the second cycle the displacement was 0.007 in. at load of 127.8 kips. This displacement reduced to 0.005 in. when the load was reduced to 8.55 kips. During the last (third) load cycle, the displacement was 0.010 at load of 127.8 kips. The displacement increased to 0.019 in. at load of 186 kips. This displacement reduced to 0.002 in. when the entire load was removed. Because of cracking at relatively low loads and size of cracks and displacements at high loads, corbel 2 is not considered adequate to anchor a 1-in. diameter Grade 150 K thread bar.

Corbel 3 developed an observable crack of 0.1 mm at the fillet interface when the load reached 98 kips (equaling $0.77 f'_c$ or $0.9 f_y$). This crack remained the same width at 127.8 kips; however, it increased to 0.2 mm at 186 kips. No other cracks were observed to occur in this corbel. Potentiometer 170 indi-

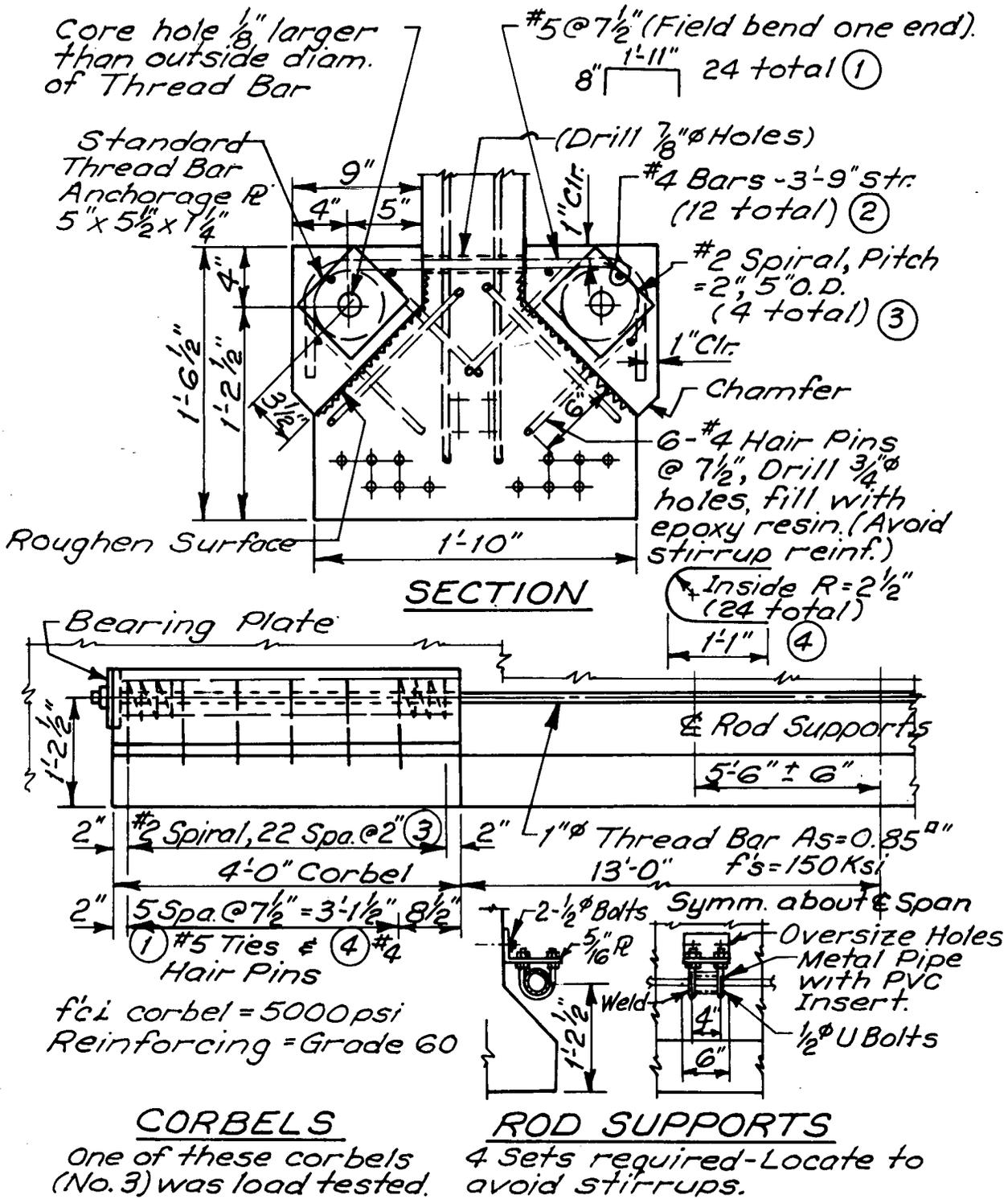
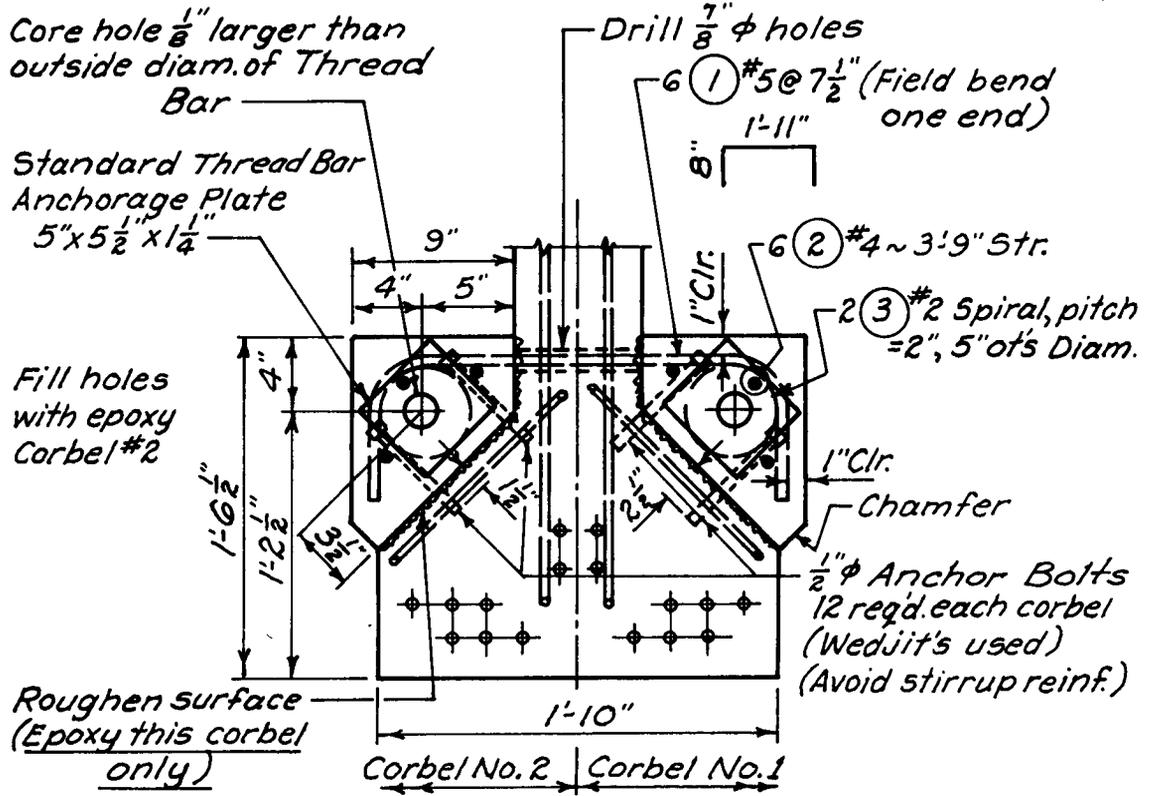


Figure A-13. Corbel 3 details for load test Nos. 2, 6, and 8.



SECTION

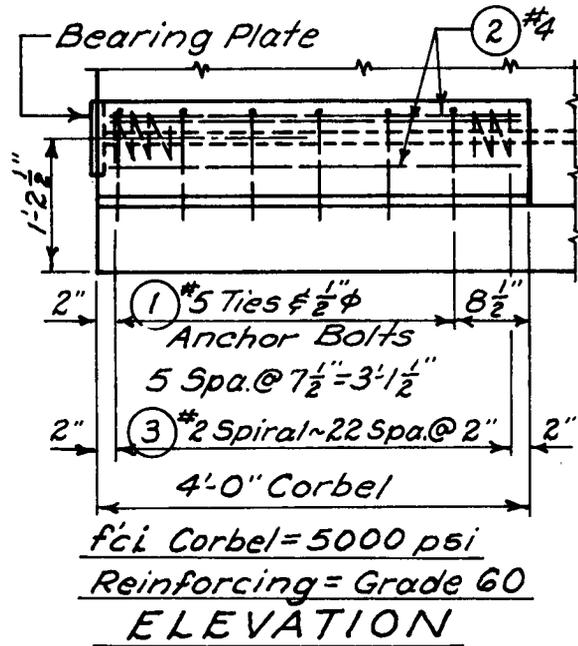


Figure A-14. Component tests corbel 1 and 2.

CORBEL TESTS 1, 2 & 3

9 - 60mm Strain Gages
2 - 2" Pots
1 - 6" Pot

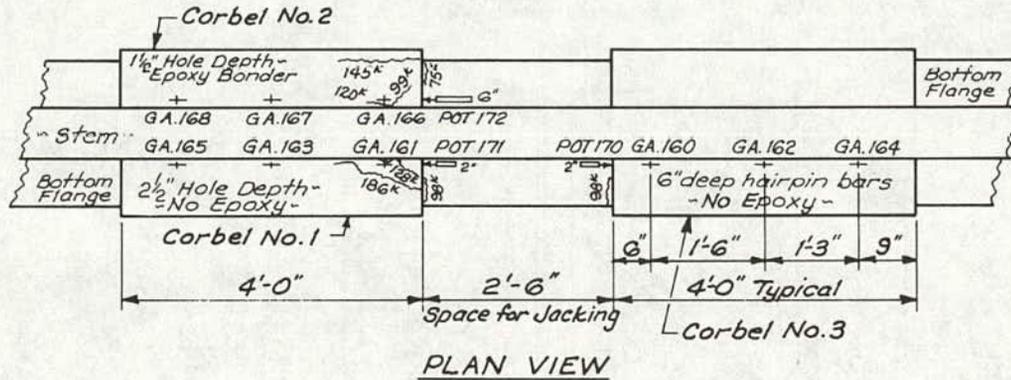
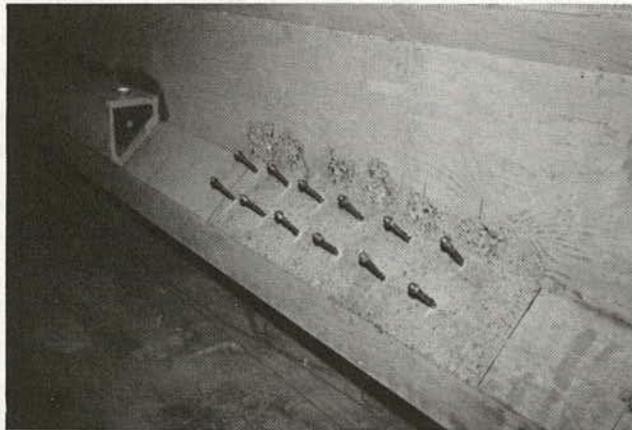
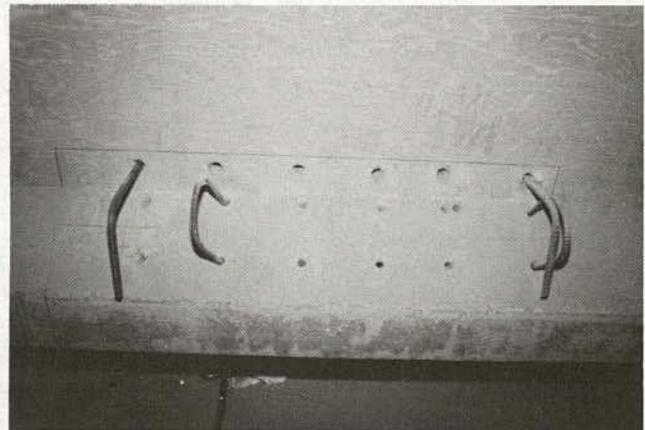


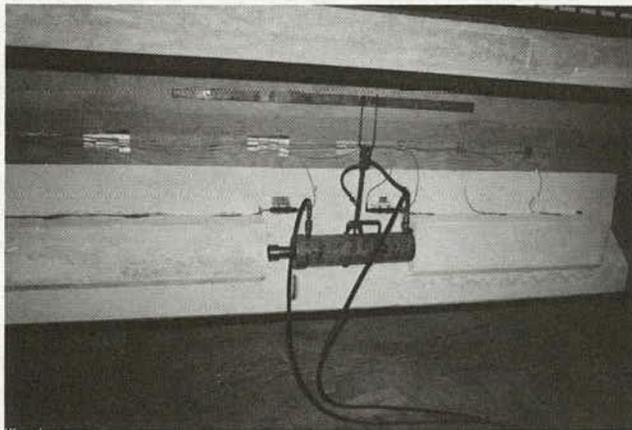
Figure A-15. Strain gage locations for corbel test Nos. 1, 2, and 3.



a. Test Nos. 1 and 2



b. Test No. 3



c. Jacking Arrangement

Figure A-16. Component corbel tests.

ated maximum longitudinal displacements that varied from 0.005 in. to 0.007 in. during the first three load cycles at ultimate strength design load of 127.8 kips. These displacements reduced to 0.001 in. when the load was reduced to 8.55 kips. During the last (fourth) load cycle, the displacement was 0.006 in. at a load of 186 kips. This displacement also returned to 0.001 in. when the entire load was removed. Corbel 3 is believed to have ample strength to anchor a 1-in. diameter Grade 150 K thread bar.

A study of the strains along the top face of the corbels is of interest. Figure A-17 is a plot of load-strain for corbel 1, last load cycle up to 186 kips maximum load. Note that the strain indicated by gage 161 stays at approximately 75 to 80 percent of the total strain up to a load of 127.8 kips, indicating that most of the load transfer occurs within a short distance from the face of the corbel. At a load of 128 kips a diagonal crack appeared across the top inside face of the corbel and another

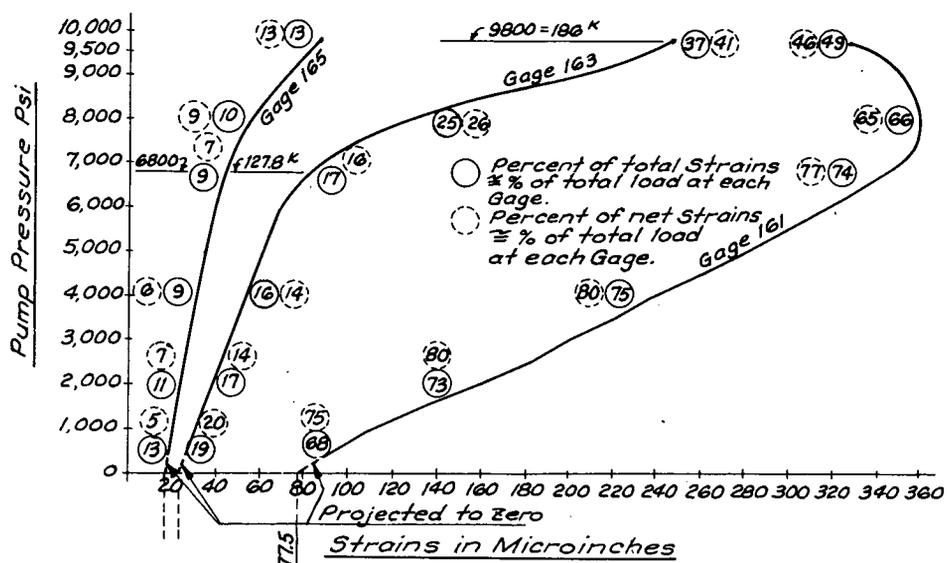


Figure A-17. Corbel No. 1 strains—last cycle.

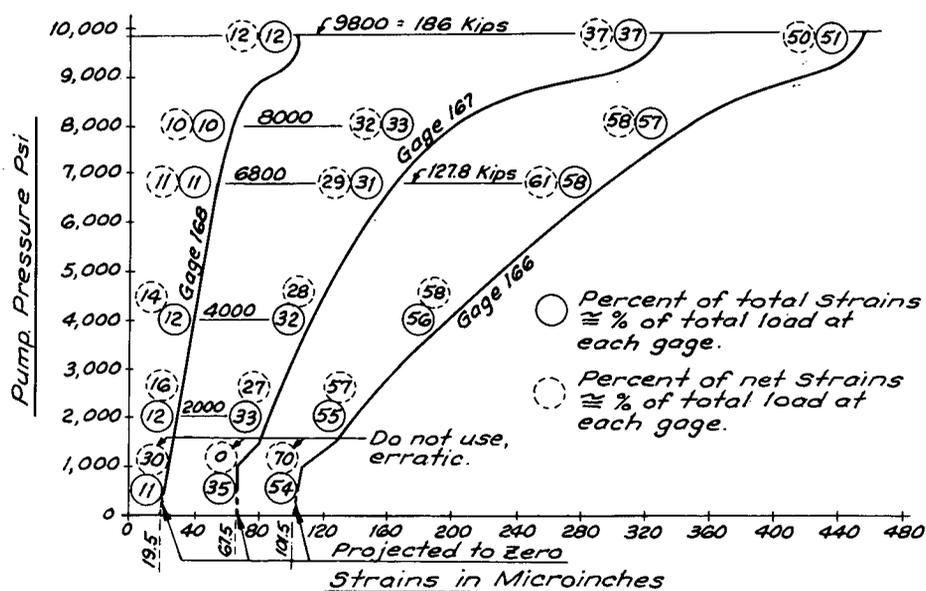


Figure A-18. Corbel No. 2 strains—last cycle.

diagonal crack appeared at 186 kips. The displacement at the face of the corbel increased from 0.004 in. at 127.8 kips to 0.006 in. at 186 kips. At 186 kips of load the amount of load transfer reduced to approximately 46 to 49 percent near the face of the corbel. Load redistribution occurred between 127.8 kips and 186 kips. Figure A-18 is a plot of load-strain for corbel 2. The percentage of load transfer stays approximately constant at 50 to 61 percent during the load cycle up to 186-kips maximum load. This deviation from corbel 1 may be partially due to the fact that the load-strain curves for gages 166 and 167 are curved lines throughout, and appear to show that more longitudinal distribution was required to withstand the test loads. This ob-

servance was confirmed by the larger longitudinal displacements that occurred at the face of corbel 2 during test loading. Because of the erratic behavior of the gage near the face of corbel 3 during the last loading cycle, it was not possible to draw an identical curve for corbel 3. However, Figure A-19 shows that approximately 81 percent of the total load was transferred near the face of the corbel for the third cycle. The longitudinal displacements and crack widths indicate that corbel 3 behavior was similar to corbel 1. Corbel 2 did not have as much strength as corbels 1 and 3, and it appears that the load-strain curves give confirmation. Corbel test results are given in Table A-9.

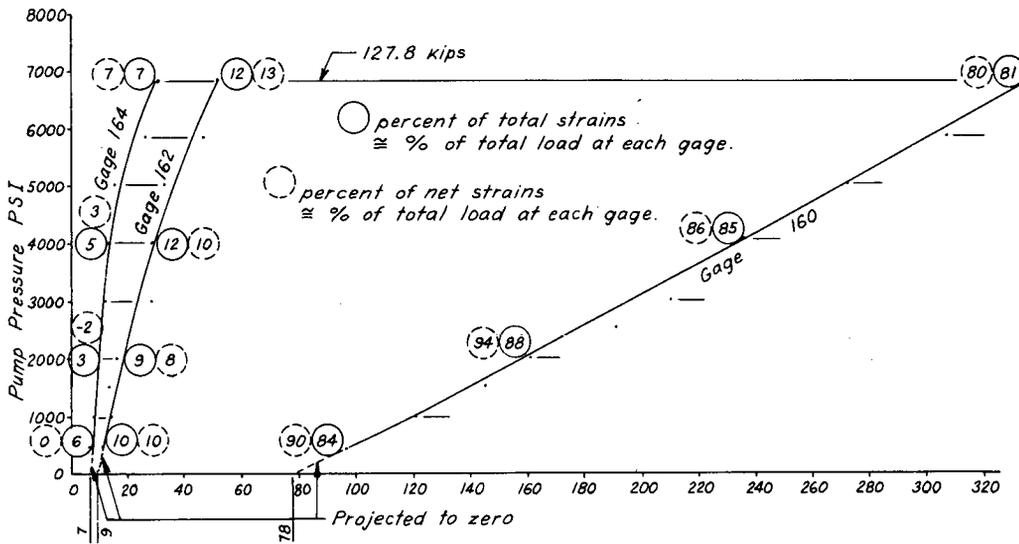


Figure A-19. Corbel No. 3 strains—third cycle.

Table A-9. Corbel test results.

	Corbel Number		
	1	2	3
Load at first observed crack	98 kips	75 kips	98 kips
Crack width	0.004 in. (0.1 mm)	0.004 in. (0.1 mm)	0.004 in. (0.1 mm)
Crack width at 127.8 kips	0.008 in. (0.2 mm)	0.008 in. (0.2 mm)	0.004 in. (0.1 mm)
Crack width at 186 kips	0.008 in. (0.2 mm)	0.012 in. (0.3 mm)	0.008 in. (0.2 mm)
Max. Longitudinal displacement at 127.8 kips	0.004 in. (0.1 mm)	0.010 in. (0.25 mm)	0.007 in. (0.18 mm)
Max. Longitudinal displacement at 186 kips	0.006 in. (0.15 mm)	0.019 in. (0.48 mm)	0.006 in. (0.15 mm)
Longitudinal displacement after load removal	0.001 in. (0.025 mm)	0.002 in. (0.051 mm)	0.001 in. (0.025 mm)

Girder Corbel Calculations

1. Given

- 4 required (SE corbel tested as corbel 3).
- 1-in. diameter, 150 K bar, $A_s = 0.85 \text{ in.}^2$, $F_u = 150 \text{ ksi}$.
- $P_u/\text{bar} = (0.85)(150) = 127.5 \text{ kips}$.

2. Calculate requirements of shear-friction reinforcement

- $A_{vf} = V_u/f_s \mu \phi = 127.5/40 (0.85)(1.0) = 3.75 \text{ in.}^2$
- Six No. 5 bars, $A_v = 6(0.31) = 1.86 \text{ in.}^2$, twelve No. 4 bars = 2.4 in.^2
- Total $A_v = 4.26 \text{ in.}^2$ Use Grade 60 reinforcement to supply additional tensile force across interface (see AASHTO Article 1.5.35D).
- V allowed = $4.26 (40)(0.85)(1.0) = 144.8 \text{ kips}$.
- Maximum allowable test load $\leq 0.75 f'_s \leq 0.9 f_y$.
- $0.75 (150)(0.85) = 95.6 \text{ kips}$, F.S. = $144.8/95.6 = 1.51$.

- $0.9 (150)(0.85)(0.85) = 97.5 \text{ kips F.S.} = 144.8/97.5 = 1.49$.

- Shear along interface = $127.5 \text{ kips}/14(48) = 190 \text{ psi}$.
- Allowable = 350 psi (AASHTO Article 1.5.35 E).

- Development length No. 4 bars = $\frac{0.04 A_b f_y}{\sqrt{f'_c}} = \frac{(0.04)(0.20)(60,000)}{\sqrt{5,000}} = 6.79$.

- Minimum development length = $\frac{3.75 - 1.86}{2.4} (6.79) = \frac{(1.89)}{(2.4)} (6.79) = 5.35 (6 \text{ in., O.K.})$.

3. Check bearing on plate and spiral:

- Bearing on plate = $127.5/(5)(5.5) = 4,636 \text{ psi}$ (O.K. for confined concrete).
- 5-in. diameter spiral; $A = \frac{\pi(5)^2}{4} = 19.63 \text{ in.}^2$
- Bearing on spiral = $127.5/19.63 = 6,496 \text{ psi}$ (O.K. for confined concrete).

4. Check No. 4 hairpin pull-out from concrete:

- Tension $P_u = (A_{conc.})(\phi)(3) \sqrt{f'_c}$.
- Note that corbels have been designed for the full value of P_u , not $0.95 P_u$ as permitted by AASHTO Article 1.6.17.
- Diagonal tension $P_u = (A_{conc.})(\phi)(4) \sqrt{f'_c}$.
- Diagonal distance from end of hairpin to flange corner = 7 in.
- Spacing of hairpins = $7\frac{1}{2} \text{ in.}$
- Distance from centerline of hairpin to edge = 3 in.
- $A_{conc.}$ for tension $P_u = (7.5)(3) = 22.5 \text{ in.}^2$
- $A_{conc.}$ for diagonal tension $P_u = (7)(7.5) = 52.5 \text{ in.}^2$
- Total $P_u = (22.5)(0.85)(3) \sqrt{5,000} + (52.5)(0.85)(4) \sqrt{5,000} = 4.1 + 12.6 = 16.7 \text{ kips}$.

- P_u allowed for two No. 4 hairpins (Grade 60) = $(2)(0.2)(60,000) = 24$ kips.
- The design of the corbels used for strengthening was considered to be conservative. The three full-scale load tests and the component test for corbel 3 confirmed the adequacy of this corbel.

Component Corbel 1 Calculations

1. Given

- 1-in. ϕ 150 K thread bar, $A_s = 0.85$ in.², $F_u = 150$ ksi.
- P_u bar = $A_s F_u = (0.85)(150) = 127.5$ kips.

2. Calculate shear carried by No. 5 bars and Wedjits:

- Transverse No. 5 bars (Grade 60) = $A_s F_s \mu = (6)(0.31)(43)(1.0) = 80.0$
- Carried by Wedjits = $127.5 - 80 = 47.5$ kips.
- Note: $f_s = 43$ ksi determined by trial.

3. Determine additional tension from moment = M/d , using two No. 5 bars only:

- $M/d = (80 \text{ kips})(5.0 \text{ in.}) / (48 \text{ in.} - 2 \text{ in.} - 7.5 \text{ in.} / 2 - 3 \text{ in.}) = 10.19$ kips.
- $10.19 / A_{\text{effective}} = 10.19 / (2)(0.31) = 16.44$ ksi.
- Maximum stress in No. 5 bars = $43 + 16.44 = 59.44$ ksi < 60 ksi.

4. Determine additional tension from moment = M/d , using four Wedjits only.

- $\frac{1}{2}$ -in. Wedjits, $2\frac{1}{2}$ -in. embedment, $P_u = 5,155$ lb.
- $M/d = (47.5 \text{ kips})(3.5 \text{ in.}) / 39.25 \text{ in.} = 4.24$ kips.
- Additional tension per Wedjit = $4.24 / 4 = 1.06$ kips.
- Allowable tension per Wedjit = $5.16 - 1.06 = 4.10$ kips.
- No. of Wedjits Required = $47.5 / 4.1 = 11.59$ (used 12).

Component Corbel 2 Calculations

- $\frac{1}{2}$ -in. diameter Wedjits; $\frac{1}{2}$ -in. embedment; pullout = 3,700 lb.
- $\frac{1}{2}$ -in. diameter Wedjits; $2\frac{1}{2}$ -in. embedment; pullout = 5,155 lb.
- Difference will have to be carried by tension in epoxy interface.
- 2 bolts = $(2)(5,155 - 3,700) = 2,910$ lb.
- $2,910 / (7.5 \text{ in.})(1.414)(7.5\text{-in. spacing}) = 36.6$ psi tension.

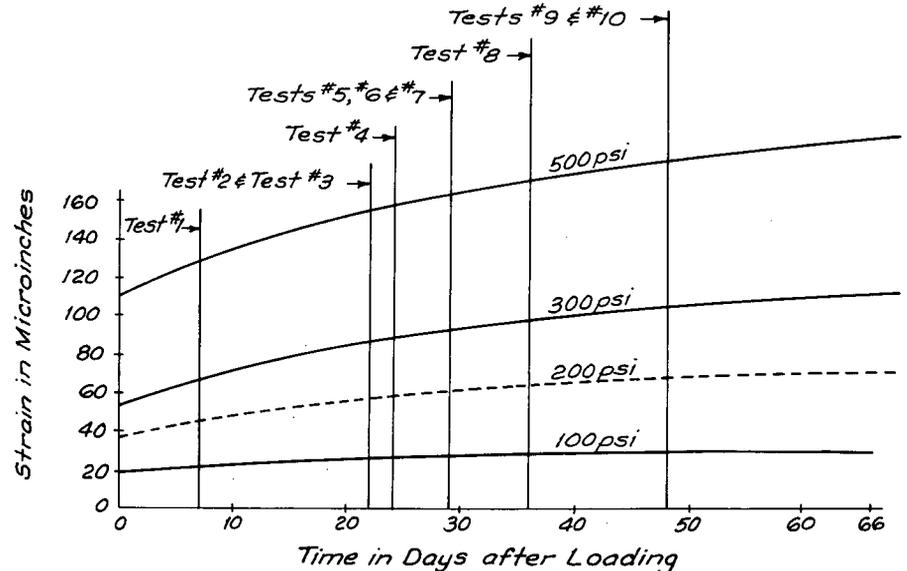
FULL-SCALE LOAD TESTS

Effect of Creep on Slab Strains

As the girder shortens and flexes because of the time-dependent effects of prestress, stresses are induced in the slab. The total strains in the slab were obtained from strain gages attached to the slab. However, a portion of this strain is inelastic. To determine the inelastic strain, creep curves (see Fig. A-20) were developed from information found in Dunham (13). This source was used because strains for low stress were given. All creep curves other than those developed by testing for a specific case must be regarded as approximate. The effect of inelastic strain is given in Table A-4. The creep modification for Test 1 was zero because of the short time in days after measured loading. Calculations for Tests 1, 5, 8, and 9 show the methodology used.

Effect of Strain Gage Accuracy

The investigators believe there is a practical limit to strain gage accuracy. Strain gage locations for the load tests are shown in Figure A-21. All gage readings were in microinches. Gages



Note: Developed from C.W. Dunham, "The Theory and Practice of Reinforced Concrete," McGraw Hill, 1944.

Figure A-20. Assumed creep curves for top slab.

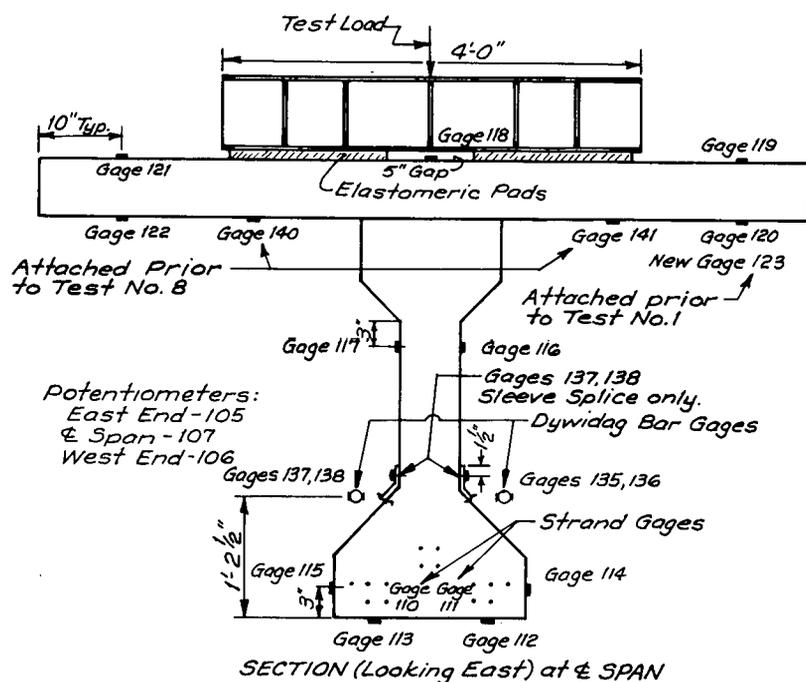


Figure A-21. Strain gage placements.

112 and 113 on the bottom of the steel sleeve, Test 9, were set at 1 and 0 (zero). About 20 min later and still at zero load, they read 1 and 3, indicating a change in strain of 0 and 3. At a test load of 26.73 kips they read 110 and 118, indicating a change in strain of 109 and 118. The deviation was 9 (or 8.26 percent). When the load of 26.73 kips was reduced to zero the gages read 13 and 4, indicating a change in strain of 12 and 4. The deviation was 8, and the percentage of deviation would be large. The elapsed time from first to last readings was approximately 23 min. The gages were spaced 14 in. apart, perpendicular to the girder; therefore, temperature differential could have been a minor contributing factor. At zero load for the first complete load cycle the gages read 29 and 29. At maximum load of 91.97 kips they read 467 and 514, indicating a change of strain of 438 and 485. The deviation was 47 (or 10.73 percent). This deviation occurred in about 7 min and cannot be explained by differential temperature. Gages 135 and 136 were attached on opposite sides of a single thread bar for Test 2. The initial readings for these gages were -20 and -14 . After jacking and seating, the gages read, 3,354 and 3,266, indicating a change in strain of 3,374 and 3,280, and the deviation was 94 (or 2.79 percent). At maximum load for the first complete cycle the gages read 3,710 and 3,617, indicating a change in strain of 3,730 and 3,631, and the deviation was 99 (or 2.65 percent).

It is apparent that gages placed at locations that should show zero deviation do, in fact, deviate. As shown in the calculations for Test 1, the strains in the top slab were converted to forces, and the force in the strands was calculated from ΣH (horizontal forces) = zero. The moment developed from these forces was compared to the applied moment (see Table A-4). The last load cycle shows a ratio of 1.05, while the first load cycle shows a ratio of 1.00, which corroborates the moment developed from the strains. The first load cycle of Test 1 is the only ratio in

the table that gives exact corroboration. However, if the strains in the top slab were 10μ in. less for the last cycle, the ratio would be 1.01 and the stress in the strands 206 ksi, equaling $0.76 f'$. Part of the lack of precise corroboration may be due to normal inaccuracies in strain gage readings.

Unusual Strain Gage Readings

Strain gages 110 and 111 were attached to two bottom strands prior to release of prestress. The gage readings corroborated computed values initially and reached a maximum compression strain 8 days after release of prestress, indicating further loss of prestress. Between 8 and 9 days the gages indicated a gradual reduction in strain which was attributed to the weight of forms being placed for casting the slab. The strains due to pouring the slab 9 days after release of prestress agreed reasonably well with computed values. However, immediately prior to Test 1, 34 days after release, gage 110 was reading 148μ in. of compression and gage 111, 71μ in. of tension. Immediately prior to Test 2, 50 days after release, gage 110 was reading 799μ in. of tension and gage 111, 517μ in. of tension. During Test 5, at maximum test load of 83.7 kips, gage 110 was reading $2,280 \mu$ in. of tension and gage 111, $2,072 \mu$ in. of tension. When these strains were converted to stress, and stresses due to initial pretension minus steel relaxation included, the result was stresses exceeding the yield point of the strands. After review of Test No. 5 strains and stresses it became apparent that gages 110 and 111 were not giving correct total strains. Prior to Test 9, gage 110 was reading "Data over Range." However, the change in strain during any particular test generally gave stresses that, when added to initial prestress minus losses, corroborated stresses developed from other strains. The strand stresses given in Table A-6, developed from gages 110 and 111, should be

regarded as approximate. Gages 110 and 111 were bonded to an individual wire in each strand. They were then insulated to ensure that they would not come in contact with any other steel. They were also wrapped to ensure water tightness. Concrete was then placed around the protected gages. Therefore, these gages might have been affected by both the tension in the strand and the strain experienced by the concrete. Because of the deflection of the span and cracking of the concrete, these gages might be registering, in addition to strand tension, some of the cumulative concrete strain over time. Then it might be logical to use change in strain with original stress plus losses to calculate strand stress for each particular load test. It appears that to avoid this problem, the gages should have been completely isolated from the concrete.

Strain gages 120 and 122 were attached to the bottom of the slab 17 days after casting the slab. Gage 120 was replaced with new gage 123 two days after installing, when gage 120 read "Data over Range." Gages 122 and 123 corroborated each other reasonably well through Test 1. However, immediately prior to Test 2, gage 122 was reading 94 μ in. of tensile strain, and gage 123 was reading 27 μ in. of compression strain. Gage 122 continued to show increasing values of tensile strain. During Test 8, gage 122 showed an initial strain of 123 μ in. of tension and at maximum load 220 μ in. of tensile strain. The 220 μ in. of tensile strain equated to 1,025 psi of tensile stress or $14 \sqrt{f'_c}$, which was unrealistically high. Gage 123 gave realistic values, showing an initial value of 48 μ in. of compression strain which increased slightly during the first 47 kips of the loading cycle and then gradually decreased to 18 μ in. of tensile strain at maximum load. The 18 μ in. of tensile strain equated to 84 psi

of tensile stress (see Fig. A-22). New gages 140 and 141 were attached to the bottom of the slab prior to Test 8 (see Fig. A-21). Load strain data from these gages did not corroborate either gage 122 or gage 123. After reviewing the behavior of gages 122 and 123 and developing moments and stresses at centerline of span, it became apparent that gage 123 was giving reasonable data, and gage 123 data were used in developing moments and stresses at centerline of span for all tests.

All other gages and potentiometers corroborated each other and gave realistic results.

Load Test 1

Load Test 1 was a test of the composite girder as cast with 16 straight strands. The calculated ultimate moment capacity of the composite girder was 2,511 ft-kip. Therefore, 75 percent of the ultimate moment was 1,883 ft-kip, including dead load moment of 554 ft-kip. The calculated maximum test load was 88.6 kips, which was increased slightly to 90.4 kips to facilitate load readings. The total applied moment was 1,910 ft-kip. The test load moment of 1,356 ft-kip (not including DLM) was equivalent to 1.95 times the HS-20 LL + I service load moment of 697 ft-kip. At time of testing the girder concrete strength (f'_c) was 6,030 psi, and top slab strength was 4,435 psi. Test 1 occurred 38 days after casting the girder and 34 days after release of strands. The slab was cast 24 days prior to Test 1. The computed 34-day prestress loss was 28.7 ksi. Figure A-23 shows slab strength versus age. Figure A-24 shows girder strength versus age.

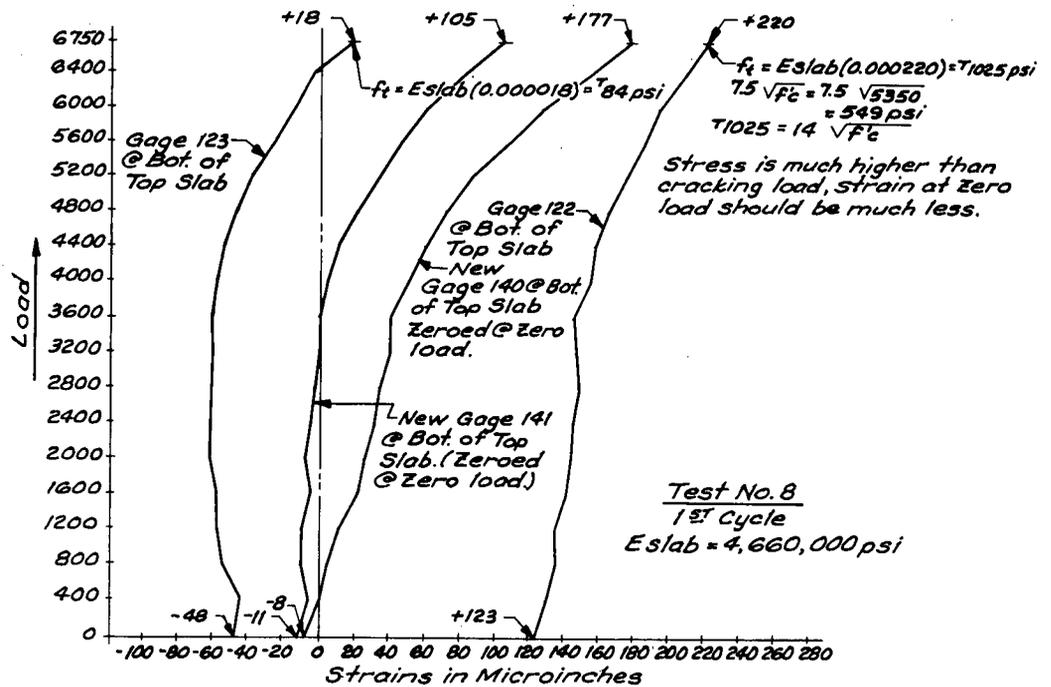


Figure A-22. Gages 122, 123, 140, and 141, load test No. 8.

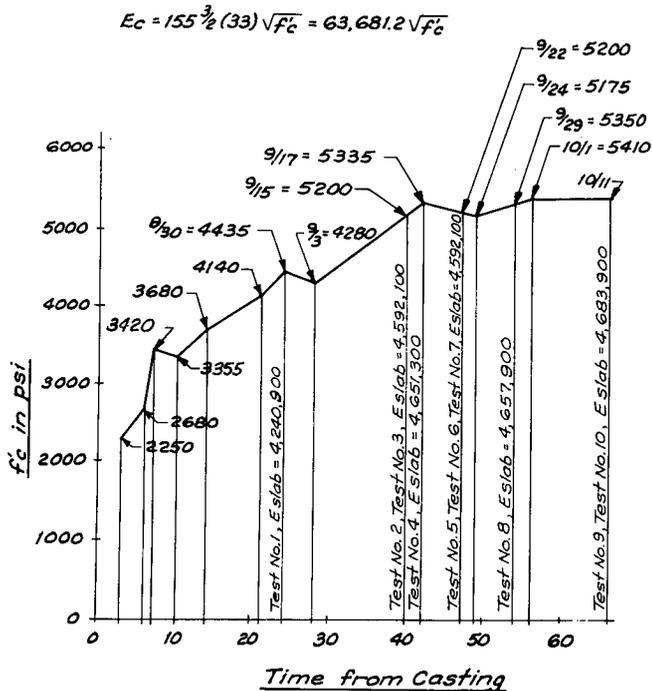


Figure A-23. Slab strength versus age.

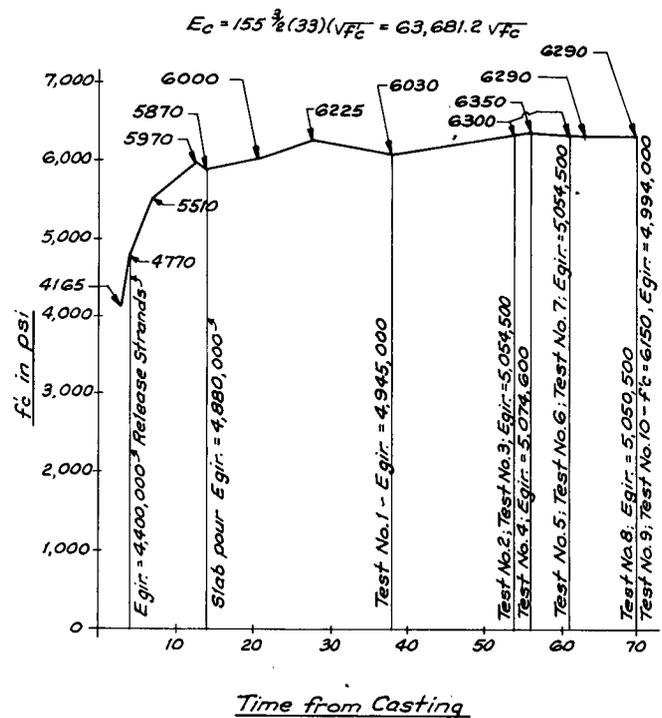


Figure A-24. Girder strength versus age.

The first observed cracks occurred at a test load of 55.1 kips. This load equated to a tensile stress of about 2.7 or 4.5 $\sqrt{f'_c}$. AASHTO specifications give the modulus of rupture as 7.5 $\sqrt{f'_c}$. It should be kept in mind that this was the result of a single test. It was not a primary objective of this test to determine cracking stress. At maximum test load, eight major cracks were observed to extend within a few inches of the bottom of the top slab. The maximum crack width was 0.5 mm. Strain data indicated tension in the bottom of the top slab. The maximum vertical deflection caused by the test was 1.19 in. The computed deflection for the last load cycle was 1.01 in. Maximum moments at centerline of span were developed from strain data, and these moments were compared to the applied moments (see Table A-4). Maximum strand stresses were developed from strain data. Strand stresses ranged from 73 percent to 80 percent of f'_s (see Table A-6).

The simplified method used to calculate deflection based on deflection due to major cracks plus partial elastic deflection is approximate. It is realized that crack widths may not vary linearly from bottom of girder to top of crack. The value of this method was its check within $\frac{1}{4}$ in., or less, of the measured deflections which gave reasonable assurance that the measured deflections were accurate. Test 7 was an exception due to the effect the internal strand splices had on crack widths. The measured deflection for Test 7 compared well with other test loads and deflections. The following calculations show the methodology used (also see Figs. A-25 and A-26 for stress and deflection diagrams):

Test 1 girder as cast with 16 straight strands.

1. Calculate load to give 75 percent of ultimate moment capacity:

- Calculated $M_u = 2,511$ ft-kip.
- 75 percent $M_u = (0.75)(2,511) = 1,883$ ft-kip.
- $(0.75) M_u - \text{DLM} = 1,883 - 554 = 1,329$ ft-kip.
- $P = \frac{4M}{l} = \frac{(4)(1,329)}{60} = 88.6$ kips. Test Load was increased to 90.4 kips to facilitate readings.

2. Cracking load results:

- a. Calculate stress at bottom from P.S. + DL using losses at 34 days after release (see Test Girder Design).

$$\begin{aligned} \bullet \text{Cr}_c (\text{Creep}) &= \frac{t^{0.6}}{10 + t^{0.6}} (11,310) \\ &= \frac{34^{0.6}}{10 + 34^{0.6}} (11,310) \\ &= (0.45)(11,310) = 5,090 \text{ psi.} \end{aligned}$$

- SH (shrinkage) = 5,000 psi.

$$\bullet (e \text{ SH}) t = \frac{t}{55 + t} = \frac{34}{55 + 34} = 0.38.$$

- SH = $(0.38)(5,000) = 1,900$ psi.

- ES (elastic shortening) = 8,340 psi.

- CR_s (steel relaxation loss) = 13,400 psi.

- Total losses for Test 1 = 5,090 + 1,900 + 8,340 + 13,400 = 28,700 psi.

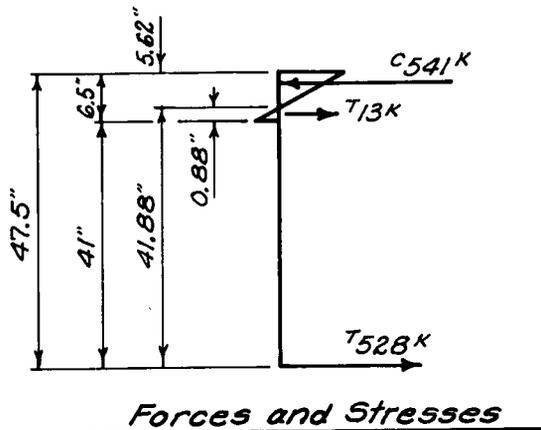
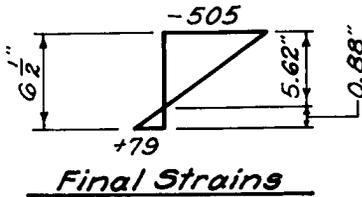
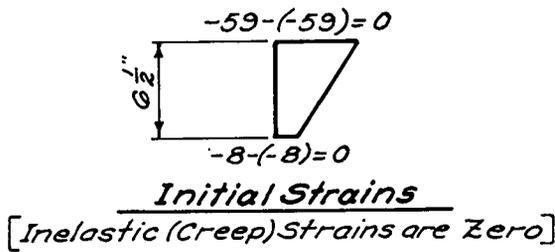
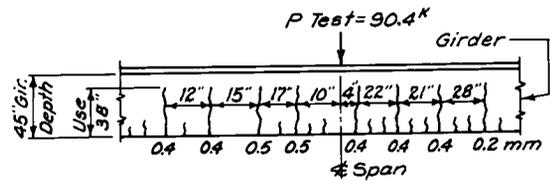


Figure A-25. Test No. 1 stresses due to strains.

- Strand stress = $189 - 28.7 = 160.3$ ksi; $P = (160.3)(0.153) = 24.5$ kips.
 - f bottom P.S. (from Girder Design) = $\frac{24.5}{22} (1,554) = 1,731$ psi (c).
 - f bottom P.S. + DL = $1,731$ compression - 525_{DL} Gir. tension - 550_{DL} slab tension = 656 psi (c).
- b. f bottom calculated from cracking load of 55.1 kips:
- $M = (55.1)(60)/4 = 826.5$ ft-kip; $f = (826.5)(12)/9,895 = 1,002$ psi (t).
 - f bottom at cracking load = $1,002$ tension - 656 compression = 346 psi (t).
 - $f'_c = 6,030$, $X \sqrt{f'_c} = X \sqrt{6,030} = X (77.65)$, thence: $X = 346/77.65 \approx 4.5 \sqrt{f'_c}$ (Approximate tension at 55.1 kips).
- c. Calculate f bottom using load-strain data from Figures A-27 and A-28:
- f bottom at observed cracking load of 55.1 = $(860 + 877)/2 = 869$ psi (t).
 - f bottom = 869 tension - 656 compression = 213 psi (t).
 - $f'_c = 6,030$, $X \sqrt{f'_c} = X \sqrt{6,030} = X (77.65)$, thence: $X = 213/77.65 \approx 2.7 \sqrt{f'_c}$ (Approximate tension at 55.1 kips).



Note: Crack measurements were approximate for Test No. 1.

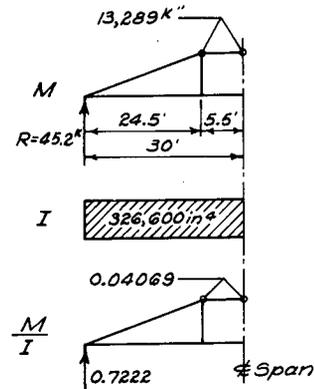
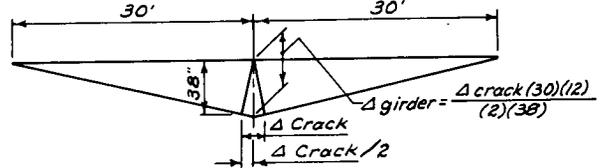


Figure A-26. Test No. 1 calculated deflections.

- Note that $4.5 \sqrt{f'_c}$ and $2.7 \sqrt{f'_c}$ do not corroborate.
 - AASHTO specifications give modulus of rupture = $7.5 \sqrt{f'_c}$.
3. Verify measured deflection due to test load of 90.4 kips (last load cycle):
- a. Assume 8 major cracks to be concentrated at centerline span, and neglect minor cracks. (Typical)
 - b. Summation cracks = $(2)(0.5) + (5)(0.4) + 0.2 = 3.2$ mm/25.4 = 0.126 in.
 - c. Elastic Δ was computed from the M/I diagram (Fig. A-26) as follows:
 - Strain in gages 112 and 113 remained approximately constant, or reduced, after reaching open cracks.
 - P test = 90.4 kips, $R = \frac{90.4}{2} = 45.2$ kips, M at 24.5 ft = $13,289$ in.-kip.
 - $Rl = 0.7222$ M/I at centerline span = 14.24 .
 - $\Delta = \frac{M}{EI} = (14.24)(1,000)(144)/4,945,000 = 0.41$ in.
 - Δ girder $\approx 0.41 + (12)(30)(0.126)/(38)(2) \approx 0.41 + 0.60 \approx 1.01$ in.
 - Measured deflection (last cycle) = 1.19 in. (1st = 1.16 , 2nd = 1.14).

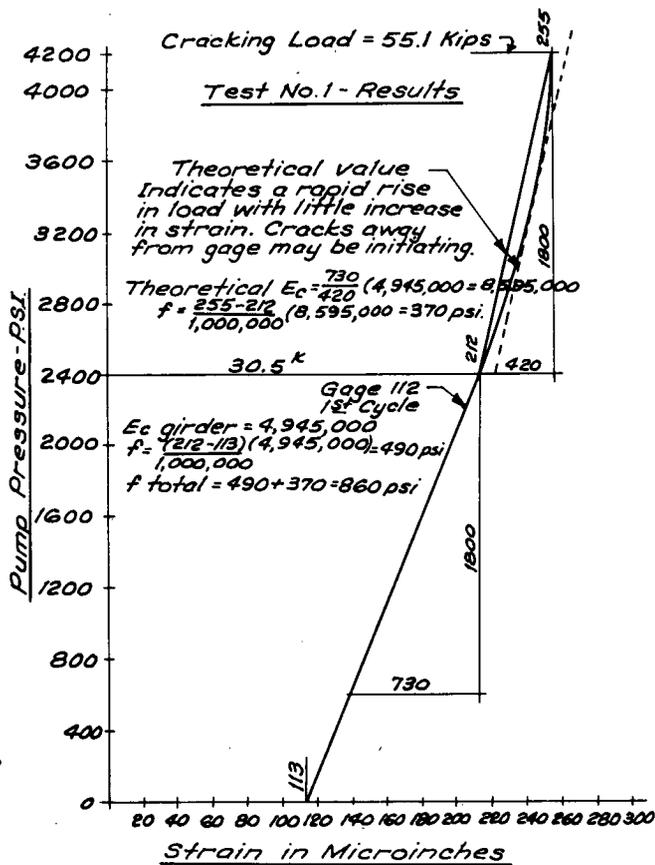


Figure A-27. Load versus strain bottom of girder test No. 1, Gage 112.

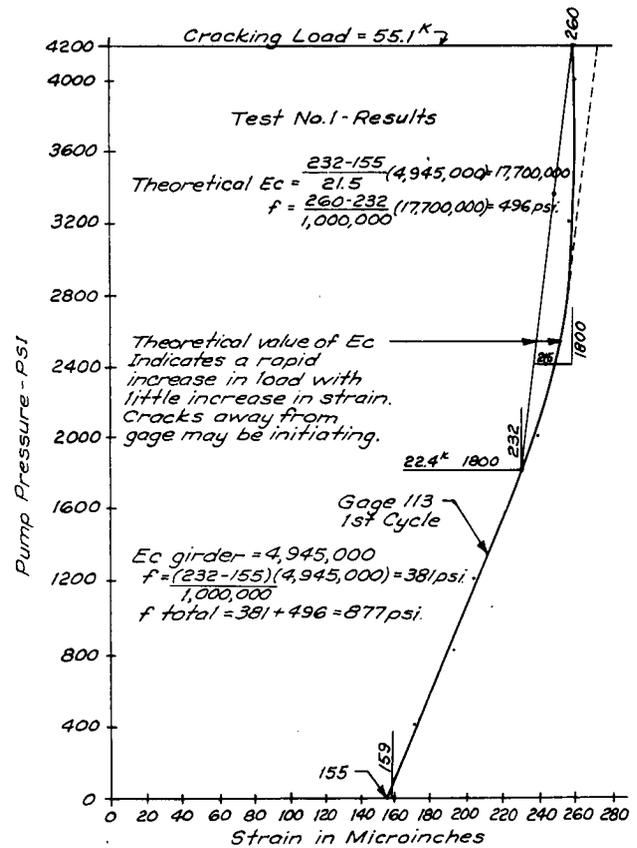


Figure A-28. Load versus strain bottom of girder test No. 1, Gage 113.

4. Calculate stresses from strains at centerline span (last cycle):

a. Inelastic strains in top slab not deducted, Figure A-25:

- $E_c = 4,240,900$ psi.
- f top of slab = $(0.000505)(4,240,900) = 2,141$ psi (c).
- f bottom of slab = $(0.000079)(4,240,900) = 335$ psi (t).
- $F_c = (2.141) \left(\frac{5.62}{2} \right) (90) = 541$ kips (C).
- $F_T = (0.335) \left(\frac{0.88}{2} \right) (90) = 13$ kips (T).

The forces in the top slab are developed from top slab strains.

The force in the strands is developed from $\Sigma H = 0$.

- F_T strands = $541 - 13 = 528$ kips (T).
- $f_s = \frac{528}{(16)(0.153)} = 216$ ksi.
- M measured = $(528) \left(\frac{41.88}{12} \right) + (13) \left(\frac{2}{3} \right) \left(\frac{0.88}{12} \right) + (541) \left(\frac{2}{3} \right) \left(\frac{5.62}{12} \right) = 2,013$ ft-kip.
- M applied = $\frac{(90.4)(60)}{4} + 554$ DLM = 1,910 ft-kip.
- Ratio $\frac{\text{Measured } M}{\text{Applied } M} = \frac{2,013}{1,910} = 1.05$.

b. Inelastic strains in top slab deducted, Figure A-25:

- f top of slab = $(0.000059)(4,240,900) = 250$ psi (c).

- From Figure A-20, $e_s = (56) \frac{59}{56} = 1.05$ (Maximum = 1.0, thence no correction for creep).
- Ratio $\frac{\text{Measured } M}{\text{Applied } M} = 1.05$ (no change).

5. Check stresses from strand gages 110 and 111 (last cycle):

a. Strand gage 110:

- $+1,302 - (-102) = (0.001404)(28,000) = 39.3$ ksi.
- f_s max. = $189 - 28.7$ (losses) + $39.3 = 200$ ksi.

b. Strand gage 111:

- $+1,847 - (+116) = (0.001731)(28,000) = 48.5$ ksi.
- f_s max. = $189 - 28.7$ (losses) + $48.5 = 209$ ksi.
- Then, $f_s = 216$ ksi $\approx 200 \approx 209$ ksi.

Load-deflection curves for the three cycles of Test 1 are shown in Figure A-29. These curves show the good corroboration obtained between the second and last (third) cycles of this test. Comparing the first cycle with the other cycles, it is seen that the deflection of the cracked section was markedly different from the uncracked section through much of the load test. However, the final deflections for all three cycles are practically equal. The maximum final difference of deflections was 3.5 percent. The first cycle test shows that the cracked condition occurred at approximately 55 kips, corroborating the observed cracking at this load. Figures A-30 and A-31 show the load-

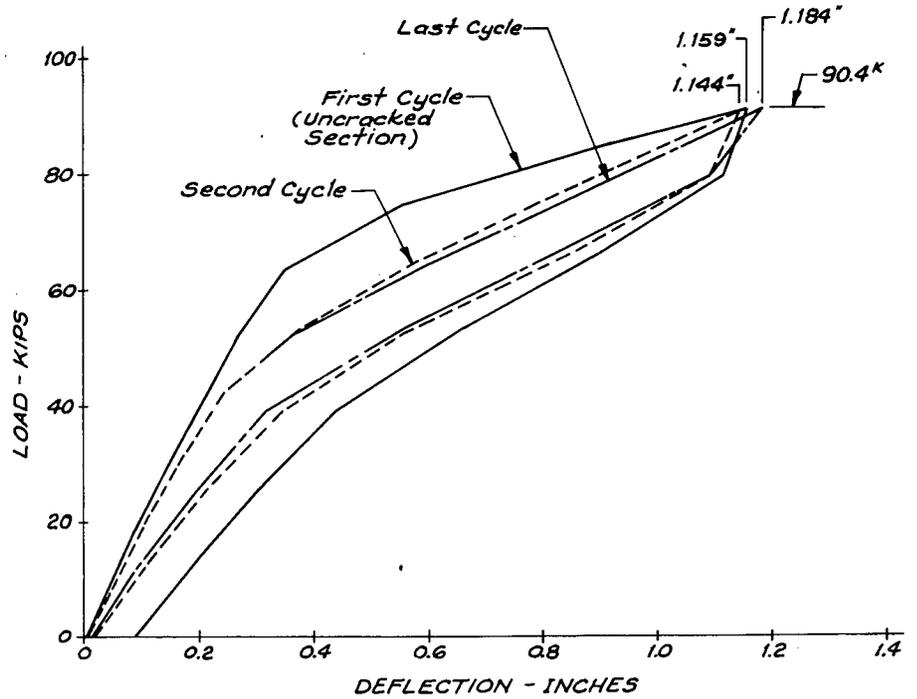


Figure A-29. Test No. 1 deflections.

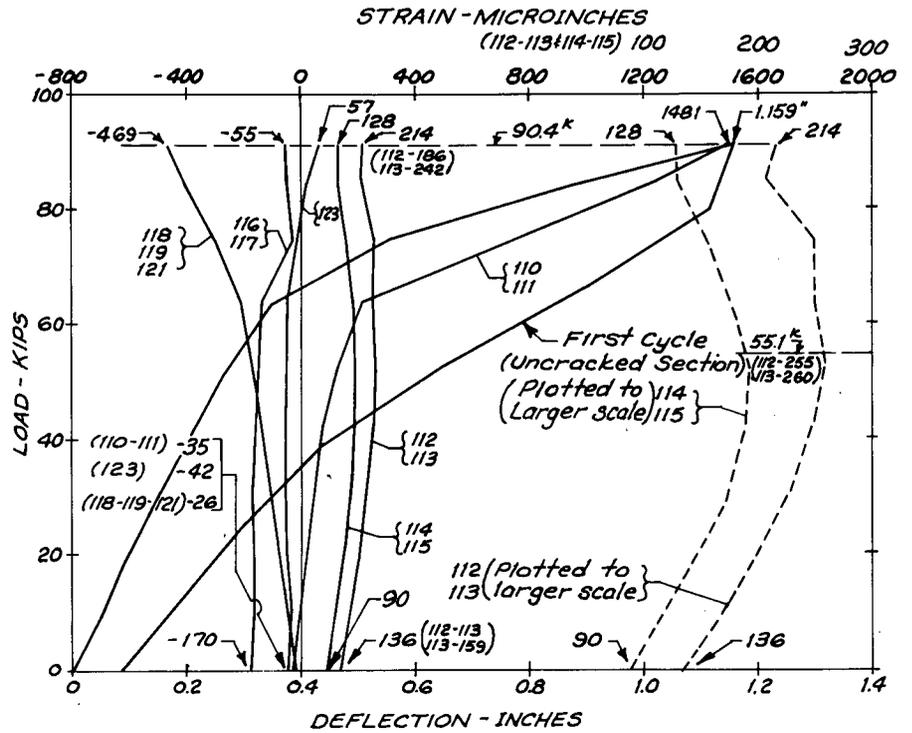


Figure A-30. Test No. 1 first cycle deflection and strains.

deflection and load-strain curves for the first cycle and last cycle of Test 1. In Figure A-30, gages 112, 113 and 114, 115 corroborate the cracking load of 55.1 kips. Strand gages 110 and 111 clearly show the change in strain when the girder became cracked, as do gages 118, 119, and 121 on the top of the slab. The sudden increase in strain occurs at the point where cracking causes the initiation of dead load redistribution. At maximum test load, strain gage readings include the effect of dead load. Figure A-31, last cycle, shows more uniform curves for both load deflection and load strain. Due to the cracked section, the load-strain curves for concrete gages 112, 113, 114, 115, 116, and 117 became erratic after the first cycle, and were not used for calculations in following tests. The only exception to this was in Test 5, where new gages 112 and 113 were installed on the patched section.

Load Test 2

Test 2 was a test of the same girder with 16 straight strands plus the addition of two external 1-in. diameter, Grade 150 thread bars.

This test simulated the strengthening of a girder that might be deficient because of increased live or dead loads. The bars were post-tensioned to 84 kips each against concrete corbels attached to both sides of the girder. One bar was temporarily jacked to 97.5 kips (a stress of 0.9 F_y) to check sufficiency of corbels. The inside face of each corbel was 13 ft 0 in. from centerline of span. Each corbel was 4 ft 0 in. long (see Fig. A-13). The centroid of each bar was 1 ft 2½ in. above the bottom

of the girder. The calculated ultimate moment capacity of the girder was 3,242 ft-kip. Thence 75 percent of the ultimate moment was 2,432 ft-kip including dead load moment of 554 ft-kip, giving an allowable test load moment of 1,878 ft-kip for this test. However, both test load deflection, not to exceed Test 1 deflection, and allowable stress in the thread bars, not to exceed 0.9 F_y , controlled. The computed test load of 114 kips was increased slightly to 114.7 kips to facilitate jacking load readings. The total applied test moment, including dead load moment, was 2,275 ft-kip. The test load moment of 1,721 ft-kip (not including DLM) was equivalent to 2.47 times the HS-20 LL + I service load moment of 697 ft-kip. At time of testing the girder concrete strength (f'_c) was 6,300 psi, and the top slab concrete strength was 5,200 psi. Test 2 occurred 54 days after casting the girder and 50 days after release of strands. The slab was cast 40 days prior to Test 2. The computed 50-day prestress loss was 29.9 ksi.

At maximum load the same eight major cracks, as in Test 1, were observed to extend within a few inches of the bottom of the slab. The maximum crack width was 0.4 mm. Strain data indicated tension in the bottom of the top slab. The maximum vertical deflection caused by the test was 1.18 in., equal to 99 percent of Test 1 deflection. The computed deflection for the last load cycle was 0.96 in. Maximum moments at centerline of span were developed from strain data and compared to the applied moments (see Table A-4). Strand stresses ranged from 61 percent to 75 percent of f'_s (see Table A-6). Thread bar stresses were 88 percent of F_y (90 percent F_y allowable). The methodology of developing moments and stresses from strains

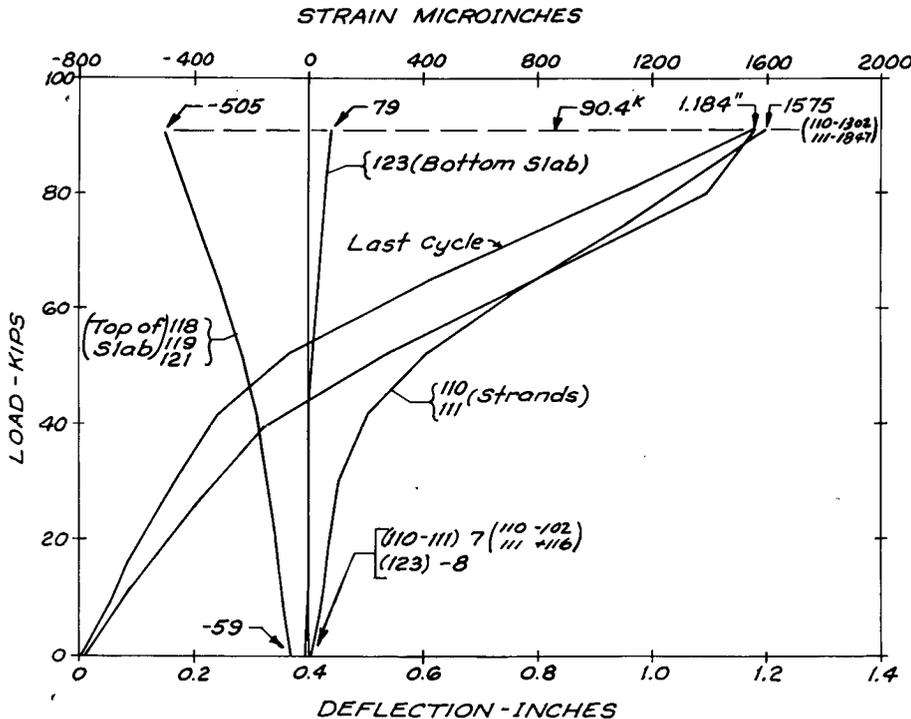


Figure A-31. Test No. 1 last cycle deflection and strains.

is shown in Test 8 (see Fig. A-32 for load-deflection and load-strain curves).

Load Test 3

Test 3 was nearly identical to Test 1 with 16 straight pre-tensioned strands, except the girder had been previously cracked. The calculated ultimate moment capacity was 2,511 ft-kip. Thence 75 percent of the ultimate moment was, 1,883 ft-kip, including dead load moment of 554 ft-kip. The total applied test moment was, 1,840 ft-kip equaling 98 percent of 1,883 ft-kip. The maximum applied test load was 85.7 kips. The test load moment of 1,286 ft-kip (not including DLM) was equivalent to 1.85 times the HS-20 LL + I service load moment of 697 ft-kip. At time of testing the girder concrete strength (f'_c) was 6,300 psi, and the slab concrete strength was 5,200 psi. Test 3 occurred 54 days after casting the girder and 50 days after release of strands. The slab was cast 40 days prior to Test 3. The computed 50-day prestress loss was 29.9 ksi.

At maximum load the same eight cracks, as in Test 1, were observed to extend within a few inches of the bottom of the slab. The maximum crack width was 0.5 mm. Strain data indicated tension in the bottom of the slab. The maximum vertical deflection caused by the test was 1.15 in., equal to 97 percent of Test 1 deflection. The computed deflection for the last load

cycle was 0.90 in. Maximum moments at centerline of span were developed from strain data and compared to the applied moments (see Table A-4). Strand stresses ranged from 73 percent to 78 percent of f'_s , (see Table A-6). The methodology of developing moments and stresses from strains is shown in Tests 1 and 5. The load-deflection and load-strain curves for Test 3 (Fig. A-33) are nearly the same as for the last cycle of Test 1. Although the load-strain curve shapes for strand gages 110 and 111 are nearly the same for Tests 1 and 3, note the shift at zero load that has occurred. This phenomenon is discussed elsewhere in this appendix.

Load Test 4

Enough concrete was removed from the bottom flange to sever four strands, one each side in the bottom layer and one each side in the second layer. Figure A-35 shows the approximate amount of concrete removed. The bulk of the concrete was removed with a chipping hammer; fine chipping near adjacent strands was done with a rivet gun chipper. The four strands were cut with an electric sander. The foregoing process was entirely satisfactory. The calculated ultimate strength of the damaged 12-strand girder was 1,887 ft-kip. Thence 75 percent of the ultimate moment was 1,415 ft-kip, including dead

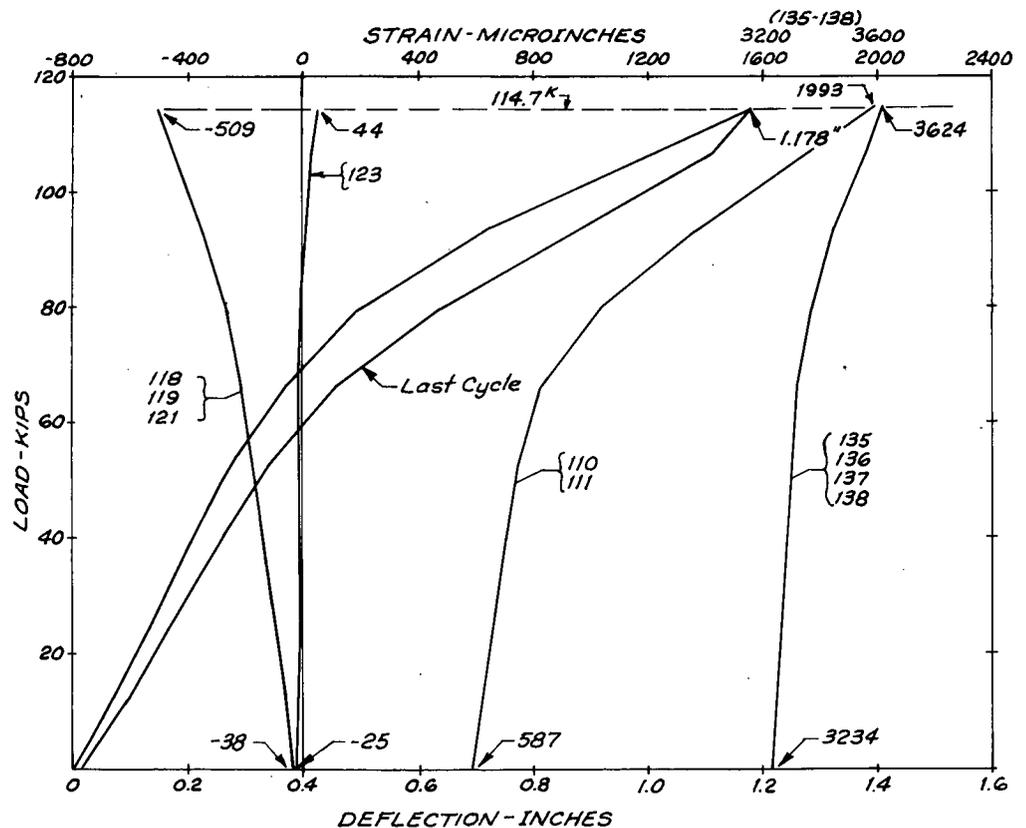


Figure A-32. Test No. 2 last cycle deflection and strains.

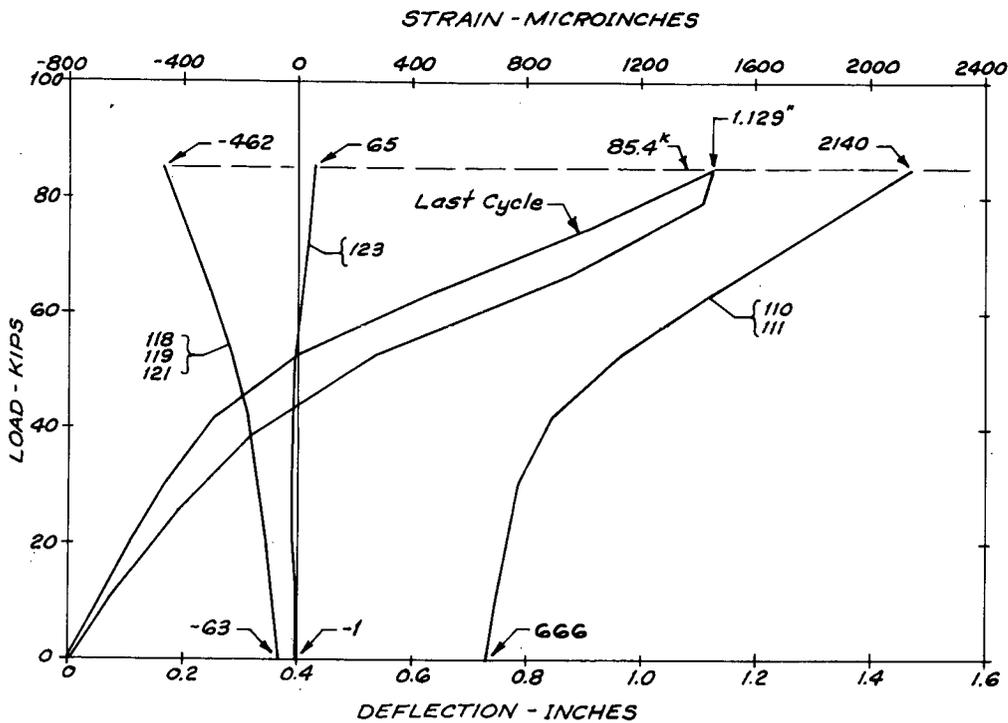


Figure A-33. Test No. 3 last cycle deflection and strains.

load moment of 554 ft-kip. The total applied test moment was 1,423 ft-kip equaling 101 percent of 1,415 ft-kip. The maximum applied test load was 57.9 kips. The test load moment of 869 ft-kip (not including DLM) was equivalent to 1.25 times the HS-20 LL + I service load moment of 697 ft-kip. At time of testing the girder concrete strength (f'_c) was 6,350 psi and the slab concrete strength was 5,335 psi. Test 4 occurred 56 days after casting the girder and 52 days after release of strands. The slab was cast 42 days prior to Test 4. The computed 50-day prestress loss of 29.9 ksi for Test 3 was also used for Test 4.

At maximum load the same eight major cracks as in Test 1 were observed to extend within a few inches of the bottom of the slab. The maximum crack width was 0.6 mm. Strain data indicated tension in the bottom of the slab. The maximum vertical deflection caused by the test was 0.79 in. This vertical deflection agreed exactly with the 0.79-in. maximum deflection for Test 7, which was also a 12-strand test. The computed deflection for the last load cycle was 0.65 in. Maximum moments at centerline of span were developed from strain data and compared to the applied moments (see Table A-4). Strand stresses ranged from 74 percent to 83 percent of f'_s (see Table A-6). The methodology of developing moments and stresses from strains is shown in Tests 1 and 5. Figure A-34 shows the load-deflection and load-strain curves for this test. Note the much lower vertical load at which strands 110 and 111 experienced rapid increase of strain. Gages 118, 119, and 121 also corroborate the load at which the girder behaves as a cracked section.

Load Test 5

Test 5 was a test of a 16-strand girder with four strands being internally spliced. Enough concrete was removed to install four

internal strand splices (see Fig. A-35 and Figs. 19, 20, and 22). These splices were used to connect the four severed strands of Test 4. Each splice was torqued to 433 ft-lb, giving a post-tension force of 24.1 kips per strand. The computed force in the 12 nonsevered strands was 24.3 kips per strand. The details of installing the splices are a part of the manual of recommended practice, and development of load versus torque is a part of component testing. The girder was preloaded to 30.5 kips prior to concrete patching. The calculated ultimate moment was 2,511 ft-kip, as calculated for Tests 1 and 3. Thence 75 percent of the ultimate moment was 1,883 ft-kip, including dead load moment of 554 ft-kip. The total applied test moment was 1,809 ft-kip, equaling 96 percent of 1,883 ft-kip. The maximum applied test load was 83.7 kips. To give the required moment of 1,883 ft-kip, the test load should have been 88.6 kips. As explained in more detail under "Strain Gage Readings" the total strains in the strand gages were giving readings that indicated that the maximum stress in the strands could be above the yield point. A decision was made during testing to stop the test at a load that was 4.9 kips less than the computed allowable test load. It is believed that testing the girder to a total test moment of 96 percent allowable was a sufficient "proof test." The test load moment of 1,256 ft-kip (not including DLM) was equivalent to 1.80 times the HS-20 LL + I service load moment of 697 ft-kip. At time of testing, the girder concrete strength (f'_c) was 6,300 psi and the slab concrete strength was 5,200 psi. The girder patch concrete strength was 5,290 psi. Test 5 occurred 61 days after casting the girder and 57 days after release of strands. The slab was cast 47 days prior to Test 5. The computed 57-day prestress loss was 30.3 ksi.

The first observed crack in the south girder patch occurred near the end of an internal splice at a test load of 49.7 kips. The crack did not develop into a major crack. The stresses

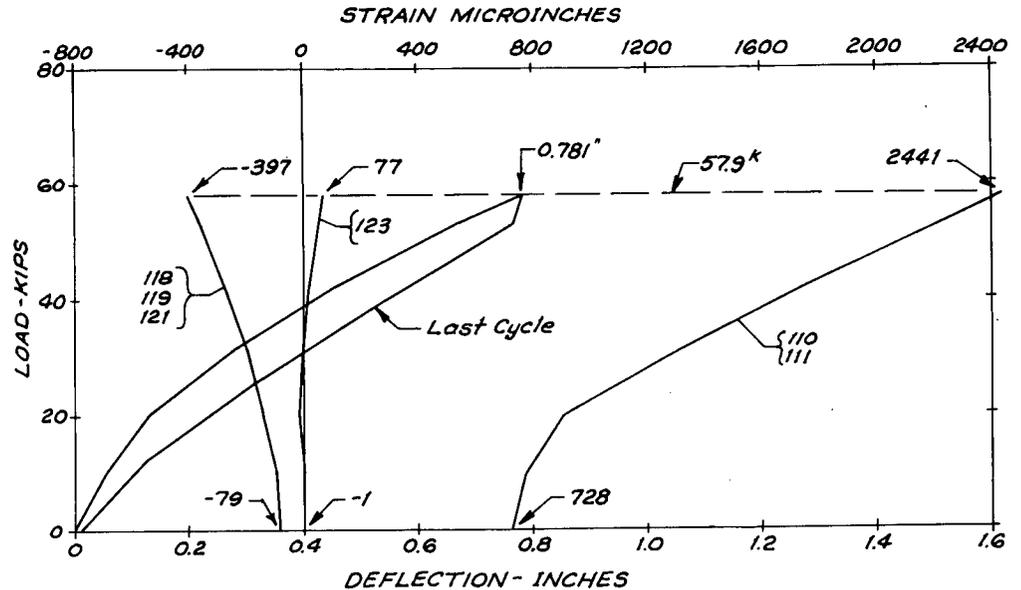


Figure A-34. Test No. 4 last cycle deflection and strains.

developed from strains indicated that preloading was successful. Using the load-strain data shown in Figure A-36, this patch developed a tensile stress of about $4.2 \sqrt{f'_c}$, and without preload the theoretical tensile stress would have been about $10.7 \sqrt{f'_c}$, resulting in a much lower cracking load. Using the load-strain data shown in Figure A-37, the patch on the north side of the bottom flange cracked at a tensile stress of about $8.5 \sqrt{f'_c}$, and without preload the theoretical tensile stress would have been about $15 \sqrt{f'_c}$, resulting in a lower cracking load. The stress in the patched area due to removal of the preload was not verified to complete satisfaction because of the inadvertent removal of the preload prior to attaching strain gages to the bottom of the girder in the patched area. At maximum load the same eight major cracks, as in Test 1, were observed to extend within a few inches of the bottom of the slab. The total number of major and minor cracks was 22. The maximum crack width was 0.7 mm. Strain data indicated tension in the bottom of the slab. The maximum vertical deflection caused by the test was 1.10 in., equal to 92 percent of Test 1 deflection. The difference in deflections was believed due to two causes. The test load was 6.7 kips less than Test 1, and the four internal splices added additional stiffness near centerline of span. The computed deflection for the last load cycle was 0.76 in. Maximum moments at centerline of span were developed from strain data and compared to the applied moments (see Table A-4). Strand stresses ranged from 71 percent to 76 percent of f'_s (see Table A-6). The following calculations show the methodology used (refer to Figs. A-38 and A-39 for stress and deflection diagrams):

1. Given:

- Test 5 girder with four internal splices.
- Section properties of nondamaged girder: $A = 559.5 \text{ in.}^2$, $I = 125,400 \text{ in.}^4$, $S \text{ bottom} = 6,190 \text{ in.}^3$
- Section properties of girder and slab: $A = 1,027 \text{ in.}^2$, $I = 326,600 \text{ in.}^4$, $S \text{ bottom} = 9,895 \text{ in.}^3$
- Section properties of damaged girder: Remove portion of

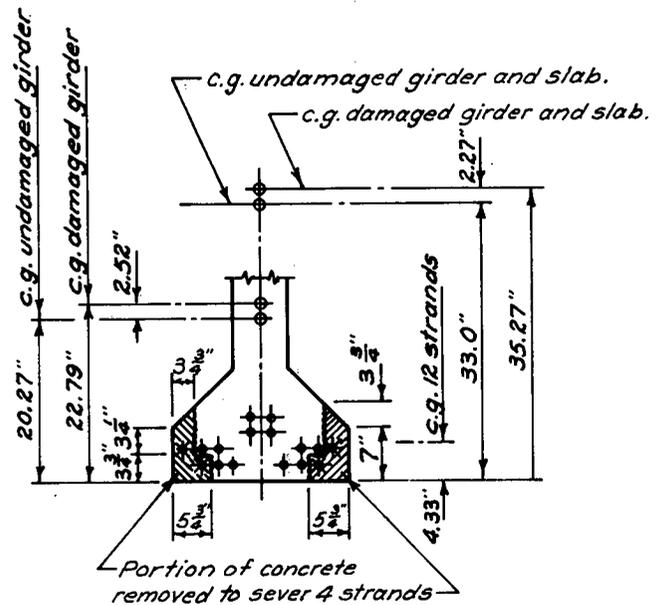


Figure A-35. Four strands severed.

concrete and sever 4 strands. 12 strands remain, 4 of 16 strands severed. $A = 485.4 \text{ in.}^2$, $I = 103,012 \text{ in.}^4$, $S \text{ bottom} = 4,520 \text{ in.}^3$

- Section properties of damaged girder and slab: $A = 952.9 \text{ in.}^2$, $I = 258,366 \text{ in.}^4$, $S \text{ bottom} = 7,325 \text{ in.}^3$
- $f \text{ bottom prior to damage (DL + P.S. + LL + I)} = 365 \text{ psi (t)}$. Allowable $f \text{ bottom} = 424 \text{ psi (t)}$.

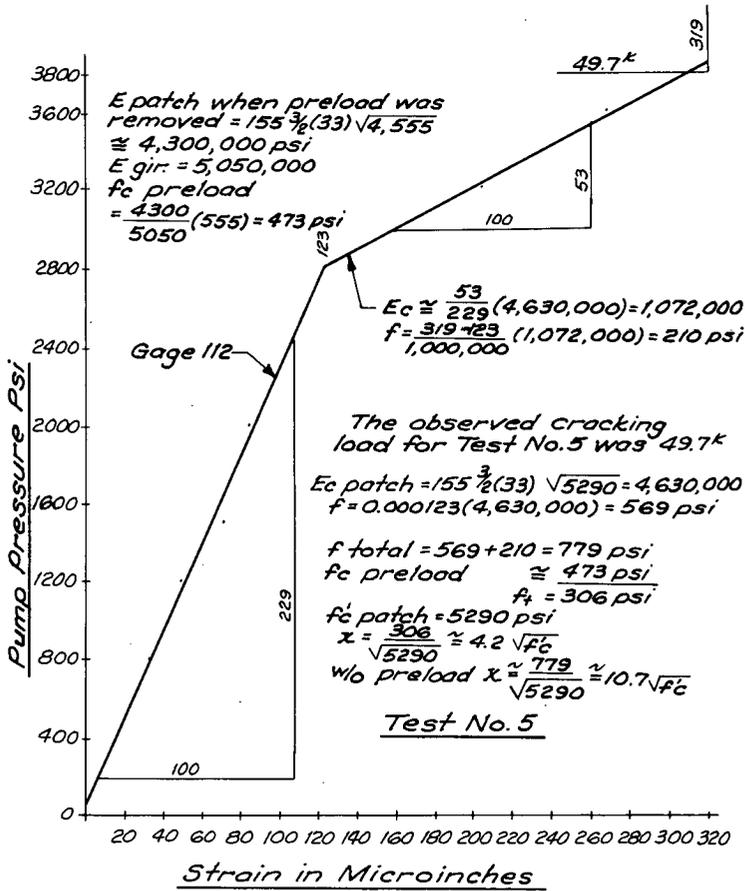


Figure A-36. Load strain curve to cracking, load Gage 112.

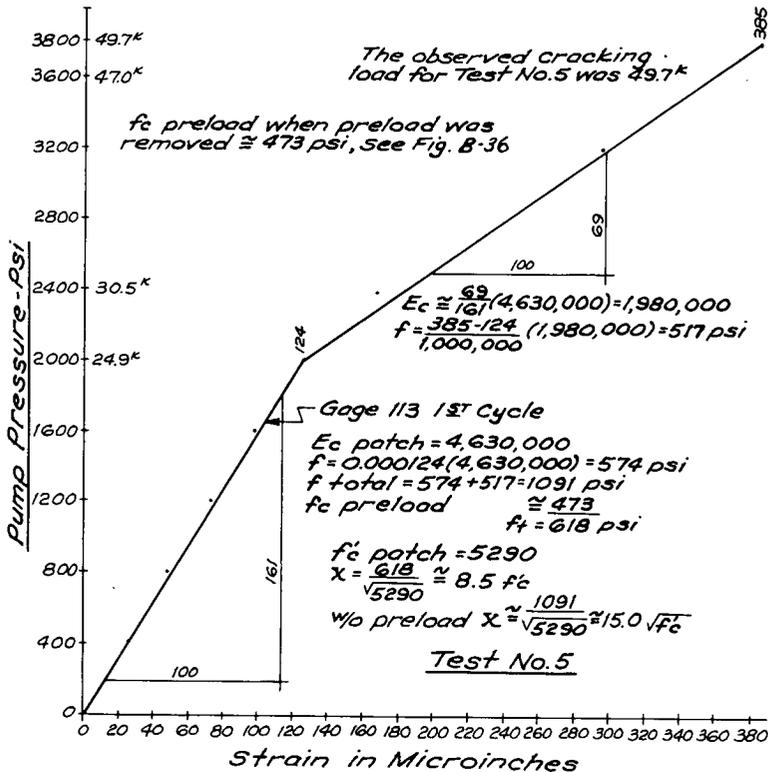


Figure A-37. Load strain curve to cracking, load Gage 113.

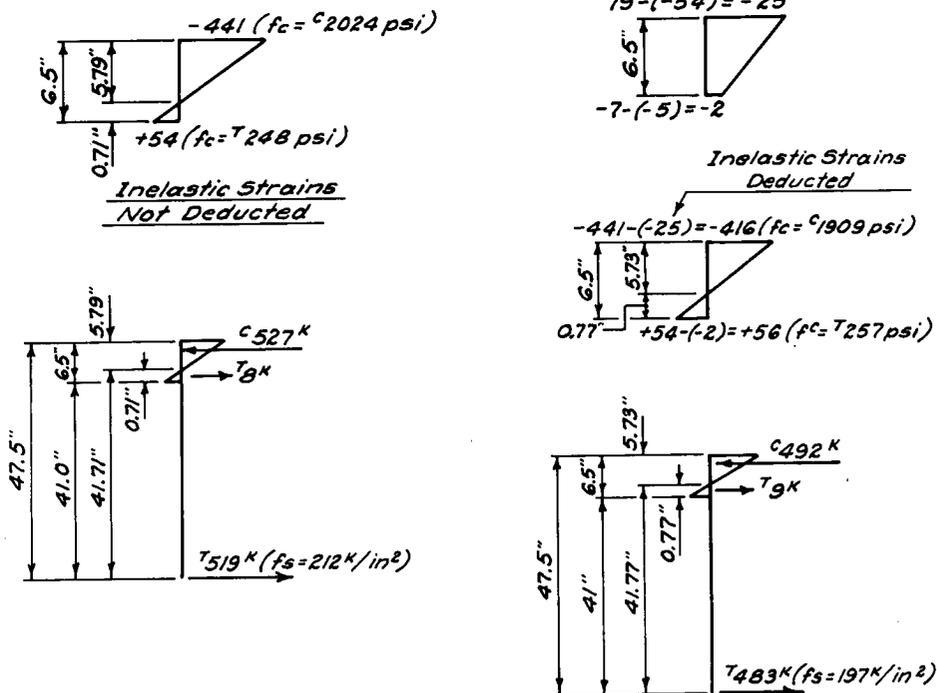


Figure A-38. Test No. 5 stresses due to strains.

2. Determine if preloading is required.
 - a. Assume that prestress plus dead load of girder and slab act on the damaged girder section alone.
 - f_c bottom prestress = $(12)(22)/485.4 + (12)(22)(18.46)/4,520 = 1,622$ psi (c).
 - f_t bottom of girder = $(554)(12)/4,520 = 1,471$ psi (DL slab + girder).
 - f total = 1,622 compression - 1,471 tension = 151 psi (c).
 - b. Assume that prestress plus dead load of girder and slab act on the composite damaged girder.
 - f_c bottom prestress = $(12)(22)/953 + (12)(22)(30.92)/7,325 = 1,391$ psi (c).
 - f_t bottom of girder = $(554)(12)/7,325 = 908$ psi (t).
 - f total = 1,391 compression - 908 tension = 483 psi (c).
 - Allowable = $0.4 f'_c = (0.4)(5,000) = 2,000$ psi (c).
 - Therefore preloading is not needed to reduce compressive stress.

3. Calculate preload required for tensile stress:

Splice strands prior to preloading. There will be no compression on the patched area unless preload is used. Determine the amount of prestress that could be restored by preloading.

 - f_t = (from LL + I at bottom of damaged section) = $\frac{M}{S}$
 $= (697)(12)/7,325 = 1,142$ psi (t).
 - f_c = (from 4 strands being tensioned at bottom of damaged section) = $\frac{(4)(22,000)}{953} + \frac{(4)(22,000)(35.27-3)}{7,325} = 92$
 compression + 388 compression = 480 psi (c).
 - f_c = (from DL + P.S. at bottom of damaged section) = 480 compression + 151 compression = 631 psi (c).

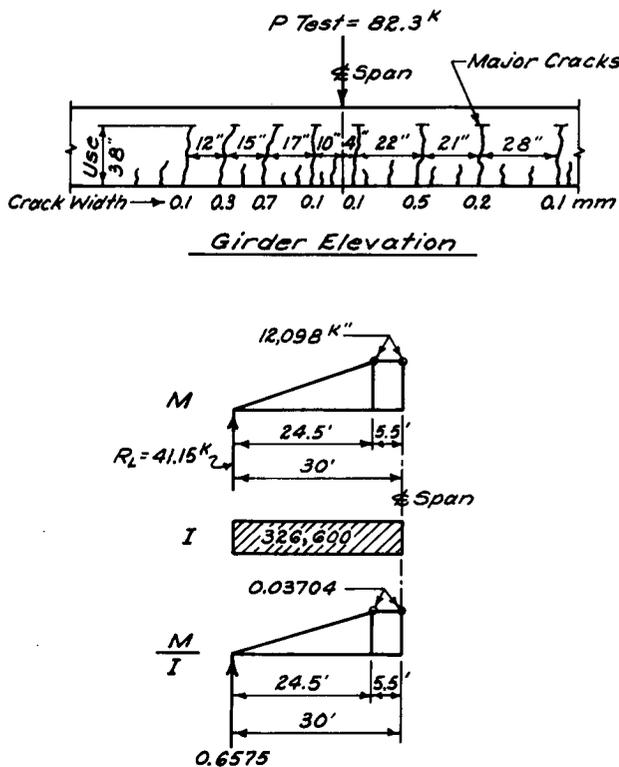


Figure A-39. Test No. 5 calculated deflections.

- $f_t = (\text{from DL} + \text{PS} + \text{LL} + \text{I}) = 1,142 \text{ tension} - 631 \text{ compression} = 511 \text{ psi (t)}$.
 - Restore girder f_t bottom to 365 psi (t).
 - Allowable f_t on damaged section = $424 + 631 = 1,055 \text{ psi (t)}$.
 - Stress needed to reduce tension on total section to 365 psi tension under LL + I = $845 \text{ tension} - 365 \text{ tension} = 480 \text{ psi (t)}$.
 - f_t on damaged section = $424 + 480 = 904 \text{ psi (t)}$.
 - $M = \frac{480}{845} (697) 396 \text{ ft-kip}$.
 - $P = \frac{4M}{\ell} = \frac{(4)(396)}{60} = 26.4 \text{ kips (Preload required)}$.
4. Review result:
- Preload stress on total section = $\frac{(396)(12)}{9,895} = 480 \text{ psi (c)}$.
 - f_t bottom LL + I = $(697)(12)/9,895 = 845 \text{ psi (t)}$.
 - f_t LL + I + f_c preload = $845 \text{ tension} - 480 \text{ compression} = 365 \text{ psi (t)}$.
 - Design $f_t = 365 \text{ psi (t)}$. (A preload of 30.5 kips was used to facilitate load readings. Therefore, $\frac{30.5}{26.4} (480) = 555 \text{ psi}$ preload tensile stress.)
5. Verify measured deflection due to test load No. 5:
- E_c girder = $5,005,000 \text{ in.}^4$ P test = 83.7 kips.
 - Summation cracks = $(4)(0.1) + 0.2 + 0.3 + 0.5 + 0.7 = 2.1 \text{ mm} = 2.1/25.4 = 0.0827 \text{ in}$.
 - Elastic Δ was computed from the $\frac{M}{I}$ diagram (Fig. A-39) as follows:
 - Gages 112 and 113 show some increase in strain beyond the cracking load of 49.7 kips. This increase may be due to the localized effect of the four internal splices. Use the same method as previously used to compute deflection.
 - P test (last cycle = 82.3 kips, $R = \frac{82.3}{2} = 41.15 \text{ kips}$).
 - M at 24.5 ft-kip = 12,098 in.-kip, $R\ell = 0.6575$, M/I at centerline span = 12.96.
 - $\frac{M}{EI}$ at centerline span = $\frac{(12.96)(1,000)(144)}{5,055,000} = 0.37 \text{ in}$.
 - Δ girder $\cong 0.37 + \frac{(12)(30)(0.0827)}{(38)(2)} = 0.37 + 0.39 = 0.76 \text{ in}$.
 - Measured deflection = 1.06 in. (Last cycle P = 82.7 kips), = 1.08 in. (Second cycle P = 83 kips), = 1.10 in. (First cycle P = 83.7 kips).
6. Compute prestress losses for Test 5 (57 days after release):
- CR_c (creep) = $\frac{t^{0.6}}{10 + t^{0.6}}(11,310) = \frac{57^{0.6}}{10 + 57^{0.6}}(11,310) = (0.53)(11,310) = 5,994 \text{ psi}$
 - SH (shrinkage) = $(5,000) \frac{(t)}{(55 + t)} = \frac{(5,000)(57)}{55 + 57} = 2,550 \text{ psi}$.
 - ES (elastic shortening) = 8,340 psi.
 - CR_s (steel relaxation loss) = 13,400 psi.
 - Total loss for Test 5 = 30,300 psi.
7. Calculate stresses from strains at centerline span (last cycle):
- a. Inelastic strains in top slab not deducted, Figure A-38.
 - E_c slab = $4,590,000 \text{ in.}^4$
 - f top of slab = $(0.000441)(4,590,000) = 2,024 \text{ psi (c)}$.
 - f bottom of slab = $(0.000054)(4,590,000) = 248 \text{ psi (t)}$.
 - $F_c = \frac{(2,024)(5.79)(90)}{2} = 527 \text{ kips (c)}$.
 - $F_t = \frac{(0.248)(0.71)(90)}{2} = 8 \text{ kips (t)}$.
 - The forces in the top slab are developed from the top slab strains. The force in the strands is developed from $\Sigma H = 0$.
 - F_T strands = $527 - 8 = 519 \text{ kips}$.
 - $f_s = \frac{519}{(16)(0.153)} = 212 \text{ ksi}$.
 - M measured = $\frac{(519)(41.71)}{12} + \frac{(527)(2)(5.79)}{(3)(12)} = 1,974 \text{ ft-kip}$.
 - M applied = $\frac{(82.3)(60)}{4} + 554 \text{ DLM} = 1,789 \text{ ft-kip}$.
 - Ratio $\frac{\text{Measured M}}{\text{Applied M}} = \frac{1,974}{1,789} = 1.10$.
 - b. Inelastic strains in top slab deducted, Figure A-38.
 - f top of slab = $(0.000079)(4,590,000) = 363 \text{ psi (c)}$.
 - From Figure A-20, $e_s = 115$, $\frac{79}{115} (79) = 54 \text{ psi}$.
 - f bottom of slab = $(0.000007)(4,590,000) = 32 \text{ psi (c)}$.
 - From Figure A-20, $e_s = 10$, $\frac{7}{10} (7) = 5 \text{ psi (c)}$.
 - f top of slab = $(0.000416)(4,590,000) = 1,909 \text{ psi (c)}$.
 - f bottom of slab = $(0.000056)(4,590,000) = 257 \text{ psi (t)}$.
 - $F_c = \frac{(1,909)(5.73)(90)}{2} = 492 \text{ kips (C)}$.
 - $F_T = \frac{(0.257)(0.77)(90)}{2} = 9 \text{ kips (T)}$.
 - F_T strands = $492 - 9 = 483 \text{ kips (T)}$.
 - $f_s = \frac{483}{(16)(0.153)} = 197 \text{ ksi}$.
 - M measured = $\frac{(483)(41.77)}{12} + \frac{(492)(2)(5.73)}{(3)(12)} = 1,838 \text{ ft-kip}$.
 - M applied = 1,789 ft-kip.
 - Ratio $\frac{\text{Measured M}}{\text{Applied M}} = \frac{1,838}{1,789} = 1.03$.
8. Check stresses from strand gages 110 and 111:
- Gage 110; $2,300 - 1,098 = (0.001202)(28,000) = 33.7 \text{ ksi}$.
 - $f_s = 189 - 30.3 + 33.7 = 192 \text{ ksi}$.
 - Gage 111; $2,109 - 626 = (0.001483)(28,000) = 41.5 \text{ ksi}$.
 - $f_s = 189 - 30.3 + 41.5 = 200 \text{ ksi}$.
 - f_s of 197 ksi $\cong 192 \text{ ksi} \cong 200 \text{ ksi}$.
- The load-deflection and load-strain curves (Figs. A-40 and A-41) for this test corroborate the results of Tests 1 and 3. Gages 110, 111, 112, 113, 114, and 115 define the cracking load in the patched section of 49.7 kips. Comparing gages 110 and 111 for the first cycle, to gages 110 and 111 for the last cycle,

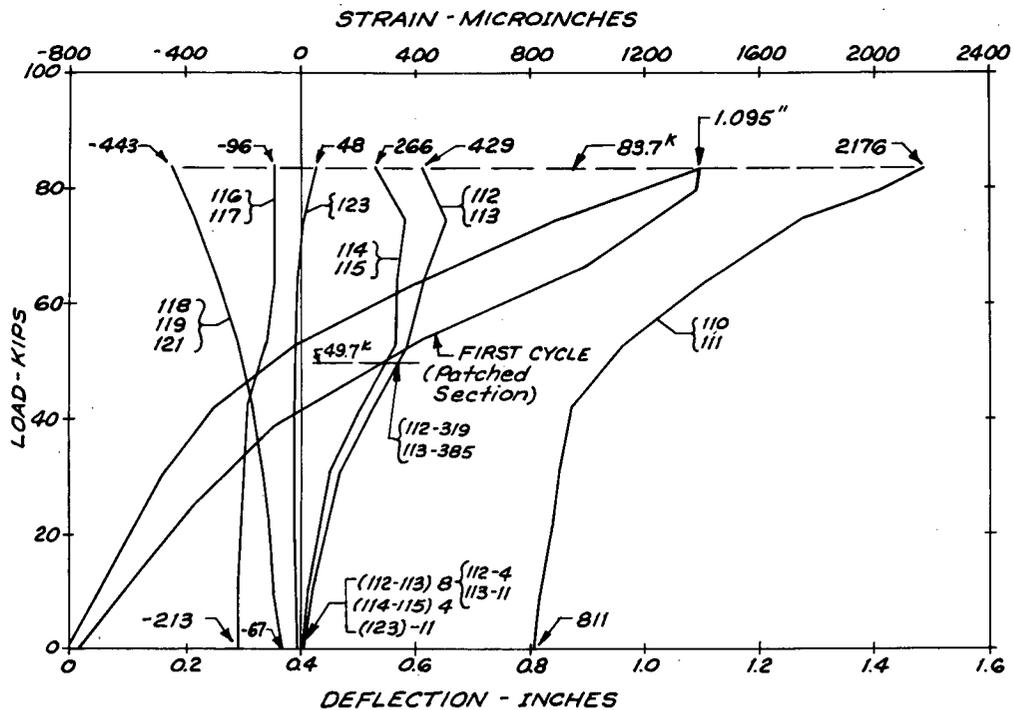


Figure A-40. Test No. 5 first cycle deflection and strains.

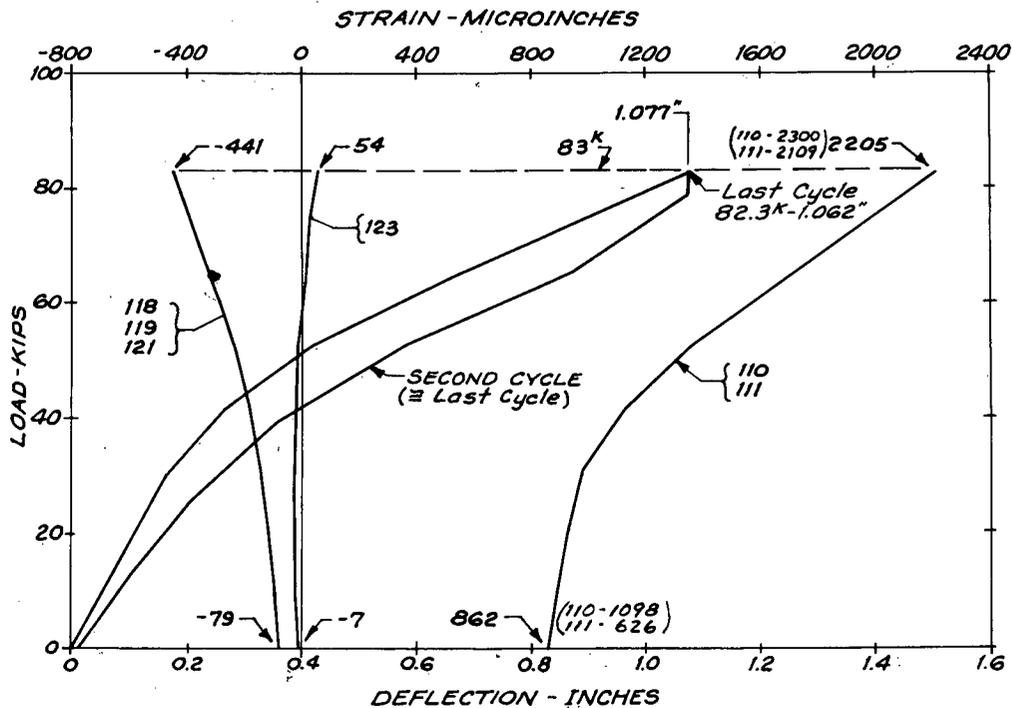


Figure A-41. Test No. 5 last cycle deflection and strains.

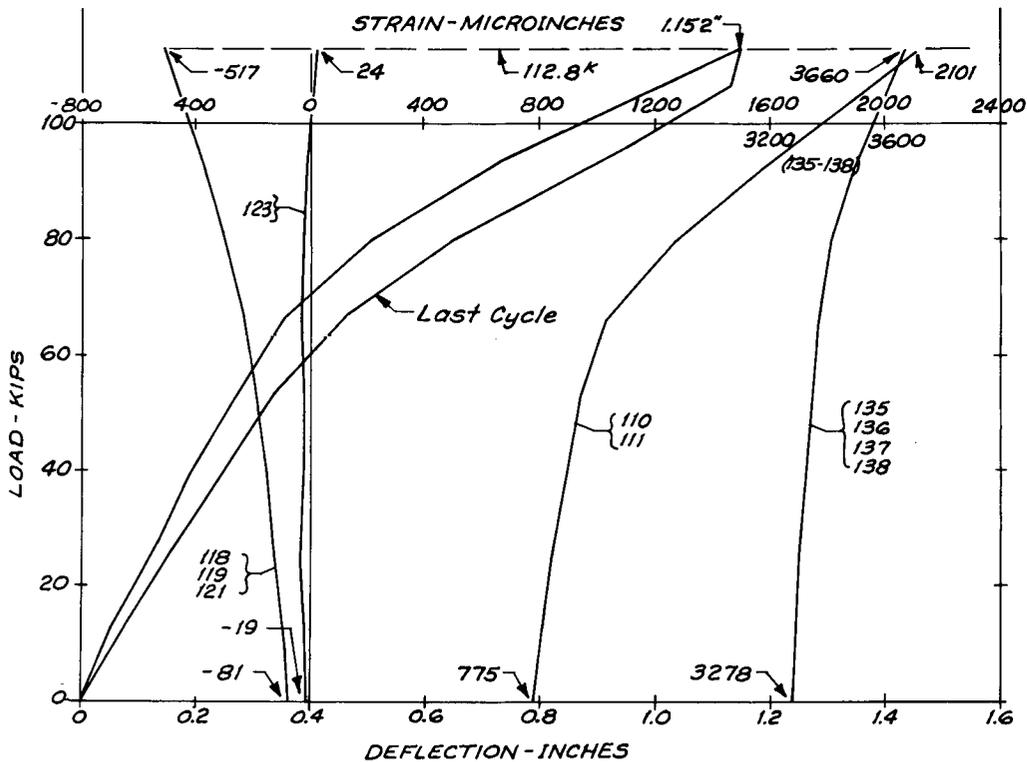


Figure A-42. Test No. 6 last cycle deflection and strains.

shows the effectiveness of preloading. Note that the final readings for gages 110 and 111 for both cycles are nearly the same. The close corroboration between Tests 1, 3, and 5 can be seen by referring to Figure 4 in Chapter Two.

Load Test 6

Test 6 was a test of the girder with 16 strands, four of which had been internally spliced (same as Test 5), plus the addition of two external 1-in. diameter, Grade 150 thread bars. The post-tensioning details were the same as Test 2. The calculated girder capacity was also the same as Test 2. A maximum test load of 113.4 kips was applied. The total applied test moment including dead load moment was 2,255 ft-kip. The test load moment of 1,701 ft-kip (not including DLM) was equivalent to 2.44 times the HS-20 LL + I service load moment of 697 ft-kip. At time of testing the girder concrete strength, f'_c , was 6,300 psi and the slab concrete strength was 5,200 psi. Test 6 occurred on the same day as Test 5; therefore, the prestress loss was the same 30.3 ksi.

At maximum load the same eight major cracks, as in Test 1, were observed to extend to within a few inches of the bottom of the slab. The maximum crack width was 0.7 mm. Strain data indicated tension in the bottom of the top slab. The maximum vertical deflection caused by the test was 1.16 in., equal to 97 percent of Test 1 deflection. The computed deflection for the last load cycle was 0.93 in. Maximum moments at centerline of

span were developed from strain data and compared to the applied moments (see Table A-4). Strand stresses ranged from 66 percent to 74 percent of f'_s (see Table A-6). Thread bar stresses were 89 percent of F_y , (90 percent F_y allowable). The methodology of developing moments and stresses from strains is shown in Test 8.

The load-deflection and load-strain curves (Figure A-42) for this test are nearly identical to the curves for Test 2. The strain gages define the points at which cracking initiates the redistribution of dead load. Note that the load-deflection curve also generally defines these points.

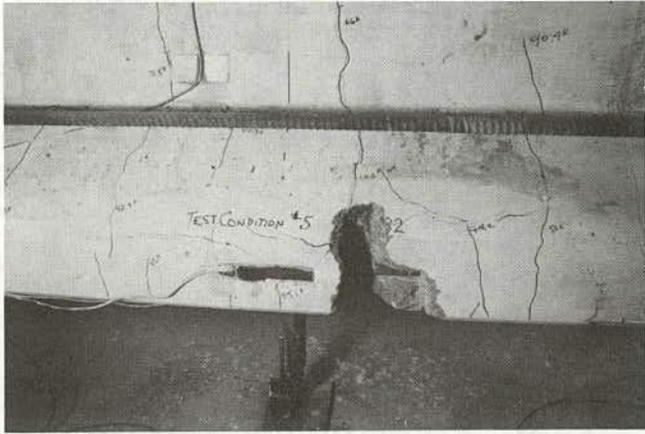
Load Test 7

Test 7 was similar to Test 4. The four internal strand splices of Test 5 were severed, leaving 12 prestressed strands. Enough concrete was removed to gain access to the four internal strand splices. The concrete was removed with a rivet gun chipping tool. The splice rods were cut with an electric sander. The concrete removed was not patched. The calculated girder strength and the applied test load were identical to Test 4. At time of testing the girder concrete strength, f'_c , was 6,300 psi and the slab concrete strength was 5,200 psi. Test No. 7 occurred on the same day as Tests 5 and 6; therefore, the prestress loss was the same 30.3 ksi.

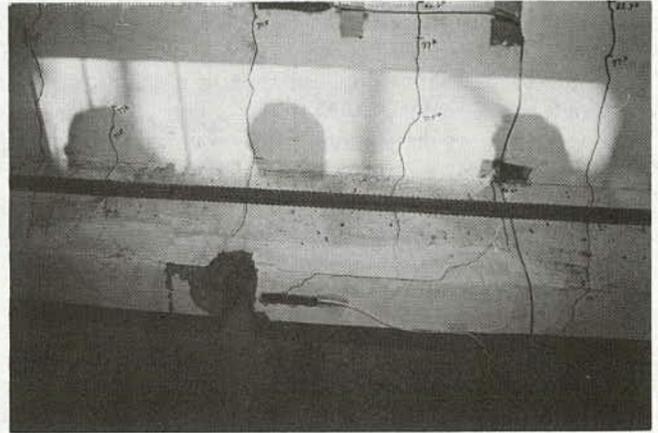
At maximum load the same eight major cracks, as in Test 1, were observed to extend within a few inches of the bottom of

the slab. The crack widths at the bottom of the girder near centerline of span were not representative due to the bursting effect when the internal splices were cut, as seen in Figure A-43. The strand splice should have been cut at splice ends as well as at centerline in order to get representative crack widths. No attempt was made to compute vertical deflections by crack measurements plus elastic deflection. However, the maximum deflection of 0.79 in. for Test 7 checked the maximum girder deflection of 0.79 in. for Test 4. Maximum moments at centerline

of span were developed from strain data and compared to the applied moments (see Table A-4). Strand stresses ranged from 74 percent to 91 percent of f'_s (see Table A-6). The methodology of developing moments and stresses from strains is shown in Tests 1 and 5. Load-deflection and load-strain curves are shown in Figure A-44. The first cycle and last cycle load-deflection curves show the good corroboration obtained from the two cycles. Note the much lower load at which cracking resulted in increased strain due to the severed four strands.



a. East Side of Girder



b. West Side of Girder

Figure A-43. Bursting effect from internal splices being cut.

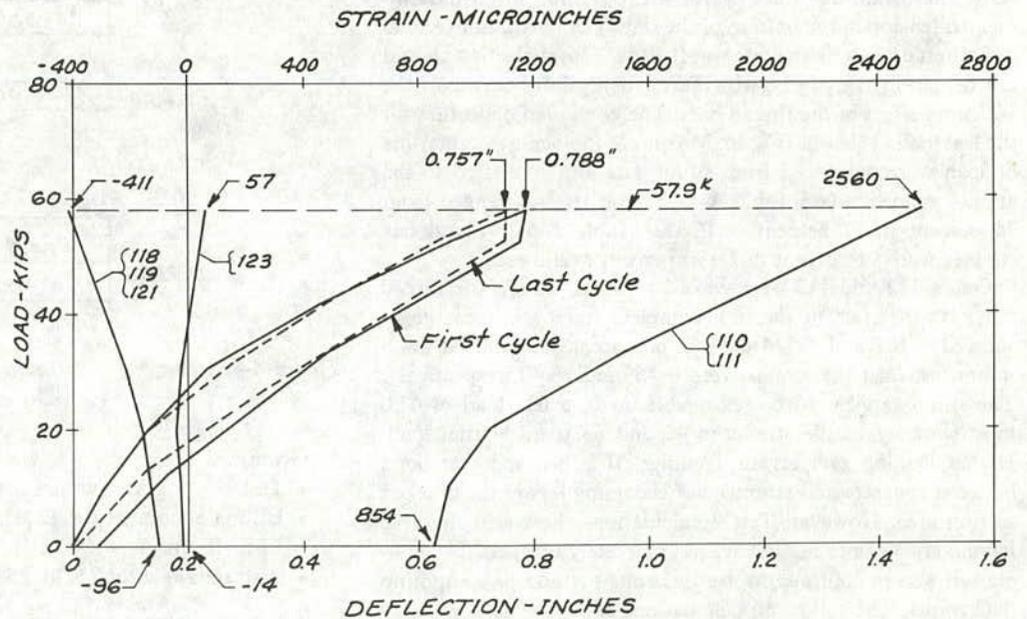


Figure A-44. Test No. 7 first and last cycle deflections and last cycle strains.

Load Test 8

Test 8 was a test of a 12-strand girder (4-strand splices had been severed in Test 7), plus repair by adding two external 1-in. diameter, Grade 150 thread bars. All loose or spalled concrete caused by Test 7 was removed. The four internal splices were removed. All concrete removed was patched. Four 3/8-in. diameter expansion bolts were used on each side of the girder to help hold the patches in place. It was determined that preloading was not needed. Earlier, Tests 2 and 6 had shown that the method used to estimate stress in the thread bars was conservative by 6 to 7 kips per bar. Therefore the initial jacking load was increased 5 kips from 84 to 89 kips per bar. In order to avoid overstressing the bottom flange, the bars were jacked in increments—52.4 kips first bar, 89 kips second bar, then 89 kips both bars. The ultimate moment capacity of the girder with 12 strands plus 2 bars was 2,630 ft-kip; however, the required ultimate moment of the 16-strand girder was 2,511 ft-kip. As determined in Test 1, 75 percent of 2,511 ft-kip equals 1,883 ft-kip. The results of Tests 2 and 6 indicated that the maximum test load could be increased to 89.7 kips with little if any over-stress beyond 0.9 f_y in the thread bars. The total applied test moment including dead load moment was 1,900 ft-kip equaling 101 percent of 1,883 ft-kip. The test load moment of 1,346 ft-kip (not including DLM) was equivalent to 1.93 times the HS-20 LL + I service load moment of 697 ft-kip. At time of testing the girder concrete strength, f'_c , was 6,290 psi, and the slab concrete strength was 5,350 psi. The girder patch concrete strength was 5,880 psi. Test 8 occurred 68 days after casting the girder and 64 days after release of strands. The slab was cast 54 days prior to Test 8. The computed 64-day prestress loss was 30.7 ksi.

The first observed crack in the girder patch occurred at a test load of 49.7 kips, which was identical to the cracking load of Test 5, thus confirming that preload was not necessary. At maximum load the same eight major cracks, as in Test 1, were observed to extend to within a few inches of the bottom of the slab. The maximum crack width was 0.6 mm. Strain data indicated tension in the bottom of the slab. The maximum vertical deflection caused by the test was 0.89 in. This deflection should not be expected to agree with Test 1 deflection because of the stiffening effect of the thread bars. The computed deflection for the last load cycle was 0.72 in. Maximum moments at centerline of span were developed from strain data and compared to the applied moments (see Table A-4). Strand stresses ranged from 74 percent to 77 percent of f'_s (see Table A-6). Thread bar stresses were 91 percent of f_y (90 percent f_y allowable).

Gages 112 and 113 were zeroed prior to jacking the thread bars. At the start of the first complete test cycle these gages showed -102 and -114 μ in. of compressive strain. At maximum test load the strains were +469 and -40, respectively. The strains appear to be reasonable up to a test load of 47.0 kips, showing tensile strains of 45 and 63 μ in. at that load. Higher loading gave erratic readings. It is believed that bond between concrete and strands was becoming less in the cracked section area. However, Test 2 calculations show that the area having cracks does not behave as completely cracked. The conclusion was to continue to use the same method of computing deflections. The following calculations show the methodology used (also see Figs. A-45, A-46, and A-47 for stress and deflection diagrams for this test):

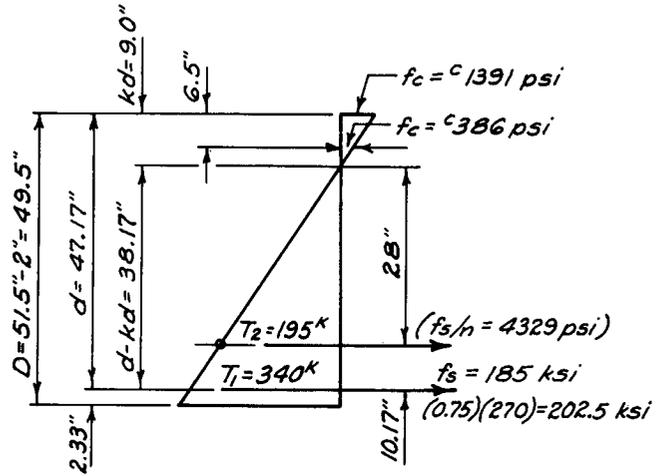


Figure A-45. Test No. 8 stress diagram.

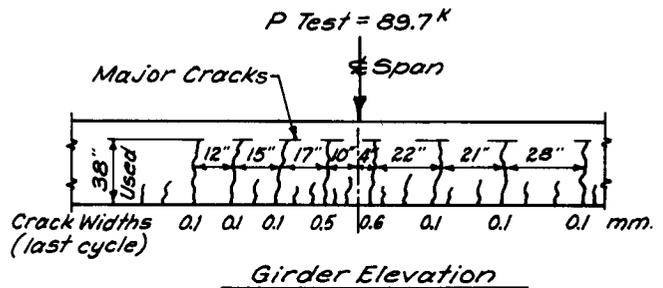
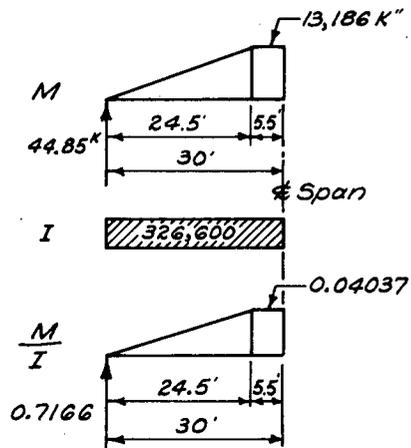


Figure A-46. Test No. 8 calculated deflections.



1. Given:
 - Test No. 8 girder with 12 strands plus 2 thread bars.
 - Ultimate moment for 12 strands (calculated in Test 4) = 1,887 ft-kip.
 - P strands = $(266)(12)(0.153) = 488.4$ kips.
 - c.g. equivalent compression zone = $\frac{(1,887)(12)}{488.4} + 4.33$
(c.g. strands) = 50.69 in. above girder bottom.

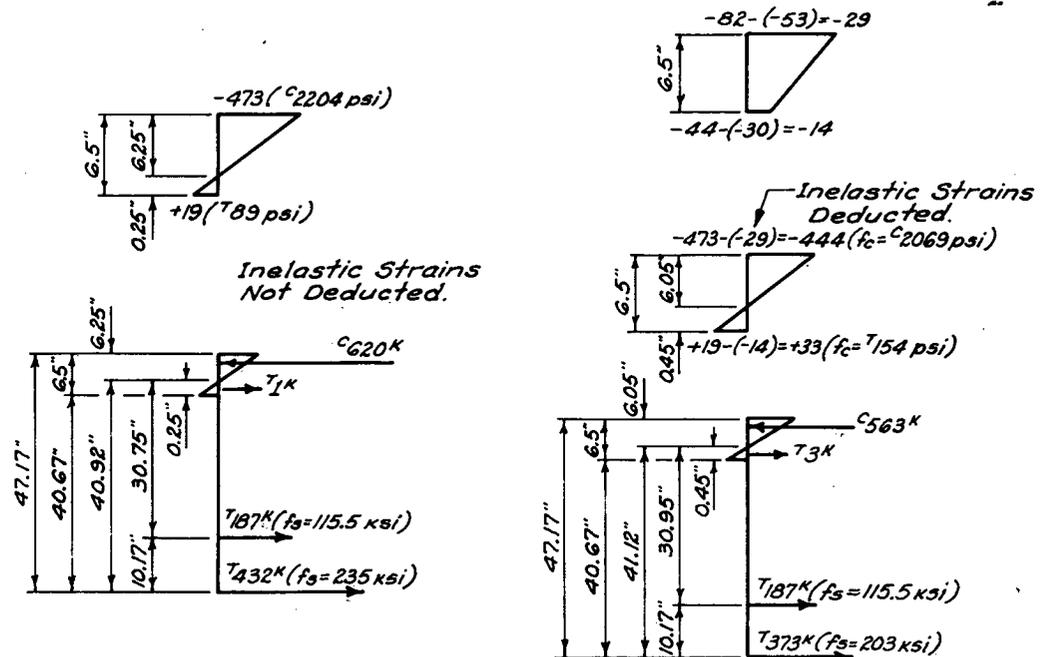


Figure A-47. Test No. 8 stresses due to strains.

- $a = (51.5 - 50.69)(2) = 1.62$ in.
 - $f_c = \frac{488.4}{(1.62)(90)} = 3.35$ ksi, f_c allowable = $(0.85)(4,000) = 3.4$ ksi.
 - For 12 strands plus two 1-in. diameter thread bars, assume $a = 2.4$ in.
 - Max M_u bars = $\frac{(A \text{ bars})(f'_s)(d)}{12} = \frac{(2)(0.85)(150)(51.5 - 1.2 - 14.5)}{12} = 761$ ft-kip.
 - $P_{ultimate} = (150)(0.85)(2) + 488 = 743$ kips.
 - $f_c = \frac{743}{(2.4)(90)} = 3.44$ ksi; f_c allowable = $(0.85)(4,000) = 3.4$ ksi.
 - Revised M_u strands = $\frac{(488)(51.5 - 4.33 - (2.4/2))}{12} = 1,869$ ft-kip.
 - M_u strands + bars = $1,869 + 761 = 2,630$ ft-kip.
 - Test moment = $(2,630)(0.75) - DLM = 1,973 - 554 = 1,419$ ft-kip.
 - $P = \frac{4M}{\ell} = \frac{(4)(1,419)}{60} = 94.6$ kips (0.9 F_y in bars may govern).
2. Compute allowable test load (Fig. A-45):
 - f_c bottom due to post-tensioning = 409 psi (c).
 - f_t bottom due to LL + I = 845 psi (t).
 - f_t bottom in patch area = $845 - 409 = 436$ psi (t).
 - AASHTO allowable = $6\sqrt{5,000} = 424$ psi (t), and since $424 \approx 436$, do not preload this test.
 - Working stress per bar = $0.6 f'_s = (0.6)(150) = 90$ ksi.
 - Bending stress in bars due to bar weight:
 - Wt. = 3.01 lb/ft. Supports at 12-ft centers.
 3. Verify measured deflection due to Test Load No. 8 (see Fig. A-46):
 - E_c girder = $5.051,000$ in.⁴ (same as Test 9); P test = 89.7 kips; $R = 44.85$ kips.
- $M = \frac{wl^2}{10} = 43.3$ lb-ft; $S = 0.0982$ in.³ $f = \frac{M}{S} = 5.5$ ksi.
 - Stress in bars will probably control. Allowable = 0.9 yield = $(0.9)(0.85)(150) = 114.8$ ksi.
 - Maximum load in bars = $(2)(0.85)(114.8) = 195$ kips.
 - Allowable increase in bar stress = $114.8 - (90 - 5.5) = 30.3$ ksi.
 - $\frac{f_s}{n} = \frac{30,300}{7.0} = 4,329$ psi. Used to establish concrete stress block. $\frac{4,329}{28} = \frac{f_c}{9.0}$; $f_c = 1,391$ psi.
 - $f_{sT_1} = 144 + \frac{(30.3)(38.17)}{28} = 185$ ksi.
 - $T_1 = (185)(12)(0.153) = 340$ kips; $T_1 + T_2 = 195 + 340 = 535$ kips.
 - $C = \frac{(1.391 + 0.386)}{2} (6.5)(90) = 520$ kips ($520 \approx 535$ kips).
 - Total $M = \frac{(340)(38.17)}{12} + \frac{(195)(28)}{12} + \frac{(0.386)(6.5)(90)(5.75)}{12} + \frac{(1.005)(6.5)(90)(6.83)}{(2)(12)} = 1,811$ ft-kip.
 - Allowable test load $P = \frac{(4)(1,811 - 554)}{60} = 84$ kips.
 - The maximum test load was 89.7 kips. See previous comments regarding jacking bars to 89 kips each, and increasing the test load to 89.7 kips.

- M at 24.5 ft = $(44.85)(24.5)(12) = 13,186$ in.-kip.
- $R\ell = 0.7166; \frac{M}{I}$ at centerline span = 14.13.
- $\Delta = \frac{M}{EI} = \frac{(14.13)(1,000)(144)}{5,051,000} = 0.40$ in.
- Summation cracks = $(6)(0.1) = 0.5 + 0.6 = 1.7$ mm = 0.0669 in.
- Δ girder = $0.40 + \frac{(12)(30)(0.669)}{(38)(2)} = 0.40 + 0.32 = 0.72$ in.
- Measured deflection = 0.89 in. (First cycle); = 0.89 in. (Second cycle); = 0.89 in. (Third cycle).
- The observed cracking load for Test No. 8 was 49.7 kips, identical to the cracking load for Test 5. This confirmed that preload was not needed for Test 8.

4. Compute prestress loss at 64 days:

- CR_c (creep) = $\frac{64^{0.6}}{10 + 64^{0.6}} (11,310) = (0.55)(11,310) = 6,221$ psi.
- SH (shrinkage) = $\frac{(5,000)(64)}{(55 + 64)} = 2,700$ psi.
- ES (elastic shortening) = 8,340 psi.
- CR_s (steel relaxation loss) = 13,400 psi.
- Total loss for Test 8 = 30.7 ksi.

5. Calculate stresses from strains at span (last cycle):

- Inelastic strains in top slab not deducted (Fig. A-47):
 - E_c slab = 4,660,000 in.⁴
 - f top of slab = $(0.000473)(4,660,000) = 2,204$ psi (c).
 - f bottom of slab = $(0.000019)(4,660,000) = 89$ psi (t).
 - $F_c = (2.204) \frac{(6.25)(90)}{2} = 620$ kips.
 - $F_T = \frac{(0.089)(0.25)(90)}{2} = 1$ kip.
 - The forces in the top slab and the thread bars are developed from recorded strains. The force in the strands is developed from $\Sigma H = 0$.
 - $T_2 = (0.003740)(29,400)(2)(0.85) = 187$ kips.
 - $f_s = \frac{187}{(2)(0.85)} + 5.5 = 110 + 5.5$ (bar bending) = 115.5 ksi. Allowable $f_s = 0.9 f_y = 114.8$ ksi.
 - $T_1 = 620 - 187 = 432$ kips; $f_s = \frac{432}{(12)(1.153)} = 235$ ksi.
 - $M = \frac{(432)(40.92)}{12} + \frac{(187)(30.75)}{12} + \frac{(620)(2)(6.25)}{(3)(12)} = 2,167$ ft-kip.
 - Applied $M = 1,900$ ft-kip.
 - Ratio $\frac{\text{Measured } M}{\text{Applied } M} = \frac{2,167}{1,900} = 1.14$.
- Inelastic strains in top slab deducted (Fig. A-47):
 - E_c slab = 4,660,000 in.⁴
 - f top of slab = $(0.000082)(4,660,000) = 382$ psi (c).
 - From Figure A-20, $e_s = 127; \frac{(82)(82)}{127} = 53$ psi.
 - f bottom of slab = $(0.0000444)(4,660,000) = 205$ psi (c).
 - From Figure A-20, $e_s = 65, \frac{(44)(44)}{65} = 30$ psi.

- f top of slab = $(0.000444)(4,660,000) = 2,609$ psi (c).
- f bottom of slab = $(0.000033)(4,660,000) = 154$ psi (t).
- $F_c = \frac{(2.069)(6.05)(90)}{2} = 563$ kips (C).
- $F_T = \frac{(0.154)(0.45)(90)}{2} = 3$ kips (T).
- $T_2 = (0.003740)(29,400)(2)(0.85) = 187$ kips.
- $f_s = \frac{187}{(2)(0.85)} + 5.5 = 115.5$ ksi.
- $T_1 = 563 - 3 - 187 = 373$ kips; $f_s = 373 / (12)(0.153) = 203$ ksi.
- M measured = $\frac{(373)(41.12)}{12} + \frac{(187)(30.95)}{12} + \frac{(563)(2)(6.05)}{(3)(12)} = 1,949$ ft-kip.
- M applied = 1,900 ft-kip.
- Ratio $\frac{\text{Measured } M}{\text{Applied } M} = \frac{1,949}{1,900} = 1.03$.

6. Check stresses from strand gages 110 and 111:

- Strand gage 110:
 - $(2,884 - 1,449) = (0.001435)(28,000) = 40.2$ ksi.
 - $f_s = 189 - 30.7$ (P.S. loss) + 40.2 = 199 ksi.
- Strand gage 111:
 - $(2,413 - 594) = (0.001819)(28,000) = 50.9$ ksi.
 - $f_s = 189 - 30.7$ (P.S. loss) + 50.9 = 209 ksi.
 - $f_s = 203 \cong 199 \cong 209$ ksi.

The load-deflection and load-strain curves for Test 8 are shown in Figure A-48. The first and second cycle deflection curves show the close corroboration obtained between cycles. The third cycle (last cycle) was nearly identical to the second cycle. All strain gage curves define the point at which the cracking load (approximately 50 kips) causes rapid increase in strains. Referring to Figure 3 in Chapter Two, it is seen that the two thread bars restored the strength of the four severed strands. By comparing Test 8 to Test 1, the increased stiffness is apparent.

Load Test 9

Test 9 had 6 out of 16 strands severed (see Fig. A-49), and the severed strands were spliced with the metal sleeve splice. The sleeve was bonded to the girder with injected epoxy resin. The strand-sleeve lap was 3 ft 3 in. in solid concrete. This lap length was based on tension tests of the Anderson Pile Sleeve Splice, and would permit a maximum stress in the strands of 270 ksi. The sleeve was 13 ft long, placed symmetrically about centerline of span (see Fig. A-50). Details of the sleeve, including installation, are presented in Chapter Two. The sleeve was over-designed because of the $\frac{5}{16}$ -in. minimum metal thickness used. Concrete removed to sever two additional strands (four strands had been severed in Test 7) was patched. The ultimate moment capacity of the 16-strand girder was 2,511 ft-kip. As determined in Test 1, 75 percent of 2,511 ft-kip equals 1,883 ft-kip. A maximum test load of 92.0 kips was applied. The total applied test moment including dead load moment was 1,934 ft-kip, equaling 103 percent of 1,883 ft-kip. The test load moment of 1,380 ft-kip (not including DLM) was equivalent to 1.98 times

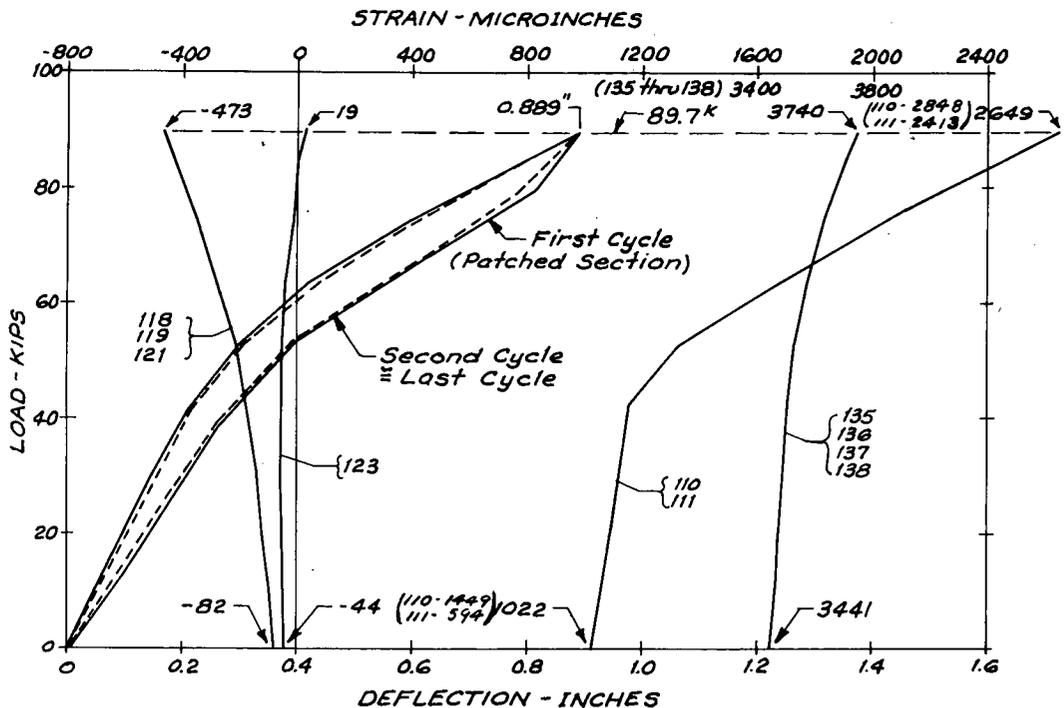


Figure A-48. Test No. 8 first and last cycle deflections and last cycle strains.

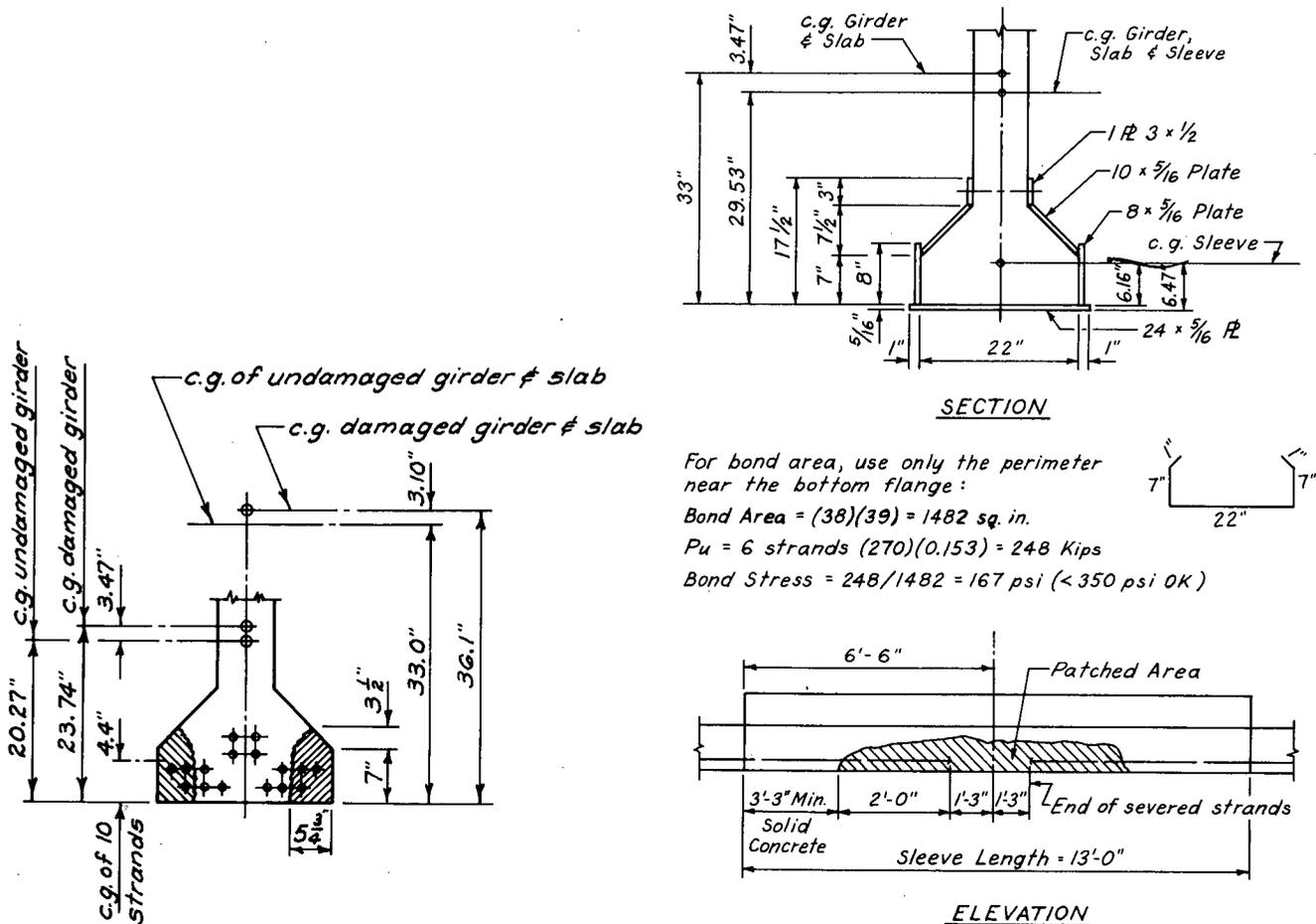


Figure A-49. Six strands severed.

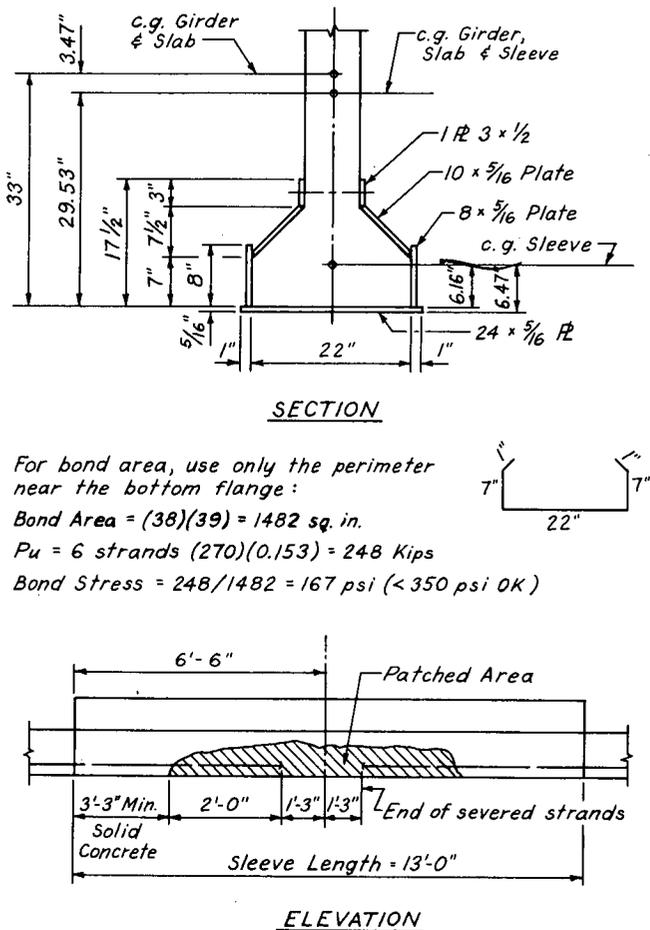


Figure A-50. Test No. 9 sleeve splice.

the HS-20 LL + I service load moment of 697 ft-kip. At time of testing the girder concrete strength, f'_c , was 6,150 psi and the slab concrete strength was 5,410 psi. Test 9 occurred 80 days after casting the girder and 76 days after release of strands. The slab was cast 66 days prior to Test 9. The computed 76-day prestress loss was 31.1 ksi.

The first observed cracks opened at the ends of the splice sleeve at a load of 73 kips or moment of 1,095 ft-kip equivalent to 1.57 times HS-20 LL + I M of 697 ft-kip. At maximum load, nine cracks were observed to extend within a few inches of the top slab. Four right-hand cracks were at previous crack locations. One of these cracks was at the east end of the sleeve. Four of the left-hand cracks were at previous crack locations; in addition, a minor crack at the west end of the sleeve developed into a crack that extended upward. The maximum crack width was 0.4 mm at the left end of the sleeve. Other than at the sleeve ends, major cracks did not occur outside the sleeve length. Strain data indicated compression at the bottom of the slab, which was a departure from previous tests. The maximum vertical deflection caused by the test was 0.64 in. The computed deflection was 0.77 in.

Maximum moments at centerline of span were developed from strain data and compared to the applied moments (see Table A-4). Strand stresses ranged from 41 percent to 63 percent of f'_s (see Table A-6). Maximum sleeve stresses were 14.2 ksi for the first load cycle and 14.8 ksi for the last load cycle (39 percent F_y and 41 percent F_u , where F_y equals 36 ksi). There was no evidence of slippage between the sleeve and the girder. The following calculations show the methodology used (also refer to Figs. A-49, A-50, A-51, A-52, and A-53 when reviewing the calculations):

1. Given:

- a. Test No. 9 girder with six severed strands plus steel sleeve.
- b. Computations prior to testing:
 - Break out portion of concrete and sever 6 strands.
 - c.g. of 10 strands = $(4)(2) + (2)(4) + (4)(7)/10 = 4.40$ in. (Fig. A-49)
 - Section properties nondamaged girder: $A = 559.5$ in.², $I = 125,400$ in.⁴; S bottom = 6,190 in.³
 - Section properties girder and slab: $A = 1,027$ in.², $I = 326,600$ in.⁴, S bottom = 9,895 in.³
 - Section properties damaged girder: $A = 458.9$ in.²; new c.g. = 3.47 in.
 $I = I_{orig.} + (A_{orig.})(d^2) - I_o$ (deduction) - AD^2
 $I = 94,297$ in.⁴, S bottom = $\frac{94,297}{23.74} = 3,972$ in.³
 - Section properties damaged girder plus slab: $A = 926.4$ in.²; new c.g. = 3.10 in.
 $I = I_{orig.} + (A_{orig.})(d^2) - I_o$ (deduction) - AD^2
 $I = 235,239$ in.⁴, S bottom = $\frac{235,239}{36.10} = 6,516$ in.³
 - f bottom from DL + LL + I + 16 strands (prior to damage) = 365 psi (t) (Allowable = 424 psi (t).)

2. Calculate stresses at bottom of damaged girder:

- a. Assume that prestress plus dead load of girder and slab acts on the damaged girder section alone:
 - f_c bottom P.S. = $\frac{(10)(222)}{458.9} + \frac{(10)(22)(19.34)}{3,972} = 1,550$ P.S. (c).
 - f_t bottom = $\frac{(554)(12)}{3,972} = 1,674$ D.L. (t).

- f bottom from P.S. + DL = $1,674 - 1,550 = 124$ psi (t).

b. Assume that prestress plus dead load of girder and slab acts on the composite damaged section:

- f_c bottom = $\frac{(10)(22)}{926.4} + \frac{(10)(22)(36.10 - 4.40)}{6,516} = 1,307$ P.S. (c).
- f_t bottom = $\frac{554}{(12)(6,516)} = 1,020$ DL (t).
- f bottom from P.S. + DL = $1,307 - 1,020 = 287$ psi (c).
- f_c allowable = $0.4 f'_c = (0.4)(5,000) = 2,000$ psi.
- $f_t = 124$ psi (t) is not considered excessive.

3. Design calculations for steel sleeve splice (Fig. A-50):

a. Given:

- $n = 7.0$; $E_c = \frac{29,000,000}{7} = 4,142,857$ psi; A_c sleeve = $(7)(21.75) = 152.25$ in.²; c.g. sleeve = 6.16 in.; I_o sleeve = $(7)(796) = 5,572$ in.⁴
- Section properties nondamaged girder, slab, and sleeve:
 - $A = 1,027 + 152 = 1,179$ in.²; new c.g. = 3.47 in.
 - New $I = I_{orig.} + A_{orig.}(d^2) + I_o$ (sleeve) + AD^2 (sleeve) = 427,700 in.⁴
 - S bottom of sleeve = $\frac{427,700}{29.84} = 14,330$ in.³
 - f_t (LL + I) = $(697)(12)(7)/14,330 = 4.1$ ksi steel stress (t).

b. Check stresses at top of sleeve, assuming P.S. + DL act on damaged girder section only:

- f_c P.S. = $(10)(22)/458.9 + (10)(22)(19.34)(23.74 - 17.5)/94,297 = 761$ psi (c).
- f_t DL = $(554)(12)(6.24)/94,297 = 440$ (t).
- f_t LL + I = $(697)(12)(29.53 - 17.5)/427,700 = 235$ (t)
- $f_c = 761$ compression - 440 tension - 235 tension = 86 psi (c).

c. Assuming P.S. + DL acts on damaged composite section:

- f_c P.S. = $(10)(22)/926.4 + (10)(22)(18.6)(36.1 - 4.4)/235,239 = 789$ psi (c).
- f_t DL = $(554)(12)(18.6)/235,239 = 526$ (t)
- f_t LL + I = (same as above)
- $f_c = 789$ compression - 526 tension - 235 tension = 28 psi (c).
- Therefore preload is not required or desirable.

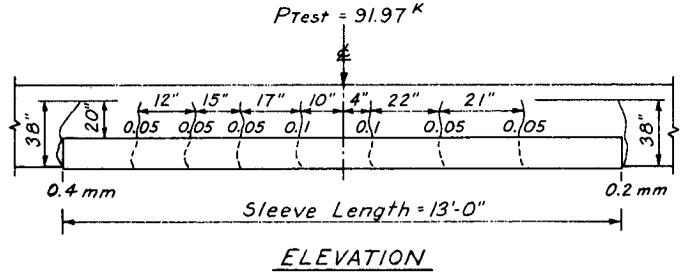
4. Calculate required test load:

- Calculated M_u of girder = 2,511 ft-kip (16 strands w/o sleeve).
- 75% $M_u = 1,883$ ft-kip, LL M = 1,883 - 554 (DLM) = 1,329 ft-kip.
- $P = 4M/\ell = (4)(1,329)/60 = 89$ kips (same as Test 1).
- 91.97 kips was used to facilitate load readings.

5. Check approximate sleeve stress using section properties of nondamaged girder plus slab and steel sleeve:

- $M = 75\% M_u = 1,883$ ft-kip.
- f_t bottom = $(1,883)(12)(7)/14,330 = 11.0$ ksi (allowable = 20 ksi).
- f_c 10 strand P.S. at top of sleeve = 761 psi (c).

- f_t for DLM at top of sleeve = 440 (t).
- Additional Moment = 1,883 - 554 = 1,329 ft-kip.
- $f_t = M_c/I = (1,329)(12)(12.03)/427,700 = 449$ (t).
- $f_t = \text{total} = 440 + 449 - 761 = 128$ psi (t).
- If girder had not been previously cracked, section would not crack with a tension of 128 psi. Therefore, six strands can be severed. The epoxy resin pressure was determined to be nominal and no stiffening of plates is necessary.



6. Verify measured deflection due to test load 9 (see Fig. A-51, computations following testing):

- E_c girder = $(155^{3/2})(33) \sqrt{6,290} = 5,501,000 \text{ in.}^4$
- Summation cracks = $0.4 + 0.2/25.4 = 0.0236 \text{ in.}$ and $[(5)(0.05) + (2)(0.1)]/25.4 = 0.0177 \text{ in.}$
- Δ girder from cracks = $(12)(30)(0.0236)/(38)(2) + (12)(30)(0.0177)/(20)(2) = 0.112 + 0.159 = 0.271 \text{ in.}$
- Δ girder from $\frac{M}{EI}$ diagram; $R\ell = 0.79$, M/I at centerline = 16.10.
- Δ girder = $(16.10)(1,000)(144)/5,051,000 = 0.46 \text{ in.}$
- Δ girder total = $0.271 + 0.46 = 0.73 \text{ in.}$
- Measured deflection = 0.61 in. (First cycle) = 0.60 in. (Second cycle) = 0.64 in. (Last cycle).
- Note that the strains in gages 112 and 113 increase from zero load to maximum in a relatively uniform curve. It is believed that the total elastic deflection plus deflection due to major cracks gives the most realistic computed deflection. Cracks are assumed to be concentrated at centerline span, same as previous tests.

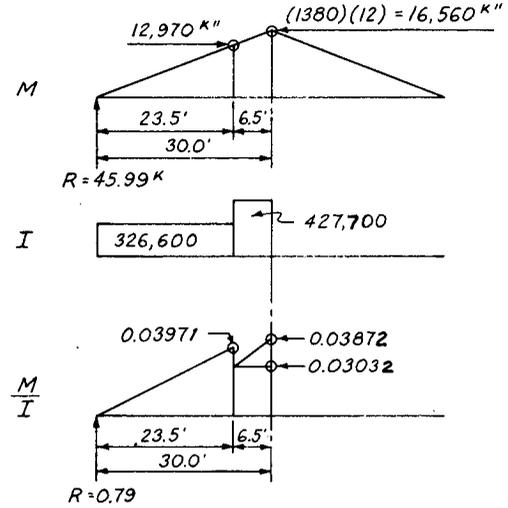


Figure A-51. Test No. 9 calculated deflections.

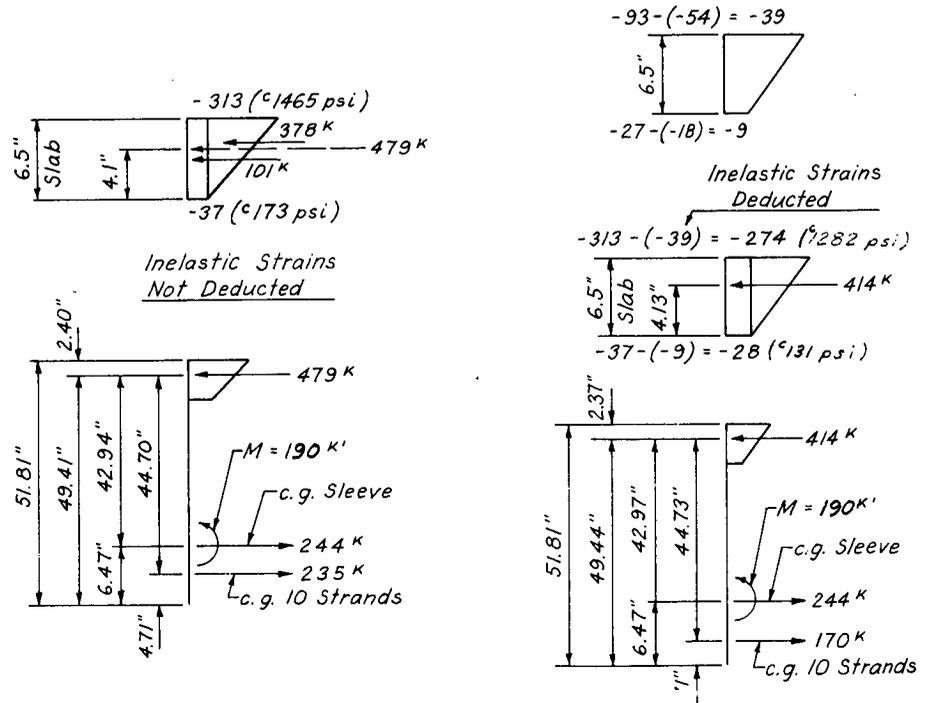


Figure A-52. Test No. 9 stresses due to strains.

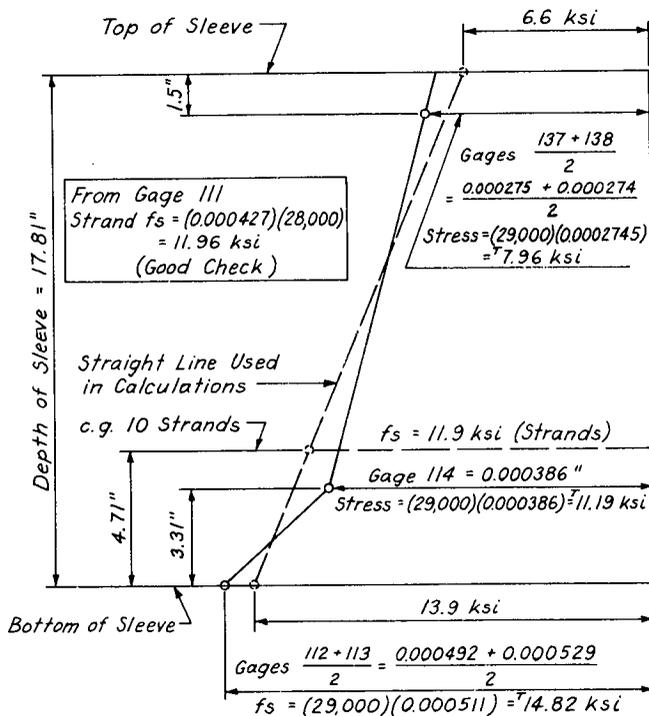


Figure A-53. Test No. 9 stresses in sleeve due to strains.

7. Calculate prestress loss at 76 days:

- C_r (creep) = $(11,310)(76^{0.6})/10 + 76^{0.6} = (11,310)(0.57) = 6,447$ psi
- SH (shrinkage) = $(5,000)(76)/55 + 76 = 2,900$ psi.
- ES (elastic shortening) = 8,340 psi.
- CR_s (steel relaxation) = 13,400 psi.
- Total loss for Test 9 = 31.1 ksi.

8. Calculate stresses from strains at centerline span (last cycle):

- a. Inelastic strains in top slab not deducted, Figure A-52.
- E_c slab = 4,680,000 in.⁴
 - f top of slab = $(0.000276)(4,680,000) = 1,292$ psi (c).
 - f bottom of slab = $(0.000037)(4,680,000) = 173$ psi (c).
 - $F_c = (1.292)(6.5)(90)/2 = 378$ kips (C).
 - $F_c = (0.173)(6.5)(90) = 101$ kips (C).
 - Centroid of forces = $[(378)(2)(6.5)/3 + (101)(6.5)/2]/479 = 4.10$ in.
 - Calculate moment in sleeve, Figure A-53.
 - f bottom = $P/A + M_{y,bot}/I$, f top = $P/A - M_{y,top}/I$
 - $13.9 = \frac{P}{21.75} + \frac{(M)(6.47)}{5,572}$; $6.6 = \frac{P}{21.75} - \frac{(M)(11.34)}{5,572}$
 - $13.9 = 0.0460 P + 0.00116 M$
 - $6.6 = 0.0460 P - 0.00204 M$
 - $24.44 = 0.0809 P + 0.00204 M$
 - $31.04 = 0.1269 P$

- $P = 244.6$ kips.
- $0.00204 M = (0.0460)(244.6) - 6.6$.

- $M = 2,280$ in.-kip = 190 ft-kip.
- T strands = 479 - 244 = 235 kips.
- $f_s = 235/(10)(0.153) = 154$ ksi.
- Measured $M = (235)(44.70)/12 + (244)(42.94)/12 + 190 = 1,938$ ft-kip.
- Applied $M = 1,934$ ft-kip.
- Ratio $\frac{\text{Measured } M}{\text{Applied } M} = \frac{1,938}{1,934} = 1.00$.
- Change in strand stress from sleeve strains = 11.9 ksi.
- $f_s = 189 - 31.1 + 11.9 = 170$ ksi > 154 ksi.
- Gage 110 is reading "Data over Range".
- Change in strand stress from gage 111 = 12 ksi (checks 11.9).
- $f_s = 189 - 31.1 + 12 = 170$ ksi > 154 ksi.
- b. Inelastic strains in top slab deducted, Figure A-52:
 - f top of slab = $(0.000093)(4,680,000) = 435$ psi.
 - From Figure A-20, $e_s = 159$, $(93/159)(93) = 54$.
 - f bottom of slab = $(0.000027)(4,680,000) = 126$ psi.
 - From Figure A-20, $e_s = 40$, $(27/40)(27) = 18$.
 - f top of slab = $(0.000274)(4,680,000) = 1,282$ psi (c).
 - f bottom of slab = $(0.000028)(4,680,000) = 131$ psi (c).
 - $F_c = (1.115)(6.5)(90)/2 = 337$ kips (C).
 - $F_c = (0.131)(6.5)(90) = 77$ kips (C).
 - Centroid of forces = $[(337)(6.5)(2/3) + (77)(6.5)/2]/414 = 4.13$ in.
 - T strands = 414 - 244 = 170 kips.
 - $f_s = 170/(10)(0.153) = 111$ ksi (low).
 - $M = (170)(44.73)/12 + (244)(42.97)/12 + 190 = 1,698$ ft-kip.
 - Applied $M = 1,934$ ft-kip.
 - Ratio $\frac{\text{Measured } M}{\text{Applied } M} = \frac{1,698}{1,934} = 0.88$.
 - Change in strand stress from sleeve strains = 11.9 ksi.
 - $f_s = 189 - 31.1 + 11.9 = 170$ ksi > 111 ksi.
 - Develop strand stress from change in gage 111 = $f_s = (0.001544 - 0.001117)(28,000) = 12$ ksi.
 - $f_s = 189 - 31.1$ (P.S. loss) + 12.0 = 170 ksi > 111 ksi.

Figure A-54 shows the load-deflection and load-strain curves for Test 9. The strand strain measured by gage 111 is, for practical purposes, a straight line and nearly parallel to the strain measured by gages 114 and 115. This corroborates the method used to develop strand stress. The top slab strain, gages 118, 119, and 121, is nearly uniform throughout the loading cycle, and the deflection is approximately one-half the deflection of the undamaged girder in Test 1.

Load Test 10

Test 10 was a test of the same sleeve splice as Test 9, except the test load was increased to a maximum of 166.8 kips. The maximum capacity of the hydraulic jack was 166.8 kips. Only one test cycle was made. The ultimate moment capacity of the 16-strand girder was 2,511 ft-kip, as determined in Test 1. The critical sections were at the ends of the sleeve splice, 6 ft 6 in. from centerline of span. The applied moment at the sleeve ends was 2,488 ft-kip, including dead load moment, equaling 99 percent of the calculated ultimate strength of the girder. The total applied moment at centerline of span was 3,056 ft-kip. The test load moment of 2,502 ft-kip (not including DLM) at centerline of span was equivalent to 3.59 times the HS-20 LL + I service load moment of 697 ft-kip. The test load moment at the ends of the sleeve was 1,960 ft-kip (not including DLM) and was equivalent to 2.81 times the HS-20 LL + I service load moment of 697 ft-kip. Test No. 10 was made the same day as Test 9; thence concrete strengths and loss of prestress were the same as Test 9.

Nineteen major cracks were observed to extend to the bottom of the slab or within a few inches thereof. Seven of these cracks occurred above the top of the 13-ft long sleeve. The cracks occurred along a girder length of 26 ft, 13 ft each side centerline of span. The maximum crack width was 6.4 mm, occurring at a sleeve end; this crack reduced to 3.2 mm after release of the test load. Strain data indicated tension at the bottom of the slab. The compression stress at top of slab, centerline of span, computed from strains was 3,005 psi, equaling 0.56 times the ultimate strength of the slab. This stress was the highest recorded stress for any test. The maximum vertical deflection was 5.28 in., equaling 4.47 times the maximum Test 1 deflection. The residual deflection after release of load was 1.48 in. Maximum moments at centerline of span were developed from strain data and compared to the applied moments (see Table A-4). Strand stresses ranged from 57 percent to 67 percent of f'_s (see Table A-6). Maximum sleeve stress was 31.3 ksi, equal to 87 percent of $F_y = 36$ ksi. No visible evidence of slippage between the sleeve and the girder occurred. Bond stress on the sleeve was calculated as follows. The only part of the perimeter of the sleeve used was the width of bottom flange (22 in.) plus the depth of bottom flange (2 times 7 in. = 14 in.) plus 1 in. of each fillet, giving a perimeter of 38 in. The ultimate strength for six severed strands was $6(0.153)(270) = 248$ kips. Using a bond length of 39 in. (see Fig. A-35), the bond stress was $248 / (38)(39) = 167$ psi. The methodology of developing moments and stresses from strains is shown in Test 9.

The load-deflection and load-strain curves for Test 10 are shown in Figure A-55. At a load of 106 kips a crack formed beyond one end of the steel sleeve splice. The applied load reduced to about 101 kips when this crack formed. The load held steady at this point. Following inspection, loading was continued, as shown in Figure A-55. From about 140 kips a rapid increase occurred in deflection. All strain gages indicated a redistribution of load between approximately 130 to 150 kips. This redistribution started at the bottom of the girder, working up to the top slab. Loading ceased at 167 kips, which was about equal to the safe capacity of the jacking floor system. There was a permanent set of 1.48 in. when load was removed.

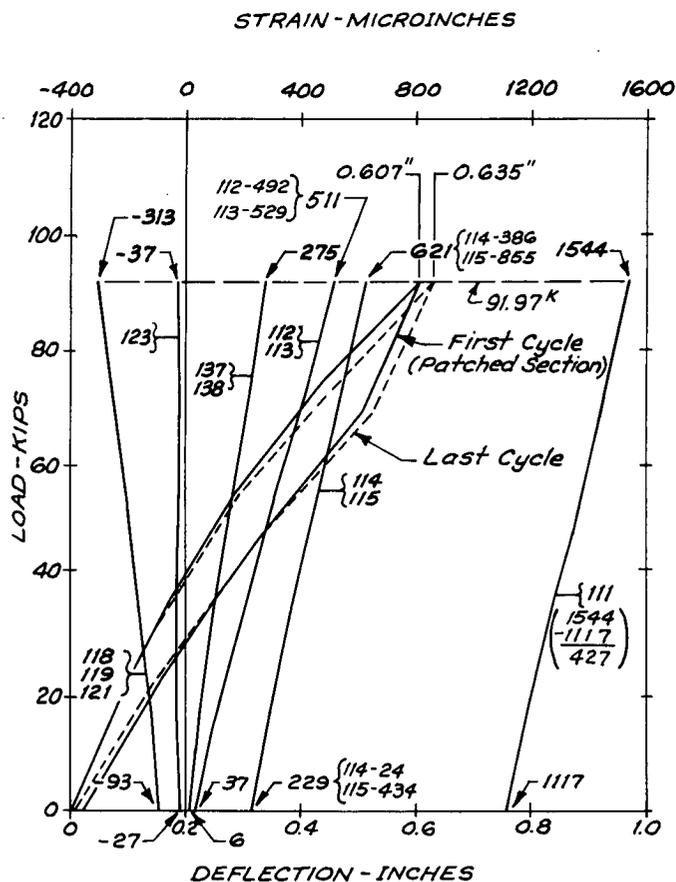


Figure A-54. Test No. 9 first and last cycle deflections and last cycle strains.

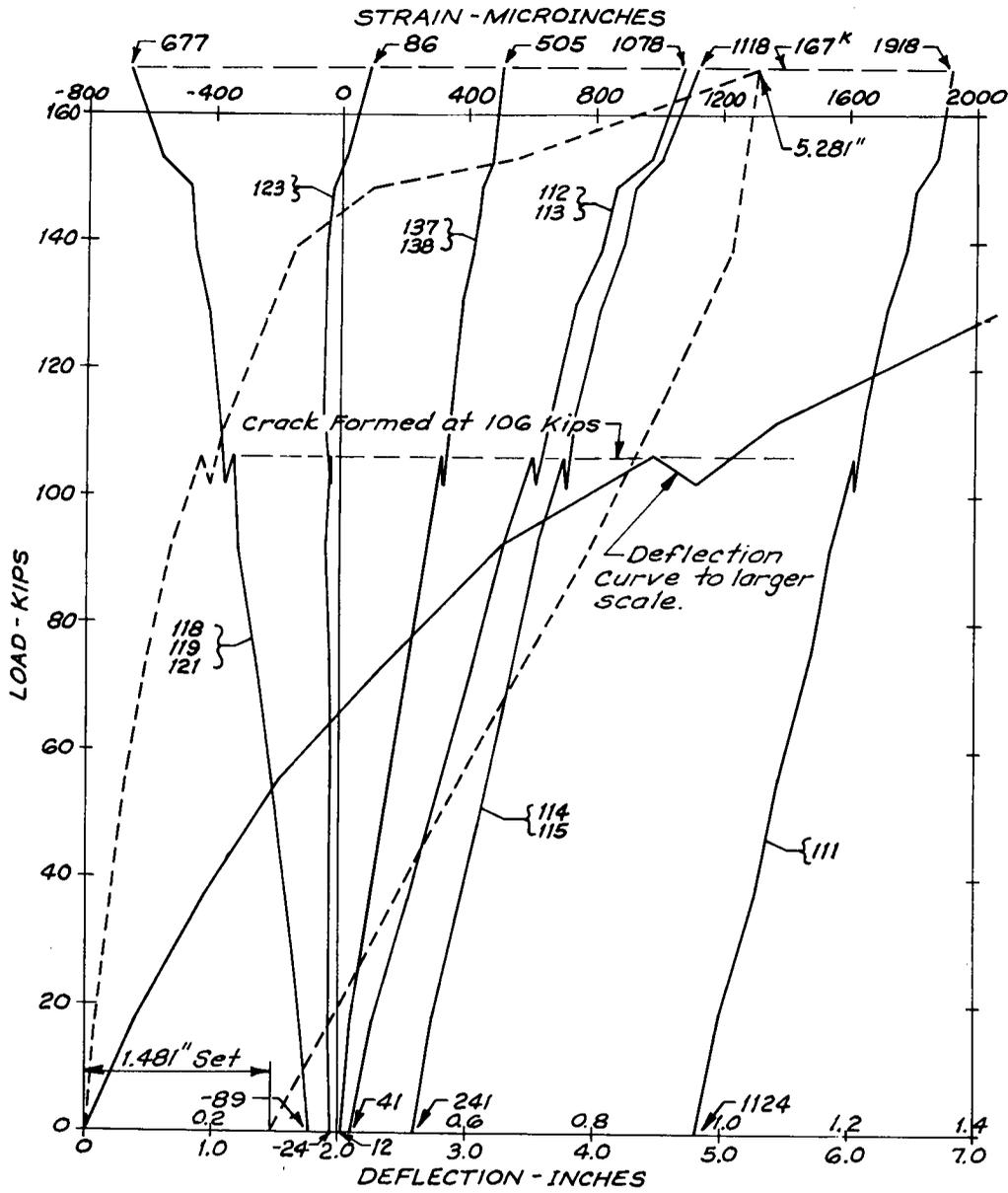


Figure A-55. Test No. 10 deflection and strains.

APPENDIX B

NOTATIONS AND DEFINITIONS

a = depth of equivalent compression zone	F_u = minimum tensile strength
A = area	F_y = minimum yield point or yield strength of steel
A_{gross} = gross area	F_T = tension force
A_{net} = net area	F.S. = factor of safety
A_s = area of nonprestressed tension reinforcement	I = impact load
A_s^* = area of prestressing steel	I = moment of inertia about the centroid of the section
A_v = area of shear reinforcement	k' = foot-kip
a_{vf} = area of shear-friction reinforcement	kd' = distance from extreme compression fiber to neutral axis
b = width of flange or width of rectangular member	ksi = kips per square inch
bot = bottom	K = kip (1000 pounds)
c = compression	ℓ = length
C = compressive force	LL = live load
c.g. = center of gravity	M = moment
d = distance from extreme compression fiber to centroid of the prestressing force	M_u = ultimate load moment
d' = distance from extreme compression fiber to bottom layer to prestressing elements	n = ratio E_s/E_c
D = nominal diameter	p^* = A_s^*/bd , ratio of prestressing steel
DL = dead load	psi = pounds per square inch
E_c = modulus of elasticity concrete	P = load
E_s = modulus of elasticity steel	P_u = ultimate load
f = stress	P.S. = prestress
f_c = concrete compressive stress	R = load reaction
f'_c = concrete compressive stress at 28 days	S = section modulus; also girder spacing
f_{cgs} = concrete stress at the center of gravity of the prestressing steel from all dead loads except the dead load present at the time the prestressing force is applied	t = tension
f'_{ci} = concrete compressive stress at time of initial prestress	t = time in days after release of strands
f_s = steel stress	T = tension force
f'_s = ultimate strength of prestressing steel	v_u = factored shear stress
f_{se} = effective steel prestress after losses	V_u = ultimate load shear
f^*_{su} = average stress in prestressing steel at ultimate load	w = weight or load per foot
f_t = concrete tensile stress	X = constant in modulus of rupture formula $X\sqrt{f'_c}$
F_c = compressive force	y = distance from c.g. to extreme fiber
	Δ = elongation or deflection
	ϕ = strength reduction factor [AASHTO Article 1.5.30(B)]
	μ = coefficient of friction [AASHTO Article 1.5.35(D)]
	ΣH = summation of horizontal forces

APPENDIX C

REFERENCES

1. American Association of State Highway and Transportation Officials, *Standard Specifications for Highway Bridges*. 12th Edition (1977) 496 pp.
2. CASSANO, R. C., and LEBEAU, R. J., "Correlating Bridge Design Practice with Overload Permit Policy." *Transportation Research Record 664* (1978) pp. 230-238.
3. TIDE, R. H. R., and VAN HORN, D. A., "A Statistical Study of the Static and Fatigue Properties of High Strength Prestressing Strand." *Fritz Engineering Laboratory Report No. 309.2*, Lehigh University (June 1966).
4. RABBAT, B. G., KAAR, P. H., RUSSELL, H. G., and BRUCE, R. N., JR., "Fatigue Tests of Prestressed Girders with Blanketed and Draped Strands." *Transportation Research Record 665* (1978) pp. 13-21.
5. RABBAT, B. G., KAAR, P. H., RUSSELL, H. G., and BRUCE, R. N., JR., "Fatigue Tests of Full-Sized Prestressed Girders." Portland Cement Association, State of Louisiana, *Technical Report No. 113* (June 1978).
6. ABELES, P. W., and BARTON, F. W., "Fatigue Test on Damaged Prestressed Concrete Beams." Duke University, Int. Symp. on Effects of Repeated Loading of Materials and Structures (Sept. 6, 1966).
7. WARNER, R. E., and HULSBOS, C. L., "Fatigue Properties of Prestressing Strand." *PCI Journal* (Feb. 1966).
8. WARNER, R. E., and HULSBOS, C. L., "Probable Fatigue Life of Prestress Concrete Beams." *PCI Journal*, Vol. II, No. 2 (Apr. 1966).
9. HANSON, J. M., HULSBOS, C. L., and VAN HORN, D. A., "Fatigue Tests of Prestressed Concrete I-Beams." *J. of the Structural Division*, Proceedings of the American Society of Civil Engineers (Nov. 1970).
10. "Cracked Structural Concrete Repair Through Epoxy Injection and Rebar Insertion." Kansas Department of Transportation, *Report No. FHWA-KS-RD 76-2*, Interim Report (1977).
11. "Repair of Hollow or Softened Areas in Bridge Decks by Rebonding with Injected Epoxy Resin or Other Polymers." State Highway Commission of Kansas, *Report No. K-F-72-5*, Final Report (1974).
12. SHANAFELT, G. O., and HORN, W. B., "Damage Evaluation and Repair Methods for Prestressed Concrete Bridge Members." *NCHRP Report 226* (Nov. 1980) 66 pp.
13. DUNHAM, C. W., "The Theory and Practice of Reinforced Concrete." McGraw Hill (1944).

THE TRANSPORTATION RESEARCH BOARD is an agency of the National Research Council, which serves the National Academy of Sciences and the National Academy of Engineering. The Board's purpose is to stimulate research concerning the nature and performance of transportation systems, to disseminate information that the research produces, and to encourage the application of appropriate research findings. The Board's program is carried out by more than 270 committees, task forces, and panels composed of more than 3,300 administrators, engineers, social scientists, attorneys, educators, and others concerned with transportation; they serve without compensation. The program is supported by state transportation and highway departments, the modal administrations of the U.S. Department of Transportation, the Association of American Railroads, the National Highway Traffic Safety Administration, and other organizations and individuals interested in the development of transportation.

The Transportation Research Board operates within the National Research Council. The National Research Council was established by the National Academy of Sciences in 1916 to associate the broad community of science and technology with the Academy's purposes of furthering knowledge and of advising the Federal Government. The Council operates in accordance with general policies determined by the Academy under the authority of its congressional charter of 1863, which establishes the Academy as a private, nonprofit, self-governing membership corporation. The Council has become the principal operating agency of both the National Academy of Sciences and the National Academy of Engineering in the conduct of their services to the government, the public, and the scientific and engineering communities. It is administered jointly by both Academies and the Institute of Medicine.

The National Academy of Sciences was established in 1863 by Act of Congress as a private, nonprofit, self-governing membership corporation for the furtherance of science and technology, and to advise the Federal Government upon request within its fields of competence. Under its corporate charter the Academy established the National Research Council in 1916, the National Academy of Engineering in 1964, and the Institute of Medicine in 1970.

TRANSPORTATION RESEARCH BOARD

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

ADDRESS CORRECTION REQUESTED

NON-PROFIT ORG.
U.S. POSTAGE
PAID
WASHINGTON, D.C.
PERMIT NO. 8970

R E C E I V E D

JUL 07 1986

MAT. LAB.

000015M001
JAMES W HILL
RESEARCH SUPERVISOR

IDAHO TRANS DEPT DIV OF HWYS
P O BOX 7129 3311 W STATE ST
BOISE ID 83707