TRANSPORTATION RESEARCH BOARD EXECUTIVE COMMITTEE 1989

Officers

Chairman
LOUIS J. GAMBACCINI, General Manager, Southeastern Pennsylvania Transportation Authority

Vice Chairman
WAYNE MURI, Chief Engineer, Missouri Highway & Transportation Department

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

Members

JAMES B. BUSEY IV, Federal Aviation Administrator-designate, U.S. Department of Transportation
BRIAN W. CLYMER, Urban Mass Transportation Administrator-designate, U.S. Department of Transportation
JERRY R. CURRY, National Highway Traffic Safety Administrator-designate, U.S. Department of Transportation
FRANCIS B. FRANCOIS, Executive Director, American Association of State Highway and Transportation Officials (ex officio)
JOHN GRAY, President, National Asphalt Pavement Association (ex officio)
THOMAS H. HANNA, President and Chief Executive Officer, Motor Vehicle Manufacturers Association of the United States, Inc. (ex officio)
HENRY J. HATCH, Chief of Engineers and Commander, U.S. Army Corps of Engineers (ex officio)
THOMAS D. LARSON, Federal Highway Administrator-designate, U.S. Department of Transportation
GEORGE H. WAY, JR., Vice President for Research and Test Department, Association of American Railroads (ex officio)
ROBERT J. AARONSON, President, Air Transport Association of America
ROBERT N. BOTHMAN, Director, Oregon Department of Transportation
J. RON BRINSON, President and Chief Executive Officer, Board of Commissioners of The Port of New Orleans
L. GARY BYRD, Consulting Engineer, Alexandria, Virginia
JOHN A. CLEMENTS, Vice President, Parsons Brinkerhoff Quade and Douglas, Inc. (past chairman, 1985)
SUSAN C. CRAMPTON, Secretary of Transportation, State of Vermont Agency of Transportation
L. STANLEY CRANE, Retired, Former Chairman & Chief Executive Officer, Consolidated Rail Corporation
RANDY DOI, Director, IVHS Systems, Motorola Incorporated
EARL DOVE, Chairman of the Board, AAA Cooper Transportation
WILLIAM J. HARRIS, E. R. Sneed Professor of Transportation Engineering & Distinguished Professor of Civil Engineering, Associate Director of Texas Transportation Institute, Texas A&M University System
LOWELL B. JACKSON, Vice President for Transportation, Greenhorne & O’Mara, Inc. (past chairman 1987)
DENMAN K. McNear, Vice Chairman, Rio Grande Industries
LENO MENGHINI, Superintendent and Chief Engineer, Wyoming Highway Department
WILLIAM W. MILLAR, Executive Director, Port Authority of Allegheny County
ROBERT E. PAAWELL, Professor of Transportation Engineering, Urban Transportation Center, University of Illinois at Chicago
RAY D. PETHTEL, Commissioner, Virginia Department of Transportation
JAMES P. PITZ, Director, Michigan Department of Transportation
HERBERT H. RICHARDSON, Deputy Chancellor and Dean of Engineering, Texas A&M University System (past chairman 1988)
JOE G. RIDEOUTTE, Executive Director, South Carolina Department of Highways and Public Transportation
TED TEDESCO, Vice President, Corporate Affairs, American Airlines, Inc.
CARMEN E. TURNER, General Manager, Washington Metropolitan Area Transit Authority
C. MICHAEL WALTON, Retired, Former Chairman & Chief Executive Officer, Consolidated Rail Corporation
IDES E. WHITE, Commissioner, New York State Department of Transportation
JULIAN WOLPERT, Henry G. Bryant Professor of Geography, Public Affairs and Urban Planning, Woodrow Wilson School of Public and International Affairs, Princeton University
PAUL ZIA, Distinguished University Professor, Department of Civil Engineering, North Carolina State University

NATIONAL COOPERATION HIGHWAY RESEARCH PROGRAM

Transportation Research Board Executive Committee Subcommittee for NCHRP

LOUIS J. GAMBACCINI, Southeastern Pennsylvania Transportation Authority
WAYNE MURI, Missouri Highway & Transportation Department
FRANCIS B. FRANCOIS, American Association of State Highway and Transportation Officials

Program Staff

ROBERT J. REILLY, Director, Cooperative Research Programs
LOUIS M. MACGREGOR, Program Officer
DANIEL W. DEARASAUGH, JR., Senior Program Officer
IAN M. FRIEDLAND, Senior Program Officer
CRAWFORD F. JENCKS, Senior Program Officer
FRANK N. LISLE, Senior Program Officer
DAN A. ROSEN, Senior Program Officer
HELEN MACK, Editor

TRB Staff for NCHRP Project 20-5

ROBERT E. SKINNER, JR., Director for Special Projects
THOMAS L. COPAS, Special Projects Engineer
HERBERT A. PENNOCK, Special Projects Engineer
JUDITH KLEIN, Editor
CHERYL CURTIS, Secretary

Field of Special Projects

Project Committee SP 20-5

VERDI ADAM, Gulf Engineers & Consultants
ROBERT N. BOTHMAN, Oregon Dept. of Transportation
JACK FREIDENRICH, New Jersey Dept. of Transportation
DAVID GEDNEY, De Leuw, Cather & Company
RONALD E. HEINZ, Federal Highway Administration
JOHN J. HENRY, Pennsylvania Transportation Institute
BRYANT MATHER, USACE Waterways Experiment Station
THOMAS H. MAY, Pennsylvania Dept. of Transportation
EDWARD A. MUELLER, Morales and Shumer Engineers, Inc.
EARL SHIRLEY, California Dept. of Transportation
JON UNDERWOOD, Texas Dept. of Highways and Public Transportation
THOMAS WILLETT, Federal Highway Administration
STANLEY R. BYINGTON, Federal Highway Administration (Liaison)
ROBERT E. SPICHER, Transportation Research Board (Liaison)

L. GARY BYRD, Consulting Engineer, Alexandria, Virginia
THOMAS D. LARSON, U.S. Department of Transportation
THOMAS B. DEEN, Transportation Research Board

Secretary

ROBERT E. SPICHER, Transportation Research Board (Liaison)
SYNTHESIS OF HIGHWAY PRACTICE

BRIDGE DECK JOINTS

MARTIN P. BURKE, JR.
Burgess & Niple, Ltd.
Columbus, Ohio

Topic Panel
IAN M. FRIEDLAND, Transportation Research Board
JAMES HOBLITZELL, Federal Highway Administration
EDWARD V. HOURIGAN, New York State Department of Transportation (Ret.)
JOHN D. O'FALLON, Federal Highway Administration
GEORGE W. KING, Transportation Research Board
EDWARD P. WASSERMAN, Tennessee Department of Transportation
HORST WELS, Pennsylvania Department of Transportation

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

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. SEPTEMBER 1989
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 governmental agencies, universities, and industry; its relationship to the National Research Council is an assurance 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 basis 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 National Research Council 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 National Research Council 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.

NOTE: The Transportation Research Board, the National Research Council, 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.

NCHRP SYNTHESIS 141

Project 20-5 FY 1984 (Topic 16-10)
ISSN 0547-5570
Library of Congress Catalog Card No. 88-50294

Price: $9.00

Subject Area
Structures Design and Performance

Mode
Highway Transportation

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. 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 American Association of State Highway and Transportation Officials, or the Federal Highway Administration of the U.S. Department of Transportation.

Each report is reviewed and accepted for publication by the technical committee according to procedures established and monitored by the Transportation Research Board Executive Committee and the Governing Board of 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 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 in 1974 from the Highway Research Board, which was established in 1920. 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.

Published reports of the
NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
are available from:

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

Printed in the United States of America
PREFACE

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

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

FOREWORD

By Staff
Transportation Research Board

This synthesis will be of interest to bridge designers, maintenance engineers, and others concerned with designing and maintaining bridge deck joints. Information is presented on the types of deck joints used in bridges and on the design of bridges without joints.

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

Bridges are continually moving and thus need either some type of deck joint or an integral design to accommodate this movement. This report of the Transportation Research Board describes the types of deck joints being used, the problems with these joints, and how integral construction—bridge decks without joints—can be used to avoid joints.
To develop this synthesis in a comprehensive manner and to ensure inclusion of significant knowledge, the Board analyzed available information assembled from numerous sources, including a large number of state highway and transportation departments. A topic panel of experts in the subject area was established to guide the researcher in organizing and evaluating the collected data, and to review the final synthesis report.

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

This synthesis was completed by the Transportation Research Board under the supervision of Robert E. Skinner, Jr., Director for Special Projects. The Principal Investigators responsible for conduct of the synthesis were Thomas L. Copas and Herbert A. Pennock, Special Projects Engineers. This synthesis was edited by Judith Klein.

Special appreciation is expressed to Martin P. Burke, Jr., Burgess & Niple, Ltd., Columbus, Ohio, who was responsible for the collection of the data and the preparation of the report.

Valuable assistance in the preparation of this synthesis was provided by the Topic Panel, consisting of James Hoblitzell, Structural Engineer, Bridge Division, Federal Highway Administration; Edward V. Hourigan, Director, Structural Design & Construction, New York State Department of Transportation (Ret.); John D. O’Fallon, Bridge Research Engineer, Structures Division, Federal Highway Administration; Edward P. Wasserman, Civil Engineering Director, Structures Division, Tennessee Department of Transportation; and Horst Wels, Bridge Design Quality Assurance Engineer, Pennsylvania Department of Transportation.

Ian M. Friedland, Senior Program Officer, Transportation Research Board, and George W. Ring, Engineer of Design, Transportation Research Board, assisted the NCHRP Project 20-5 Staff and the Topic Panel.

Information on current practice was provided by many highway and transportation agencies. Their cooperation and assistance were most helpful.

The author is grateful to all of those engineers who while struggling to improve the performance of their own departments were willing to devote some of their valuable time to help in the development of this synthesis. It is hoped they will find this work of some aid in improving that performance.
BRIDGE DECK JOINTS

SUMMARY

Bridges are continually moving. The movements are produced by expansion and contraction caused by temperature changes, by creep and shrinkage of prestressed concrete and concrete decks, by moisture change, by forces of vehicles on the structure (vertical deflections and lateral movements from braking), and by long-term movements such as those caused by settlement and earth pressure. The deck joints or the design of the bridge must be such as to accommodate these cyclic movements.

The most common method of accommodating movement is the deck joint, commonly misnamed an “expansion joint” for the thermal component of bridge movements. Bridge deck joints fall into two broad categories: open joints and closed joints.

Open joints allow the passage of water and debris through them. They are considered effective if the water and debris will fall clear of supporting structures or will be carried away by drainage troughs. The open joints include formed joints and finger-plate joints. Formed joints are simply open slots in the deck, usually with metal armor. They are most commonly used longitudinally in medians. Finger-plate joints allow for greater movements and are usually provided with a drainage trough. Longer fingers are designed for cantilever loads and are usually supported from below.

Closed joints are designed to keep water and debris from passing through. These include poured seals, compression seals, cellular seals, sliding plates, prefabricated elastomeric seals, and modular elastomeric seals. Poured seals are useful only where there will be very small movements—up to $1/4$ in. (6 mm). Compression seals can be used for up to 4 in. (100 mm) of movement provided that the joint has straight parallel sides and that a good adhesive is used to bond the seal to the sides of the joint. Cellular seals are similar to compression seals and have been used for movements up to 4 in.

Sliding-plate joints are common on short- and medium-span bridges with movements of 4 in. (100 mm) or less and without differential vertical movements. The sliding surfaces are designed to be in uniform contact but this is rarely achieved in practice; thus, water is likely to leak through these joints. The bridging plates should be designed as cantilevers to prevent joint distress and failure.

Prefabricated elastomeric seals come in various types including plank, sheet, and strip seals. Plank seals have been used for movements up to 13 in. (330 mm) and the others up to 4 in. (100 mm). Many of the designs for the prefabricated seals are proprietary. Considerable research has been done on prefabricated seals, with performance ranging from poor to very good. Modular elastomeric joints have been designed for movements from 4 to 24 in. (100 to 610 mm). Like the prefabricated seals, many designs are proprietary and research results have indicated performance ranging from poor to very good.

Integral construction—bridges without deck joints—is being used by bridge designers to avoid the problems associated with bridge deck joints. Over the last few decades many states have built bridges without deck joints. The lengths of jointless bridges have increased from about 150 ft (46 m) to as much as 2700 ft (820 m) in
recent years. Some of these bridges have a continuous deck with joints at the abutments but many are being built with integral abutments and no deck joints. The abutments and piers of integral bridges are designed to be flexible enough to accommodate the movements. Approach slabs are attached to the abutments and a cycle-control joint at the opposite end of the approach slab is designed to accept the movement. Integral construction has been successful, and several transportation departments are now converting existing multiple-span bridges to continuous.

For bridges with open joints or with leaking closed joints, it is necessary to protect the structure from the corrosive effects of the drainage water, especially drainage that contains deicing chemicals. Elastomeric drainage troughs installed beneath the joints on a slope of about 1 in. per ft (8 percent) have proved effective in collecting water and debris. Steel troughs, coated or galvanized, are also used as well as stainless steel. Alternatively, or in addition, structure elements can be designed to shed water and debris and can also be coated or sealed. Periodic cleaning may also be a cost-effective way to attain long-term durability.

In order to properly design bridge deck joints or integral bridges, the designer must be aware of the factors that affect bridge performance and bridge movements. These include earth pressure, pavement pressure and growth, embankment translation and consolidation, settlement, thermal and moisture changes, elastic shortening, creep, movements and rotations of skewed and curved bridges, and movements induced by bearings that do not move parallel to the surface of the deck or that rotate in a way that moves the deck up or down.

The success or failure of a particular design is directly related to the expertise of the individuals who contributed to the design's conception, development, production, adaptation, fabrication, testing, and installation. These individuals should be familiar with bridge behavior and structural design if the quality and durability of bridge deck joints is to improve.
INTRODUCTION

Although often thought of as static structures, bridges are continually moving.

Expansion and contraction caused by temperature changes, deflection caused by loads, movement of the adjacent earth, pressures of ice and stream flow, and centrifugal and longitudinal forces of vehicles all combine to produce motion in a bridge. Such movement is deceptively slow; however, the forces involved are tremendous. If the ability to move is not built into the bridges, it pushes and tears at its supports until it achieves the freedom of movement it requires.

Movement is usually accommodated by bearings; however, because bridge bearings are often sources of trouble, many bridges are designed so that the entire structure takes care of movement without bearings. This is done by making bridge piers flexible, using radial expansion if the bridge is on a curve, allowing for movement in the abutments, and being sure the approach pavement is not so rigid that the bridge is locked into place and not allowed to expand and contract as temperatures dictate. When these methods cannot be used, however, expansion details must be provided to allow the bridge free movement (1).

The design and performance of a bridge’s bearings have a significant effect on the performance of the bridge’s deck joints whether those joints are incorporated within the bridge proper or adjacent to the bridge on its approaches.

This synthesis focuses specifically on the development, design, construction, and evaluation of deck joints for bridges; the experience over the last two decades in the development of various types of elastomeric joint seals; the development of integral construction; and some of the developments in joint waterproofing and joint drainage management.

Important to an understanding of the deck joint experience, however, is an awareness of the many disparate phenomena that conspire against the achievement of simple and effective designs for bridges. These include deicing chemicals, overloaded vehicles, snowplows, pavement pressures, and bridge geometry.

The increase in the use of deicing chemicals in the 1960s and 1970s led to the deterioration of concrete bridge decks and other components of bridges. Deck joints that are not watertight contribute to this deterioration; the damage shown in Figure 1 was caused by leaking deck joints and was so severe that complete replacement of the bridge was required (at a cost of $25 million). Similar bridges without leaking joints are still in service without major problems.

Overloaded trucks cause many problems with bridges and bridge deck joints; overloads as high as 449,000 lb have been reported (59). Snowplows can damage joints that are not designed carefully. The growth of concrete pavements can create pressures on bridges that will close the joints and cause considerable damage (see Appendix A). The geometry and skew of a bridge affect the performance of a joint, especially where the design is such that the movements of the bridge are not parallel with those of the joint.

This synthesis describes the experiences of many state and province bridge engineers as that experience pertains to bridge deck joints, integral construction, drainage control, material quality, etc. These experiences can only be completely understood by considering the many elements either stated or unstated that have influenced their decisions and consequently their results. The summary of these experiences is based primarily on a mail survey made in late 1985 and early 1986, on published research reports, and on a few papers by transportation department engineers and manufacturer’s representatives. The response to the survey was substantial enough to consider the summary representative of bridge experience in the United States and Canada.

JOINT MOVEMENT

The first major deck joints in small bridges were introduced when the change from masonry bridges to bridges with masonry substructures and timber or metal superstructures was initiated. These joints not only simplified the design of bridges whose major elements were composed of dissimilar materials, they also simplified the repair and replacement of superstructures composed of these less durable materials. The joints also served important structural purposes because they protected slender superstructure members from being compressed by uncertain

FIGURE 1 Deterioration from deicing chemicals. (Ohio DOT photo)
and unquantified earth pressure, and from the adverse consequences associated with differential settlement.

Along with expansion of manufacturing capacity and the improvement in material strength and quality, bridge spans became longer. The design and construction of bridge deck joints also became more complex to accommodate the longitudinal movements of these longer spans. Heavier vehicular traffic also contributed to the structural changes that took place in the design and construction of bearing devices and transverse deck joints for these longer bridges.

Following the introduction of moment distribution in the early 1930s, bridge engineers began eliminating intermediate deck joints of multiple-span bridges. The adoption of continuous-span construction, which was motivated by the desire to improve bridge economy, performance, and durability, compounded the bridge engineers' problems in another respect because it increased longitudinal movement of the deck joints at the abutments of these bridges. The recently constructed Long Island Bridge in Kingsport, Tennessee, is composed of 29 continuous spans with a total overall length of 2700 ft (820 m) (Figure 2). Consequently, the movement at bridge deck joints has changed from a barely perceptible movement in the earliest bridges to current cyclic movement ranges (contraction, expansion, etc.) in excess of 12 in. (300 mm).

The design of deck joints for cyclic movement has also been compounded in the case of cast-in-place concrete structures and prestressed or post-tensioned concrete structures because the movements associated with moisture changes, long-term creep, and elastic effects must also be allowed for in the design. In addition, the cyclic end rotations associated with the movement of vehicular traffic on long spans and deep structural members, the differential deformation of elastomeric bearings, and the mechanical movements of structural bearings also must be considered in the joint design if these designs are to function effectively for extended periods.

Some of the movements mentioned above commence during a bridge's construction. They can have an adverse effect on the performance and integrity of the deck joints unless these movements are anticipated in the design of the joints, or unless the joints are installed after these initial movements have occurred.

Beginning in the early 1950s with the construction of toll roads and other major parkways and expressways, the winter maintenance policies of many transportation departments began to change. To keep these roadways open year round, these departments adopted a dry-pavement policy in which deicing chemicals were used to help keep roadways and bridge decks clear of snow and ice. The full consequences of bridge deterioration and corrosion caused by deicing chemicals penetrating bridge deck joints were not fully realized until the late 1950s and early 1960s. By that time bridge deterioration and corrosion were extensive and transportation departments and manufacturers began the search for waterproof deck joints, a search that has been only partially successful and one that has continued up to today.

**TERMINOLOGY**

As is evident in the preceding discussion, the misnomer "expansion joint" is missing, and except for this paragraph and in quotes from other publications, the term has not been used in this synthesis. It has become apparent that the naming of bridge joints for the minor component of thermal movement is partly responsible for the inadequate sizing and design of many deck joints for short- and medium-span bridges.

Many bridge engineers, technicians, and others determine the size of joints for bridge decks based exclusively on an estimation of the thermal movement of the joints. However, these deck joints also should be sized to provide for the initial and post-construction movements caused by shrinkage of concrete decks; creep of prestressed and post-tensioned concrete decks; cyclic movement caused by moisture changes; cyclic rotations induced by the movement of vehicular traffic; rotations associated with deck placement and camber growth; and long-term movements caused by earth pressure, embankment translation, settlement, pavement pressure, etc. Thus, the use of the term "expansion" as the functional name for deck joints has only served to concentrate attention on thermal movements and obfuscate all of these other important bridge movement characteristics.

Consequently, it is suggested that the term "expansion joint" be abandoned as at least one small way to help focus more attention on other major aspects of bridge movement as that movement is reflected in the movement of bridge deck joints.
CHAPTER TWO

JOINT TYPES

A number of different joint types have been developed for various deck joint movement ranges. Some joint types, the pre-formed elastomeric, for example, have a number of different basic designs. Both proprietary and nonproprietary designs have been developed. This chapter is devoted to a brief description of these joint types, the experience of transportation departments in the design and application of these joints, and an evaluation of the relative success or failure of transportation department engineers in achieving economical and effective deck joints for bridges.

OPEN JOINTS

The primary functions of open deck joints may be enumerated as follows:

- Permit cyclic and long-term movement
- Support traffic
- Pass water and debris
- Survive service

The various types of open joints that satisfy these functions to a reasonable degree can be enumerated in a sequence based on the increasing amounts of movement that they can effectively accommodate. They include the:

1. Formed joint
2. Finger-plate joint

The formed joint is generally restricted to a maximum movement range of about 3 in. (75 mm). This results in a 1 in. (25 mm) joint opening at normal temperatures and a 3 in. (75 mm) joint opening at the lowest expected temperature.

For applications where greater movements are expected or where a 3 in. unrestricted opening is considered unsuitable, finger plates are usually employed.

Formed Joint

With respect to satisfying the primary joint functions, it is clear that any reasonably conservative formed joint design will result in a joint with a high functional efficiency. Consequently, this type of joint can be classified as the most efficient cost-effective joint currently available to the bridge engineer (Figure 3). Such a classification assumes, of course, that the passage of water and debris through the joints will fall clear of supporting structural members. Joints in raised medians or between median barriers are the most common application. Formed joints have also been used in transverse joints in long continuous structures where the joints are located midway between adjacent piers. They are also applicable for joints above bridge seats for those bridges located in geographical areas with minimal precipitation and temperatures above 35°F (2°C).

In geographical areas where deicing chemicals are used, the adverse consequence associated with retained water and debris on substructure members precludes the use of open joints in locations other than those areas where the water and debris can drain from all supporting substructure members.

Finger-Plate Joint

Where a deck joint must provide for greater superstructure movements, the finger-plate joints shown in Figures 4 and 5 are employed. Generally, these joints are provided with deflectors or some sort of trough below the joint to intercept deck water and debris and conduct it away from substructure members. (See Chapter Four for a description of these joints and the various types of troughs that are now being employed by transportation departments.)

In 1980, the Federal Highway Administration (FHWA) issued a Technical Advisory (2) that contains comments and suggestions for “tooth joints” that emerged from a seven-year study by 40 states using more than 825 joint applications (3).
With respect to the design of this type of joint, the Advisory recommends:

1. Limit deck surface openings on toothed joints to permit safe operation of motor bikes. When the maximum longitudinal opening in the direction of traffic exceeds 8 inches, the transverse opening should not exceed 2 inches. For longitudinal opening less than 8 inches, the transverse opening may be increased to 3 inches.
2. Where narrow bicycle tires are anticipated, use special floor plates in the shoulder area.
3. The minimum joint opening (maximum design temperature) in the longitudinal direction should be 1 inch.
4. At maximum joint opening (minimum design temperature) the tooth overlap should be at least 2 inches.
5. Align teeth in the longitudinal direction of the bridge and align bearing devices to assure thermal movement in this direction.
6. Because of the rapid accumulation of cycles of fatigue loadings, welding details are critical.
7. Give special attention to the development of details and design specifications to be assured that the anchorage will have strength substantial enough to preclude precipitating a failure of the joint system. On structural steel supported bridges, the joints should be rigidly fixed to the stringers or girders.

Benson (4) documented the evaluation of deck joints made by a team of engineers for the state of Arkansas. The discussion of these evaluations, which reviewed the performance of finger joints, sliding-plate joints, strip seals, and compression seals, commences: “Based on team evaluations, the finger joints provide the best performance. . . .”

In another report, Hamilton (5) reported the results of an examination of 100 bridge joints in Maine. With respect to steel finger joints with drainage troughs, Hamilton reported that the ratings for these joints were good. Only one bridge had low ratings, which were caused by maintenance and watertightness problems. One maintenance manager remarked that this type of joint is the best in service even though annual cleaning is necessary to remove the debris. There was some concern among designers regarding ice buildup in the trough, which would restrict expansion movements.

Hamilton reported that open finger joints without drainage troughs were no longer specified because they allowed deicing chemicals and debris to spill onto the substructure, causing concrete deterioration and structural steel corrosion. This joint was found very durable relative to snowplow damage (5).

In 1985, Dahir and Mellott (6) reported on the inspection and evaluation of the joints of 146 bridges located throughout Pennsylvania. With respect to finger-plate joints they comment:

Some of the cantilevered fingers can be bent or even broken under the continuous pounding of heavy truck traffic unless they are designed with sufficient tensile strength, well aligned and properly anchored during construction. . . . During construction and bridge deck maintenance, care must be taken to have the finger dam system well anchored and at the proper elevation with the adjacent pavement surface. . . .

CLOSED JOINTS

The primary functions of closed deck joints may be enumerated as follows:

- Permit cyclic and long-term movement
- Support traffic
- Repel water and debris
- Survive service

The various types of closed joints that satisfy these functions to a reasonable degree can also be enumerated in a sequence based on the increasing amounts of movement that they can effectively accommodate. They include:

1. Poured seal
2. Compression seal
3. Cellular seal
4. Sliding plate
5. Prefabricated elastomeric seal
6. Modular elastomeric seal

As enumerated above, the poured joint seal can accommodate the least amount of movement, say about ¹/₄ in. (6 mm). The compression seal, sliding plate, and prefabricated elastomeric seal are generally designed for movement ranges of from ¹/₄ in. to 4 in. (100 mm), although some of the prefabricated elastomeric joint devices have been designed for movements of up to 13 in. (330 mm). The modular elastomeric seal has generally been designed for movement ranges of from about 4 in. to 24 in. (610 mm), with an occasional joint designed for movements in excess of 24 in.
With respect to satisfying the primary functions, each joint type has a number of inadequacies. For example, a well-designed and constructed sliding-plate joint facilitates cyclic and long-term movement, supports traffic, survives the structure, and is reasonable in cost. However, the lack of continuous contact of the contiguous sliding surfaces of the joint permits moderate amounts of deck drainage to leak through the joints. In geographical areas where deicing chemicals are applied to bridge decks, even moderate amounts of leakage significantly accelerate steel corrosion and concrete deterioration.

The poured joint seal supports traffic, repels water and debris, survives limited service, and is low in cost. However, because it can provide for only a small amount of movement, it must be restricted to bridges with relatively static substructures and to superstructures where the amount of movement is small and can be accurately estimated. Also, because this device depends on quality sealant materials and careful construction, the ultimate success of the device will be almost directly related to tight material specifications and good fieldwork. A poor choice of structure, underestimation of the amount of movement, and/or poor fieldwork will almost certainly result in a failed joint.

Each of the other joint types has other inadequacies and it is the bridge engineer's primary responsibility to choose the most appropriate joint type for the movements anticipated and to design the joint to achieve the highest functional efficiency commensurate with a justifiable first cost and a reasonable lifecycle cost.

**Poured Joint Seal**

For deck joints that have only a very moderate amount of movement, say less than 10 to 20 percent of the joint width, poured sealants have been used to prevent both debris and water from penetrating the joints. Figure 6 shows details and some of the changes that have been made during the development of this type of seal. Initially, just the filler and an asphalt sealer were used. Then, to improve flexibility of the sealer and also to separate chemically incompatible fillers and sealers, bond breaker was developed and specified. Alternatively, an elastic backer rod was devised to both separate the materials and to help to improve the geometric shape of the sealant. Ultimately, the edges of the joint were armored to prevent concrete faulting and spalling.

![Diagram of Poured Joint Seal](image-url)
There is a significant amount of literature on poured sealants. A bibliography spanning the years between 1917 and 1980 was prepared by the Transportation Research Board's Committee A2G03 (7). It contains "a world summary of joint and crack sealing [and] related documentation for pavement, bridges and structures." Much pertinent research and experience related to poured sealants are identified in this bibliography.

In 1962, Schutz (8) described a new approach to joint design based on a five-year study of failed joints. He suggested three primary causes for the poor results that had been experienced with poured sealants: poor joint design, use of inappropriate sealants, and poor workmanship. In the 1960s and early 1970s, papers by Britton (9), Stewart (10), Oehler and Bashore (11), Thornton (12), and Gunderson (13) outlined the field problems with poured seals and their relatively short service life. Many other transportation department engineers also recognized the need for a joint seal that was not as sensitive to field installation conditions and inept and indifferent field labor. Consequently, at about this time, much effort and attention was focused on other types of preformed or prefabricated joint seals to supplement and possibly replace the poured sealants in bridge deck joints.

**Compression Seal**

Beginning in the mid 1960s, preformed elastomeric compression seals began replacing poured joint sealers and sliding-plate joints in smaller bridges. Since these first installations, numerous types of elastomeric joint seals have been developed and improved in an attempt to achieve a joint seal design that would be both effective and durable. Most of these designs have been disappointing. Many of them leaked; some required more maintenance than the original bridge built without them; and the cost for seals has become alarming. For example, in one rehabilitation project for two moderate-span bridges containing nine joints, the joint remodeling and seal installation cost about $250,000. Yet these seals failed after the first winter of service and had to be removed because they were becoming a hazard to the movement of vehicular traffic. Other seals have remained intact but were not watertight. Some seals, notably the simple compression seals, strip seals, and the modular joints containing these seals as elements, have experienced a fair measure of success.

Watson (14) described some of the first experiences with compression seals used in bridge joints in 1965 and wrote numerous papers and other publications on the use of elastomeric seals for bridge joints throughout the next 20 years.

In 1967, Kozlov (15) published a paper on elastomeric compression seals that contained background on elastomeric materials, seal behavior, and application procedures. In 1969 he described the laboratory evaluation of elastomeric materials for compression seal applications (16). These two papers were followed by others that supplemented and updated the earlier papers.

Watson's papers and Kozlov's research formed the basic background that many transportation department engineers used to guide their own applications of elastomeric compression seals for bridge deck joints. More recently, Puccio (17) presented a paper that contains a rather comprehensive discussion of compression seals. It is a valuable paper containing much background material and useful application information.

Figure 7 shows the compression seal configuration that has generally been adopted for bridge joint applications. Earlier seal applications were made using formed concrete joints (Figure 7a). Lack of uniformity of the formed joints soon led to the requirement that concrete joints be made by cutting with gang saws to ensure that joints were straight and the sides parallel and properly spaced to suit the specified seal sizes. For many bridges, the joints are now reinforced with well-anchored steel armor (Figure 7b). This protects the joints from faulting and spalling, a problem encountered on bridges subjected to large traffic volumes, heavy truck traffic, and snowplows.
Initially, the seals were installed with the aid of an oil soap. Later, to help fill the voids between the seal and abutting joint surfaces, a neoprene-based lubricant with a 25 percent solids content was adopted. Currently, a moisture-curing polyurethane material with a 75 percent solids content is being used throughout the industry. A high-performance bonding procedure developed at Akron University is being used by the Ohio Department of Transportation for compression seals (57). The procedure uses a "flexible" epoxy adhesive, cyclized elastomers, and abrasively cleaned joint surfaces. Once used with strip seals, it had to be abandoned because the high bond strength prevented the removal of damaged strip seals. Because the adhesive is temperature sensitive, the procedure is also limited to installation temperatures of about 45°F (7°C) or above to cure properly.

With more development in this area, the future preformed elastomeric seals will probably depend less on the compression characteristics of the seal configuration and more on a good structural bond between the seal and joint faces.

The details illustrated by the drawings in Figure 7 are somewhat misleading. Properly supplemented with dimensions, section sizes, and material properties, these details would be sufficient for square bridge decks without curbs, raised barriers, and fascia parapets. However, for the routine bridge where skew, curvature, curbs, barriers, or parapets must be recognized and be provided for, the joint details become more complex and the probability of achieving an effective seal becomes more difficult. Improperly designed joints, even those joints constructed with high-quality materials and superior adhesives, will probably fail even before a bridge is opened to traffic.

Elastomeric compression seals have been found suitable for bridge applications by many transportation departments and, in fact, are given very high ratings by these departments; however, other transportation departments have abandoned their use entirely. Some recent state research reports give a pragmatic view of elastomeric compression seals.

A 1983 Colorado report by Swanson (18) on the performance of 72 joint seals in one bridge stated that the joints were heavily damaged by snowplows. Nevertheless, only 4 percent of the joints were found to be leaking. Swanson concluded:

The most important factor which will ultimately determine the success of an expansion joint is proper installation during construction.

A finished joint which is recessed \(\frac{1}{4}\) inch to \(\frac{1}{4}\) inch and has good approach protection will most likely not be damaged by traffic or snowplow equipment.

CDOH will continue to use compression joint seals where the movements are less than 2 inches.

In 1984, Weishahn (19) conducted a field review and evaluation of the joints in 49 bridges in Nebraska constructed during the period between 1969 and 1979. With respect to the 25 joints containing compression seals, he reports:

This type of device material is very effective when design allows its use within its limitations. If theoretical movements allow its use, they will perform satisfactorily. When all portions of the structure are moving and functioning properly, the ease of maintenance is acceptable. A very important consideration that must be controlled is the placement location of the hardware used to restrict movement of the material within the joint. If the joint material is installed too high, there will be snowplow and traffic damage to it. If the material is installed too low, debris will accumulate and traffic will force the material through the bottom of the joint.

Hamilton (5) evaluated the deck joints of 17 bridges in Maine with elastomeric compression seals. He commented:

Problems resulted from specifying too small a cross sectional seal for a large skew bridge which resulted in the seal rupturing. Occasionally, the seals failed at a splice. At locations where there was unanticipated joint movement due to abutment tilting, unstable fixed bearing location or embankment movement, seals have occasionally fallen out or been squeezed to the point that the rubber extruded from the joint.

However, the report concludes with the statement:

Armored joints with elastomeric compression seals were used with much success. Bituminous concrete pavement adjacent to these joints seems to be greatly affected by freezing/thawing cycles, probably due to a lack of proper compaction immediately next to the joint armor. This, in turn, opens the door for snowplow damage to the joint armor.

Another recent report documents the evaluation of deck joints made by a team of engineers for the state of Arkansas (4). With respect to preformed compression seals, 172 joints, aged five to six years, were examined. The following was reported:

The preformed compression seals also exhibited a high degree of debris accumulation. In addition, many of the seals were not adhered to the sides of the joint throughout their length, but had pulled loose. This lack of adherence was particularly evident on the wider seals. No adhesive has been found which will satisfactorily bond the compression seal to the steel armor. This is considered by the Department to be the major flaw in this system. Problems with wide seals on joints not a part of this study has led to the discontinuance of this type of seal for joints wider than 3 inches.

Cellular Seal

The cellular seal was introduced in a proprietary form by E-Poxy Industries. E-Poxy Industries' "Evazote" is composed of a closed-cell foam. Nonproprietary forms include both closed- and open-cell foams and sponges, some with various types of encapsulations.

The joint detail used with cellular seals is similar to those that have been developed for the compression seal (Figure 7). Being a recent addition to the transportation field, cellular seals have received scant mention in recent joint seal research reports. One maintenance engineer did remark, however, that he has used cellular seals on occasion to replace compression seals that had failed.

One of the major problems with the closed-cell type of cellular seal or encapsulated seals is that they experience compression set after being subjected to sustained compression. A particular advantage of this type of seal is that it can be fabricated in one piece to fit joints with complicated joint geometry. It appears probable that future developments with cellular seals, including improvements in material quality and adhesive bonding procedures, will ultimately result in a seal type that will be readily adaptable to a broad range of bridge joint applications.

Sliding-Plate Joint

Since about 1900, sliding-plate devices have been used for the deck joints in short- and medium-span bridges. Generally, these
devices are limited to horizontal movement ranges of up to 4 in. (100 mm) and bridge lengths between joints of about 350 ft (110 m). They are structurally simple and, for short- and medium-span bridges with static substructures and secure foundations, standard joint designs have been developed that are routinely expected to survive the structure (Figure 8a). However, where differential vertical movements can occur at such joints, where substantial joint anchorages have not been provided, or where the joints are not recessed slightly below the roadway surface, even these joints can be subjected to significant distress and rapid deterioration.

Although the sketch shown in Figure 8a indicates that the sliding surfaces of the joint device are in contact, the drawing represents an ideal rarely achieved in actual practice. Actually, it is not unusual to find joints of this type with a considerable variation in the fit of the contiguous sliding surfaces. In most joints, gaps of \( \frac{1}{16} \) in. to \( \frac{1}{8} \) in. (1.6 to 3.2 mm) or more are not unusual. The fact that most of these joints leak to a significant degree is the reason why the joint design of Figure 8b was developed.

Research on sliding-plate joints is rather limited. Nevertheless, recent reports contain some mention of them.

In the 1977 final report for the National Experimental and Evaluation Program (NEEP) Project 11 (3), the Summary of the report stated in part:

Steel toothed and sliding plate joints, although not intended to be leakproof, have had long service life in bridge decks. A number of states have continued to use these joints but have incorporated full joint width neoprene troughs with and without downspouts to collect the water and carry it off the structure. . . .

Hill and Shirolé, of the Minnesota Department of Transportation and the city of Minneapolis, respectively, report (20):

Heavy wheel loads pound improperly placed and exposed plates, angles, seals, and glands to cause rapid disintegration of adjacent materials. Plates and angles bend, warp, and sometimes break off from their anchorages. Types G, I, L, O, and P have been especially prone to this problem.

The reference to “G” and “I” above is to sliding-plate joints and “L” to finger-plate joints; “O” and “P” refer to joints with armor angles used to reinforce open joints and joints containing compression seals. It is clear from this evaluation that vehicular traffic and maintenance effects must be realistically provided for if the structural portions of these joints are to survive the structure. Some of the distress experienced by these joints probably is related to differential vertical movement at the joints, movement that was probably not provided for in their design.

Benson (4) reported:

The sliding-plate joints evaluated [in Arkansas] had the worst performance. The setting of the plates and their supports during construction was difficult, and slight errors in setting led to early failure. All of the joints on two of the bridges had failed completely, and the other joints showed signs of impending failure. The bolts holding the plates on the supports broke, allowing the entire plate to pull free from the deck. . . .

Hamilton (5) reported on sliding-plate joints in Maine that employed a bolted closure plate. He states:

The poor features of this joint are that the plates need to be adjusted periodically to reduce noise levels; and when the plates get out of adjustment, they create a hazard for snowplows. When the plates become bent from contact with plow blades, they create a very unsafe condition for the travelling public.

In 1984, Weishahn (19) evaluated the performance of the joints in 49 bridges constructed between 1969 and 1979. With respect to sliding-plate joints he reports:

Prior to 1973, the Nebraska Department of Roads primarily used sliding steel plates or the heavy steel-toothed devices in their bridge deck joints. We viewed these as unsatisfactory due to leakage, which could not be controlled, and the resulting corrosion problems. Also tried was the “trough” under the device to catch the drainage, which also proved unsatisfactory.

Prefabricated Elastomeric Seal

Perhaps no other type of closed joint has given transportation engineers more trouble and concern than the prefabricated elastomeric seal. Developed in the late 1960s and early 1970s, the
application, field testing, and evaluation of these seals has been going on ever since.

With respect to satisfying the primary functions for joints, all of the joint types appeared to be capable of providing a level of performance to justify their initial consideration. The plank seals have the greatest burden because they must function as small movable bridges supporting vehicular wheel loads on spans of up to 9 in. (230 mm). And as has been reported, some of the plank seals have failed because of their inability to support such loads while permitting the desired cyclic and long-term movements. Many of the discontinuous plank seals, especially those that were installed in skewed bridges, have deteriorated under normal traffic faster than expected. Those that contain large areas of elastomer exposed on the roadway surface have been damaged by snowplows to such an extent that many have had to be removed to protect the movement of vehicular traffic.

Figure 9 shows three representative types of prefabricated elastomeric seals, namely, the plank, sheet, and strip. The plank seal is used for movement ranges of from 2 to 13 in. (50 to 330 mm). The other seal types are generally used for movement ranges of 4 in. (100 mm) or less.

The plank seal was originally developed by the General Tire and Rubber Company. Some of the present joints of this type include General Tire's "Transflex," Watson/Acme's "Waboflex," and Royston's "Unidam."

There are many versions of the sheet seal, including both proprietary and nonproprietary designs. Present proprietary designs include Felt Products Corporation's "Fel-Span" and "Pro-Span," Watson/Acme's "Elastoflex" and "Bendoflex," General Tire's "Gen-Strip," D.S. Brown's "Delastiflex," and Structural Accessories' "Onflex."

The strip seal employing elastomeric and steel extrusions was first developed by the Maurer Sohne Manufacturing Company, and a number of similar designs were developed by others. Present designs include Maurer's "Strip," D.S. Brown's "Steelflex," and Structural Accessories' "Onflex-Membrane."

Research on preformed elastomeric seals is extensive. The summary of the 1977 final report for NEEP Project 11 (3) stated in part:

A substantial majority of the steel reinforced elastomeric joint devices furnished in 4 feet (1.22 m) and 6 feet (1.83 m) butted segments have experienced problems. Although tongue and groove configurations and flaps have been incorporated into these devices and all sorts of butt splice sealants have been used, most of these devices leak to some degree. . . .

Swanson (18) of the Colorado Department of Transportation reported on a 1983 evaluation of 13 different systems used to seal the 128 joints in 21 structures. He concludes the evaluation in part as follows:

The most important factor which will ultimately determine the success of an expansion joint is proper installation during construction. All measurements have shown that attachments and anchors are very important in the final alignment and position of the expansion joint. A finished joint which is recessed 1/8 inch to 1/4 inch and has good approach protection will most likely not be damaged by traffic or snow removal equipment. Rigid inspection during installation is essential. Most expansion joints can work effectively if they are installed properly, however, the results of this study, which includes only those expansion joints on the experimental list . . . indicates that compression joint seals, Acme strip seals, Onflex, and Delastiflex have performed the best.

None of the devices in Table B [devices that have been in place for 3 years or more] can be classified as acceptable or unacceptable at this time because the cause of failures have not been conclusively proven to be related to installation, snowplow, traffic, or simply the failure of the device. CDOH will continue to use . . . continuous strip seals such as Onflex and Acme where movements are less than 4". Even though they don't have a good performance record, Waboflex and Transfiex will be used when movements are between 4" and 13" because they are the only products on the market that can be used for these large movements.
Fincher (21) of the Indiana Department of Transportation made a 1983 report on a five-year evaluation of 15 joint types, comprising 97 joint installations on 38 structures. He concluded that the experimental joint systems surveyed allowed less water and debris to reach the deck support systems than the sawtooth and sliding-plate joints, although as the joints were continuing to deteriorate, leakage would increase and spread to all the joints. Fincher stated:

Initial installation of the joints appears to be a significant factor of concern. Many of the joints were constructed with poor vertical alignment, making the joints susceptible to snowplow damage. Also the sealants used around the joints are failing prematurely causing water to leak through the joint anchorage. Some of the joints were found to be loose shortly after construction. Installation to proper grade and alignment of some joint systems is reported to be difficult and time consuming.

Indications are that none of the joints evaluated in this study have sufficient merit to be considered for unrestricted use on Indiana structures.

In a 1983 interdepartment memorandum, Alexander (22) reported on a Kansas Task Force review, analysis, and evaluation of all existing joint-seal devices (comprising 30 different devices from six manufacturers) and the practices of design and construction in specifying and installing the devices. In one of the few evaluations that recognize specific contributory causes for joint-seal distress, he reports in part:

...failures have occurred which are not the fault of the Manufacturer or their device. Due to the [lack of] attention to details by K.D.O.T., 12% of the joints have failed. Six joints have failed because the Maintenance Department failed to maintain adequate pavement pressure relief joints in the approach pavement. Nine joints have failed because the Construction department failed to install joints within the 1/8" installation tolerance or failed to provide the necessary shelf elevation. Twenty-two joints failed because the Design department failed to specify a device with adequate movement. In addition to the failures, there are other problem areas that need to be addressed.

The following observed conditions will have to be corrected or minimized for K.D.O.T. to continue the use of watertight joints. Joints were originally set flush with the deck surface. Slight variations in elevation resulted in snowplow damage to the devices. Joints were then recessed 1/4" and the armorment remained flush with the deck surface. Again, slight variations in armament elevation resulted in snowplow damage to armorment and surrounding concrete. Armorment and joint have been set level, when they were always designed to be set parallel to crown grade line. Curb sections detailed to conform to curb shape are difficult to fabricate and as a result, have been damaged by snowplows. Manufacturers have supplied segmental units which are not spliced at the crown. Result, loose bolts and snowplow damage. Improper anchorage has resulted in loose bolts and moisture leakage around the device. All turned up curb sections cause debris accumulation. This debris accumulation has punctured the gland of strip seal. Gland pullout has been caused by faulty installation or unclean locking devices or grips. Deck spalls and concrete voids around armorment have been caused by inappropriate details. . . .

In 1984, Frederick (23) made a report on an interim evaluation of 15 different joint systems in New York. He reported that plank joints and joints made with aluminum were easily damaged by snowplows. This damage increased leakage and decreased the useful life of the joint. Joint systems that required field splices of sections to make a complete joint were incapable of maintaining a watertight seal; the sealant used in the splices did not last (23). Frederick also reported that the performance of expansion anchors has been poor. The dynamic loading on the joint has caused the anchors to wear the concrete around the expansion end of the anchor. Eventually this allows the anchors to work out of the deck, causing failure of the joint (23).

In a 1984 paper, Hill and Shirolé (20) reported on the evaluation of more than 2000 joints that had been installed in Minnesota. They conclude:

1. Joint devices and glands must be continuous and not segmental.
2. Concrete material should be used on either side of the expansion device and the joint should be sealed between the device and the concrete.
3. The expansion joint device should be recessed 0.25 to 0.5 in. below the adjacent concrete.
4. Snowplow guards for glands should be added on expansion devices placed at 20-degree or greater skews. Three-eighths steel bars placed out of wheel tracks will work adequately.
5. Claws of expansion device must hold the device securely. Bolted down claws generally loosen up and allow the gland to easily pull out.
6. Devices must be protected with a coating such as galvanizing.
7. Routine bridge maintenance should include cleaning the gland out and minor repairs to the gland.
8. Cast-in-place plate anchorage systems hold the device securely during construction and in service. Drilled-in anchorages work loose and expose the device and gland to potential damage.

In 1984, Weishahn (19) reported on a field review and evaluation of 43 state and six county bridges in Nebraska and stated that they had not found a joint that was entirely satisfactory with respect to being watertight.

In 1985, Dahir and Mellott (6) reported on the evaluation of 146 bridges located throughout Pennsylvania. With respect to strip or gland systems they report:

These devices provide movements up to four inches. Several of these systems have been used in Pennsylvania on an experimental basis. Generally, their performance has ranged from fair to quite good. . . .

With respect to plank-type systems they report:

These systems were intended to replace the finger dam system, accommodating movements between four (4) and thirteen (13) inches, with no need for maintenance. They included the Transflex, the Waboflex, the Fel Span and the Unitidam joint systems. The performance of these systems has varied from poor to barely satisfactory. Generally, these systems are expensive and performance has been disappointing.

In 1986, Price and Simonsen (24) reported on the evaluation of more than 100 installations comprising eight different systems that all utilized a "continuous length sealing gland." As a background for this evaluation, they state:

From 1972 to 1978 the majority of expansion joint devices installed in Michigan were metal-reinforced elastomeric pad systems. In general, these were an improvement over the metal plate joint. However, leakage between pad sections and the metal plate joint was a problem of sufficient magnitude to abandon the system in favor of one utilizing a continuous length sealing gland. Since 1978 the continuous length sealing gland system has been specified for bridge deck expansion joints in Michigan.
The evaluation of continuous-strip seals is summarized in part as follows:

Evaluation of the bridge expansion joint installations has indicated that other than poor design, the three major factors that adversely affect the function of the bridge expansion devices are:
1. Failure of the concrete under the expansion joint on rehabilitation work,
2. Failure of the device's anchorage, and
3. Improper installation techniques.

Hilton (25) reported in 1986 on the evaluation of a continuous-plank seal. He stated that after five years of service the east joint had either collapsed or compressed considerably beyond the \(\frac{1}{4}\) in. (6 mm) depression with respect to the original concrete deck as originally installed. The elastomer in the west joint, which had been in service for only four years, exhibited more severe sagging in the right lane than did the east joint. In the area of the right wheel path of the right lane the elastomer had failed (25).

A team of engineers from the Arkansas State Highway and Transportation Department composed of the Bridge Design Engineer, an Assistant Construction Engineer, the Heavy Bridge Maintenance Engineer, and a Research Engineer examined a total of 135 joints in 21 bridges. In the 1986 report of that evaluation, Benson (4) comments:

...the finger joints provided the best performance, and the sliding plate joints the worst. The strip seals and preformed compression seals were approximately equal in overall performance.

With respect to the performance of strip seals, he elaborates:

All of the strip seals showed a severe problem with debris accumulation. In most cases, the gutter formed by the seal was packed full of debris. The rate of accumulation is so rapid that cleaning is not economically feasible. It is not known whether this debris accumulation leads to failure. Approximately half of the strip seal joints evaluated had locations where the neoprene seal had been pulled out of the metal extrusion, effectively eliminating the watertight feature. This type joint will continue to be used on wide joints where finger joints or sliding plates are not feasible.

Figure 10 illustrates a modular elastomeric joint seal of the type originated by the Maurer Sohne Manufacturing Company of West Germany. The device has been marketed in essentially its present form for the last 16 years. A number of devices of similar design by other manufacturers have been designed and constructed and, at present, most major joint manufacturers have a modular design that they believe is competitive with the Maurer design both functionally and economically.

In the early 1970s many of the other modular designs that were offered to the transportation departments were of questionable quality and integrity. In fact, some of the first designs failed almost immediately after they were installed in bridge decks. Others failed to function as intended shortly after they were subjected to traffic. Subsequently, the designs for modular joints have gradually been improved and more conservatively designed, and some of the devices available to the transportation profession today appear to have sufficient integrity to justify their consideration.

The modular elastomeric seal is generally used for joint movement ranges greater than 4 in. (100 mm). The standard devices are designed for movements of up to 24 in. (600 mm). However, a special modular design has been prepared recently that will accommodate up to 48 in. (1200 mm) of movement.

Because the movement ranges of most bridge joints are rather moderate, and because the modular joints are relatively expensive when compared with the prefabricated elastomeric joint, only a limited number of modular joints have been installed. Nevertheless, some recent inspection reports are available where modular elastomeric joint seals have been evaluated.

In the 1977 final report for NEEP Project 11 (3), the summary stated:

For joint movements greater than 4 inches (101.6 mm) the earlier modular systems have proved troublesome. A persistent problem seems to be an inability of the steel I-beam separators to remain vertical and in continuous contact with transverse support beams in the device. This contributes to the problem of unequal compression in the individual neoprene seal modules. Leakage has occurred in these joints.

FIGURE 10 Modular elastomeric seal.
For the larger movements the heavier more elaborate modular systems such as the “Wabo Maurer” are performing well. The problem of separator beam hold down and tilting appears to have been solved.

A 1981 inspection by Azevedo (26) for the Kentucky Department of Transportation covered approximately 50 recent (one to four years) modular joint installations. Other than debris retention, no adverse criticisms were given. Azevedo concluded that both the molded neoprene rubber joints and the modular joints appeared to be improvements over the sliding-plate and finger dams. The high installation costs of the modular systems may be negated by improved performance and reduced future maintenance needs (26).

In 1984, Frederick (23) reported on the condition of 15 different joint systems in New York. With respect to five different modular devices that had been in service for less than two years, he reports satisfactory performance except for two different devices containing aluminum structural members that were not performing well. One was reported to be only slightly filled with dirt and debris but with noise levels that were excessive and joint headers that had cracked sufficiently to cause leaking. One of these devices was rebuilt in July of 1981 to correct the noise problem, but when it was inspected in the fall of 1981, the noise level was approximately the same as before the joint was repaired.

In 1985 Dahir and Mellott (6) reported on some of the early modular systems:

Unfortunately, experience with these systems in Pennsylvania and elsewhere... shows that, while some of these expensive systems have performed fairly well, most have had problems no less troublesome than those they were supposed to eliminate when using the conventional finger dam system.

With respect to an aluminum modular device they comment:

The...system has had anchorage problems, leakage, noise, debris accumulation and is subject to damage by snowplows. The snowplow damage which is associated with the excessive exposed rubber surface has made the...system unsuitable for installation in Pennsylvania.

In a 1986 report, Hamilton (5) expressed the attitude of many engineers when he commented that modular joint devices need to be further evaluated, because each installation in Maine was different from the previous one, in order to overcome some limitation of the previous design.

Finally, Kazakavich and Massimillian (27) of the New York State Department of Transportation reported on the evaluation of six different modular devices installed in one bridge. In a 1987 update of the original inspection and evaluation, it appears that, except for the aluminum devices, all of the systems are performing acceptably after six years of service. Kazakavich and Massimillian believe that, based on the annual inspections, the aluminum systems are not adaptable to snowplow environments (27).

JOINT EXPERIENCE SUMMARY

Bridge Deck Joint Evaluations

As part of this synthesis, a mail survey was conducted to determine the level of success that transportation departments have experienced in the use of various types of bridge deck joints. The survey asked the departments to identify the various types of joint designs and integral construction provisions that have been and are being used and to rate each type. Joint types to be considered were open joints, finger joints, slider plates, compression seals, strip seals, sheet seals, plank seals, foam or sponge seals, modular-sheet seals, modular-compression seals, drainage troughs, integral designs, and other.

A copy of the evaluation form, completed by a transportation department from an industrialized state with substantial vehicular traffic, is shown in Figure 11. The ratings on this form are not typical but they do indicate that where proper control is exercised throughout all aspects of product selection, testing, joint design, and construction, a bridge joint type can be selected and installed with sufficient integrity and durability to satisfy expectations and justify its continued use.

The evaluations contained on the forms received from 31 transportation departments have been summarized in Figure 12. This summary is useful because it shows in a very generalized way the relative success or failure that has been experienced by transportation departments in the selection and application of various types of bridge deck joints within the past 10 to 15 years. With the exception of the open joint, finger-plate joint, and possibly the unarmored compression seal, it should be understood that these ratings are intuitive averages for all joints of a particular type and not for a specific design within one of the types.

Many of the evaluations received could not be represented in this summary because the joints being evaluated were not sufficiently described or illustrated to place them in one of the categories. The “armored joint with elastomeric seal” evaluation given in Figure 11 is one such joint design. Also, notice in Figure 11 that the ratings for performance, durability, and maintenance are essentially the same. It was found that most of the evaluations from other departments were similar in that they also showed only minor differences in the ratings for these three aspects, differences that were not substantial enough to justify their inclusion in the summary of Figure 12.

Even though Figure 12 illustrates intuitive averages of transportation departments' success or failure in achieving design objectives with specific categories of deck joints, these averages do illustrate some very definite trends. For example, the histogram for “armored” compression seals shows that a large percentage of departments believe that they are achieving a high level of success in the design and construction of these joints. It shows that two departments believe that they are 100 percent successful and 11 departments have achieved an 80 percent level or better in their attempts to achieve successful applications with this type of joint. It also shows that some departments have had very negative experiences with this type of joint: Their success in achieving their design objectives is 40 percent or less. Because a large number of departments are experiencing such a high level of success, it is evident that the lack of success of other departments must be based on some aspect of selection, design, and construction control and not because of some inadequacy in the type of joint. The same chart does indicate, however, that the “unarmored” compression seal has characteristics that prevent the best design and construction control from achieving a level of success high enough to justify its continued use; only three departments were able to achieve their design objectives 70 percent of the time.
Reasons for Adverse Joint Ratings

As illustrated in Figure 11, transportation departments were asked to give the reasons for their adverse ratings and identify specific flaws or faults. The reaction to this request was generally very positive, although a few forms were received in which the explanations for poor ratings were not indicated. Most of the evaluations that were received did contain brief comments suggesting one or two primary causes for adverse ratings. For joints with the worst performance, these causes were few and very consistent from department to department. For joints with the best performance, no comments were generally given. For the joint type showing considerable interest (most applications) and with a broad variation in performance, a considerable number of different causes for adverse ratings were given. These comments serve to focus attention on those aspects of joint selection, design, and construction that deserve special attention, or at the very least, more attention. The comments that appeared to be repetitive or characteristic for a particular type of joint are summarized below:

A. Sheet Seal: This joint leaks at splices and is subject to snowplow damage. Construction and product design appear primarily responsible for the adverse ratings.

B. Plank Seal: This joint leaks at splices, is subject to snowplow damage, and experiences anchor bolt failures. Construction, product design, joint design, and maintenance all contributed to the adverse rating.

C. Strip Seal: This joint has poor curb geometry. The seals pull loose from anchor grooves, they split, and they are difficult to replace. Construction and fabrication appear primarily responsible for adverse ratings.

D. Poured Seals: The quality of the material and maintenance appears to be responsible for sealants that settle, deteriorate, and require periodic resealing.

E. Cellular Seals: Material quality and product design appear to be responsible for adhesive failures.

F. Compression Seals: Construction appears to be primarily responsible for the adverse ratings, although joint design makes
NOTE: These charts summarize transportation department evaluations of their own success in achieving design objectives when using various types of bridge deck joints and deck joint seals.

FIGURE 12 Summary of transportation department evaluations of bridge deck joints.
a substantial contribution along with material quality and product design. Major flaws include seals that leak, fall out, pop out, and exhibit compression set; joint edges that spall and are poorly spaced; anchors that fail (primarily straight studs welded to armor angles); and voids that are left under joint edge armor. Incidentally, compression seals had the greatest number of evaluations with no adverse comments.

G. Sliding-Plate Seals: Maintenance and joint design contribute to joints that leak and anchorages that fail.

H. Modular Seal: Product design and maintenance appear responsible for noise and snowplow damage.

I. Finger Plates: Maintenance and joint design appear to be responsible for joints that leak, clog, and need frequent cleaning.

In developing and simplifying the form for a generalized joint evaluation, one of the primary reasons for bridge joint failure (unanticipated structure movement) was inadvertently omitted from the form. Nevertheless, several evaluations that were received singled out this reason for some of the joint failures that were experienced. One engineer recognized this problem when he commented on the compression seal and its lack of tolerance for unanticipated movement:

The joint is designed only for temperature. Provisions for creep and the "natural settling out" of a bridge are not included in the joint design. Bridges move about after construction until they reach equilibrium with the environment in which they are located. In the process of this moving about or "settling out," expansion joints may open or close as much as $\frac{1}{4}$ to 1" from the original as constructed joint opening. In other words, the joint opening without regard to temperature may be more or less than the constructed opening. When the joint settles out to an open position without regard to temperatures, all contact between the sides of the seal and the concrete can be lost. At this point, severe leakage occurs and the seal can be removed by hand without effort.

It appears that one of the primary reasons for the undersizing of compression seals for bridge joints is manufacturer's catalogs that contain bridge joint seal sizes in $\frac{1}{4}$ in. (6 mm) increments and instructions for sizing the seals based on temperature effects only.

With a few exceptions, the evaluations received for this synthesis did not mention creep, abutment tilting, abutment settlement, or approach pavement growth as causes of joint failure, although these and other aspects of bridge design and construction are generally recognized as primary causes of bridge distress and bridge joint failure (5, 20, and 22). These aspects are briefly considered in Chapter Five.

QUESTIONNAIRE RESPONSES

In addition to the deck joint evaluation form, a questionnaire was included in the mail survey for this synthesis. Four questions elicited informative responses.

In response to the questions "What are your present typical maximum deck dimensions for decks without intermediate [deck] joints? maximum length? maximum width?" a fair number of responses were received that justify a summary and a few comments.

With respect to the maximum bridge deck length without intermediate transverse deck joints, most of the responses varied between 300 and 600 ft (90 and 180 m). There were two responses of 100 ft (30 m) or less, six greater than 600 ft, and nine with no length limitations. The responses are illustrated in Figure 13. It appears that most departments in limiting deck lengths to 300 to 600 ft are choosing those deck lengths whose movements can be accommodated by the usual or standard bearings and deck joints. For lengths greater than 600 ft, special and more complex bearing and joints are usually required.

With respect to the maximum bridge width without longitudinal deck joints, there were only two responses greater than 100 ft (30 m). Fifteen departments responded with deck widths between 40 and 100 ft (12 to 30 m) and 14 departments had no specific limitation (Figure 14). Longitudinal deck joints are used primarily to minimize the differential lateral movement between the superstructure and
substructure and the need for special bearing designs to accommodate such movement. With the development of the elastomeric bearings and other compound bearings that can accommodate both longitudinal and lateral movements, wider bridges and fewer longitudinal deck joints will probably become more commonplace in future bridge designs.

In response to the question "Considering the effects of [deck] joint width on the movement of motorcycles, bicycles...what are your maximum [roadway] joint openings?" maximum widths of transverse roadway joints for all departments were remarkably similar. Responses varied from 2 to 5 in. (50 to 130 mm), with 14 departments using 4 in. (100 mm) and only 1 department using a joint width greater than 4 in. (Figure 15). Six departments had no specific limitation. Fifty-six percent of those departments stated that their joint width was always measured parallel to the centerline of roadway.

The maximum width most generally specified for longitudinal roadway joints was 1 in. (25 mm) (Figure 16).

When asked the advantage of specifying alternative proprietary products, 85 percent of the responding departments said that the practice resulted in lower prices, one suggested that it encouraged new products, and two suggested that the practice ensured the ready availability of products.

With respect to the disadvantages of specifying alternative proprietary products, the questions provoked a number of different responses. They have been paraphrased in the tabulation below:

<table>
<thead>
<tr>
<th>No. of Depts</th>
<th>Paraphrased Responses</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>Difficult to determine quality and equality of alternative products.</td>
</tr>
<tr>
<td>4</td>
<td>Considerable plan preparation time needed to ensure receiving quality design.</td>
</tr>
<tr>
<td>4</td>
<td>Contractors will furnish cheapest product, not the best product.</td>
</tr>
<tr>
<td>2</td>
<td>Discourages product improvement.</td>
</tr>
<tr>
<td>1</td>
<td>Requires much time in approval process.</td>
</tr>
<tr>
<td>1</td>
<td>Lowest bid may not meet delivery dates.</td>
</tr>
<tr>
<td>1</td>
<td>Requires replacement of multiple types instead of one or two.</td>
</tr>
<tr>
<td>1</td>
<td>None, many types essentially equal.</td>
</tr>
<tr>
<td>11</td>
<td>No response.</td>
</tr>
</tbody>
</table>

From the above, it is apparent that many of the departments agree that the practice of permitting alternative designs ensures competitive bidding and suitable prices for proprietary products. However, the practice also makes it difficult to achieve quality products, inhibits product improvement, and has a number of other disadvantages. If the primary goal of transportation departments is to achieve economical deck joints for bridges, then the above responses suggest that they are generally successful. On the other hand, if the goal of transportation departments is to achieve effective and durable deck joints for bridges, then based on the foregoing responses and evaluations, that goal appears as elusive as ever.
SUMMARY

As illustrated in Figure 12 and as reflected in the comments, conclusions, and suggestions contained in much of the current research, bridge designers of short- and medium-span bridges are beginning to favor those joint types that are more an integral part of the bridge deck and less dependent upon mechanical anchorages. Much criticism has been leveled at finger-plate and plank deck joints. One concern is the hazard created for vehicular traffic when portions of these devices are dislodged or become loose because of inadequate size, anchorage, snowplows, etc. For example, after several finger plates broke off a finger joint on a bridge in Louisiana, a steel cover plate was welded to several of the remaining fingers as a temporary repair to cover the hole produced by the missing fingers (28). A few months later this plate and a finger plate to which it was welded were dislodged by a tractor-semitrailer. The plate punctured a fuel tank on the truck, spilling diesel fuel on the bridge deck. The slippery fuel was the primary cause of a series of skidding accidents involving 26 vehicles; 3 persons died and 18 were injured. In another state, a section of a prefabricated elastomeric plank seal came loose from a bridge deck joint because of failure of the drilled-in expansion bolt anchors. A motorcycle struck the joint component lying on the roadway and crashed into a guardrail. The driver sustained serious injuries and the passenger suffered severe head injuries that left him completely paralyzed, probably for life. Although these accidents were unusual, they do indicate that safety is a primary concern in the selection and design of bridge deck joints.

The limited success that has been achieved in the development of effective and durable seals for bridge deck joints has served to motivate some bridge engineers to consider another option. This option, the use of integral types of bridge construction, has allowed bridge engineers to move troublesome joints off of bridges and onto bridge approaches. Some background and experience from that movement is summarized in Chapter Three.
CHAPTER THREE

INTEGRAL CONSTRUCTION

INTRODUCTION

Integral construction may be defined as the practice of constructing bridges without deck joints. When using such construction to eliminate intermediate joints in multiple-span bridges, it is understood and accepted that the continuity achieved by such construction will subject superstructures to secondary stresses. Such stresses will be caused by the response of continuous decks to thermal and moisture changes and gradients, settlement of substructures, post-tensioning, etc. When using such construction to eliminate deck joints at abutments, it is likewise understood and accepted that such structures will, in addition, be subjected to secondary stresses caused by the restraint provided by abutment foundations and backfill against the cyclic movement of bridge decks. The justification for such construction is based on the recognition that for short- and medium-span bridges and for bridges of moderate lengths, significantly more damage and distress has been caused by the use of bridge deck joints than by the secondary stresses they were intended to prevent. In addition, the elimination of costly joints and bearings, and the details and procedures necessary to permit their use, generally results in more economical bridges. Consequently, more and more bridge engineers are now willing to relinquish some control of secondary stresses primarily to achieve simpler and less expensive structures with greater overall integrity and durability.

CONTINUOUS SUPERSTRUCTURES

In May 1930, a landmark paper by Cross (29) presented a simple and quick method for the analysis of integral-type structures such as continuous beams and frames. The method was quickly adopted by bridge engineers and the bridge design practices of many transportation departments began to change. Before Hardy Cross's moment distribution, multiple-span bridges were generally constructed as a series of simple spans (Figure 17a). Following the introduction of moment distribution, bridge engineers began eliminating deck joints at piers by providing continuous superstructures (Figure 17b). Figure 18 shows the beginning of the routine use of continuous construction, and the per decade increase in the number of transportation departments that have adopted the use of continuous construction. As shown in Figure 18, 26 of 30 departments responding to a mail survey conducted for this synthesis, or 87 percent of the responding departments, now routinely use continuous construction for short- and medium-span bridges.

Currently, Tennessee appears to be leading the way in constructing continuous bridges. The Long Island Bridge (Figure 2) at Kingsport, Tennessee, constructed in 1980, consists of 29 continuous spans without a single intermediate joint, for a total length of about 2700 ft (820 m). Movable bearings and deck joints have been furnished only at the abutments.

INTEGRAL BRIDGES

During the past two to three decades, many bridge engineers have become acutely aware of the relative performance of bridges built with deck joints at abutments and bridges built without them. In most respects, bridges without joints—integral bridges—have performed more effectively; they remain in service for longer periods of time with only moderate maintenance and only an occasional repair.

The growth and pressure generation of jointed rigid pavement has damaged many abutments and abutment backwalls (Figure 19) (Appendix A). The use of deicing chemicals to maintain dry pavements throughout the winter season has also had a significant effect on the durability and integrity of bridges built with deck joints. Open joints and sliding-plate joints of shorter bridges and open finger joints of longer bridges have allowed...
deck drainage, contaminated with deicing chemicals, to penetrate below deck surfaces, causing corrosion and deterioration. The performance of the joints that were designed to overcome these problems was often unsatisfactory, causing bridge engineers to consider other options.

Bridges with integral abutments suffered only minor damage from pavement pressure, were essentially unaffected by deicing chemicals, and functioned for extended periods without appreciable maintenance or repair. Moreover, they were usually less expensive to construct.

Consequently, more and more bridge engineers began to appreciate the merits of integral bridges for short to moderate bridge lengths. Gradually, design changes were made and longer bridges were built and evaluated. Ohio's initial 1946 length limitation for continuous concrete slab bridges was 175 ft (53 m). In a 1973 study of integral construction, four states responded that they were using steel bridges in the 201- to 300-ft (61- to 91-m) range and 15 states were using concrete bridges in the 201- to 300-ft range. In a 1982 study of integral abutment bridges, even longer bridges are reported:

Continuous steel bridges with integral abutments have performed successfully for years in the 300-foot range in such states as North Dakota, South Dakota, and Tennessee. Continuous concrete structures, 500 to 600 feet long with integral abutments have been constructed in Kansas, California, Colorado, and Tennessee.

Based on a mail survey conducted for this synthesis, 11 states indicated continuous bridges with integral abutments with lengths in the 300-ft (91-m) range; Missouri reported concrete and steel bridges in lengths of 600 and 500 ft (180 and 150 m), respectively, and Tennessee reported lengths of 800 and 400 ft (240 and 120 m) for similar bridges. Finally, as shown in Figure
20 of 30 transportation departments, or 67 percent of the respondents, are now using integral construction for continuous bridges. Appendix B shows some typical designs of integral abutments.

The attributes of integral bridges have not been achieved without cost. These bridges operate at very high stress levels, stress levels that cannot easily be quantified. In this respect, bridge engineers have become rather pragmatic. They would rather build the less expensive integral bridges and tolerate these higher stresses than build the more expensive jointed bridges with their vulnerability to destructive pavement pressures and deicing chemical deterioration. In 1985, Loveall (32) reflected this attitude:

In Tennessee DOT, a structural engineer can measure his ability by seeing how long a bridge he can design without inserting an expansion joint.

Nearly all our newer (last 20 years) highway bridges up to several hundred feet have been designed with no joints, even at the abutments. If the structure is exceptionally long, we include joints at the abutment but only there.

Joints and bearings are costly to buy and install. Eventually they are likely to allow water and salt to leak down onto the superstructure and pier caps below. Many of our most costly maintenance problems originated with leaky joints. So we go to great lengths to minimize them.

**STRUCTURE DISTRESS**

Responses to an early survey about the construction of continuous bridges with integral abutments indicated a rather widespread concern by bridge engineers for the potentially high stresses that would be present in the longer bridges with integral abutments (30). This concern more than any other appeared to be responsible for the early lack of enthusiasm for the use of integral abutments for the longer continuous bridges. Although the great majority of bridges with integral abutments exhibit adequate performance, many of these bridges operate at high stress levels. For instance, abutments supported on a single row of piles are considered flexible enough to accommodate thermal cycling of the ends of the superstructure and dynamic rotations of the spans induced by the movement of vehicular traffic. Nevertheless, piles of these abutments are routinely subjected to axial and flexural stresses approaching, equaling, and exceeding the yield stress of the steel (31 and 33). Occasionally, a combination of circumstances does occur that results in visible distress.

Responding to a 1973 survey, a number of bridge engineers said that some wingwalls of their integral abutments had minor cracks (30). This problem was corrected with the use of more generous wingwall reinforcing steel.

Considering the potential for high stresses, the experience with some of Ohio's integral abutments for continuous cast-in-place concrete slab bridges can be informative. Figure 21 illustrates the standard abutment design used for these bridges. This design has been used in essentially this same form for more than 40 years for three- and four-span slab bridges with lengths of up to 175 ft (53 m). In this design, the continuous deck slab is anchored to the abutment with a 2 1/2 X 7 1/2 in. (64 X 190 mm) unreinforced key. The short wingwalls of the abutment are supported on straight extensions of the abutment cap. In the design of this abutment, it has been assumed that the straight abutment cap supported on the single row of piles is flexible enough to respond to superstructure movements without substantial resistance, even for skews of up to 30 degrees and for...
bridge lengths of up to 175 ft (53 m). The deck slab was designed only for vertical dead and live loads. No additional provisions or allowances were made for stresses induced by initial deck shrinkage or for the resistance of the abutment to the cyclic thermal movement of the deck.

Figure 22 shows a different abutment design that was used in 1959 for two twin continuous four-span slab bridges. With the slab anchored to the abutment as described above, the abutments for those two bridges were provided with two rows of piles instead of the usual single row. In addition, the front row of piles was given a 1:4 batter. When compared to a single row of vertical piles, such pile placement and orientation substantially increased the resistance of abutments to the longitudinal shrinkage and cyclic thermal movement of the deck slab. The actual result of this change in abutment design is indicated by the fractured bridge slab seat shown in Figure 22, detail A.

The bridge seat failure occurred at three of the four abutments. The bridges had four continuous spans with a total length of 152 ft (46 m) face to face of abutments. The bridges were unskewed. Because the flexible design shown in Figure 21 has routinely been used for longer bridges and for bridges with skews up to 30 degrees, the fracture shown in Figure 22 can be attributed primarily to the use of a relatively rigid abutment design, a design that produced too much resistance to the shortening of the deck slab.

On several other bridges constructed according to the details shown in Figure 21, failures of the bridge slab seat have also occurred. However, these bridges used turn-back wingwalls instead of the standard straight wingwalls. In addition, a pile was provided to support the end of each wingwall. The two additional piles placed behind the single row of primary piles considerably stiffened the abutments longitudinally to such an extent that the bridge seats fractured shortly after the deck slabs were cast.

Except for these few slab seat fractures, the integral abutment design shown in Figure 21 has now been used for more than 40 years without other fractures. Because these fractures were attributed to the use of what can be classified as nonflexible abutment designs, no change was made in the standard key detail to make it more resistant to shear forces.

Currently, precast concrete or prefabricated steel superstructures are generally replacing small cast-in-place bridges in many states and provinces, so problems associated with initial shrinkage are gradually being eliminated. However, where cast-in-place construction continues to be used, flexibility of substructures continues to be a critical part of the bridge design. For example, Loveall (32) says:

Structural analysis of our no joint bridges indicates that we should have encountered problems, but we almost never have. Once we tied the stub abutment of a bridge into rock, and the structure cracked near its end, but we were able to repair the bridge and install an expansion joint while the bridge was under traffic. The public never knew about it. That was one of few problems.
The importance of considering restraint stresses in cast-in-place construction was indicated by Gamble (34), who discussed the cracking that occurred in a continuous concrete frame bridge. Even though the concrete in this structure was considerably below the specified cylinder strength and the shear reinforcement provided did not meet current requirements, the failure was attributed to the stiffness of the structure and its resistance to the shrinkage and contraction shortening of the bridge deck. Failures of this type emphasize the necessity of achieving flexibility in structural design and conservative reinforcement to withstand the potential stresses induced by foundation restraint and superstructure shortening.

The development of new forms of construction is bound to be accompanied by instances of structural distress, and this has certainly been true for continuous bridges with integral abutments. However, as is shown in Figure 20, the increased use of integral abutments suggests that 60 percent of the transportation departments are satisfied with the performance of integral construction and are using such construction in one form or another for longer and longer bridges. With continued care and consideration, it appears that the trend will continue.

**DESIGN DETAILS**

Appendix B shows integral abutment details for seven transportation departments. It is probably not accidental that there is a fair amount of similarity in these designs, because structural details from early successful designs are adopted and adapted by other bridge engineers for use by their departments. Even though there are similarities, there also are differences that reflect the types of bridges being built and the care and concern being given to the choice and development of specific details. It should also be realized that these sketches are “bare bones” presentations in that they do not reflect other important design aspects (skew, construction procedures, etc.) considered in the application of these details for specific bridges. These aspects cannot be illustrated and described in any detail in a presentation as brief as this one. Nevertheless, these aspects can have a considerable effect on the performance, integrity, and durability of the integral design, and they are given here for those engineers considering such designs for the first time. Passive pressure, pile stresses, approach slabs, and cycle control joints are four topics under which these considerations may be presented.

**Passive Pressure**

To moderate the passive pressure developed in an abutment backfill by the expanding bridge, bridge engineers have:

a. Limited the length of bridges to control the amount of backfill compression and consequently the magnitude of passive pressure.

b. Used porous backfill or selected granular backfill to control the magnitude of passive pressures.

c. Used uncompacted granular backfill for the same purpose.

d. Used approach slabs to prevent backfill compaction by vehicular traffic.

e. Used a void under the approach slab (Figure 23).

f. Used pressure attenuators (several inches of resilient material) between abutments and backfill.

g. Used moderate bridge skews to shorten the length of abutments exposed to passive pressure.

h. Shortened penetrations of abutments into embankments for the same purpose.

i. Used abutment benches in front of abutments to shorten the wingwalls exposed to passive pressure.

j. Used turn-back wingwalls for the same purpose.

k. Used embankments with 2:1 slopes or steeper to shorten wingwalls for the same purpose.

l. Used semi-integral designs, such as Figure 23, with or without a void under the approach slab, to eliminate passive pressure in backfill below the bridge seat.

**Pile Stresses**

Knowing that longitudinal forces in superstructures are somewhat directly related to the resistance of abutment foundations to longitudinal movement, bridge engineers have:

a. Generally limited the foundation of integral abutments to rather slender piles.

b. Limited the use of piles to only a single row.

c. Used vertical piles to minimize resistance to longitudinal deck movements.

d. Oriented the weak axis of steel “H” piles normal to the direction of movement.

e. Specified bored holes in stiff foundation materials and filled the holes with fine-grained material before driving the piles.

f. Provided rotational provisions in the abutment cap to prevent pile flexural stresses caused by the movement of vehicular traffic (Figures 23 and 24).

g. Wrapped pile tops of timber piles, or the concrete encased portion, with a resilient cover to prevent pile fixity (Figure 25).

![FIGURE 23 Integral abutment with void under the approach slab and movement provision in abutment cap (Ohio).](image-url)
h. Limited piles to steel "H" sections, sections that may have a longer fatigue life.

i. Reduced the penetration of abutments into embankments to reduce the resistance of surrounding soil to pile and abutment movement.

j. Controlled the skew to minimize pile deflection in both longitudinal and lateral directions.

k. Provided a deck joint between the substructure and superstructure to minimize foundation restraint to longitudinal movement (Figure 23).

To minimize dead load bending of piles for abutments without rotational provisions, the deck slab is cast before the top portion of the abutment is placed. In this way, dead load rotation of the superstructure takes place before joining the pile cap to the superstructure.

**Approach Slabs**

The FHWA Technical Advisory on integral construction (35) suggests:

Approach slabs are needed to span the area immediately behind integral abutments to prevent traffic compaction of material where the fill is partially disturbed by abutment movement. The approach slab should be anchored with reinforcing steel to the superstructure.

This appears to be sound advice especially for longer steel bridges and those bridges that confine the surface drainage on decks by means of curbs, barriers, raised medians, etc.

About 20 years ago, California conducted periodic inspections to monitor the performance of 27 bridges. These bridges contained both integral and semi-integral abutments with bridge lengths from 269 to 566 ft (82 to 173 m). The interim report for that study recommended the use of reinforced concrete approach slabs with all jointless bridges (30). It appears that this recommendation was based in part on the finding: "Some problems have occurred from erosion and piping of abutment support soils due to small amounts of water flowing down behind the abutments."

The report for a test by the South Dakota State University (36), which measured the pile stress in a full-scale model of an integral bridge, concluded in part: "The use of approach slabs which allow rotation and translation of the abutment and, if possible, avoid continuing compaction of the backfill is recommended."
When Ohio first adapted integral abutments to steel bridges in 1962, it was found that the constant cycling of the more thermally responsive steel bridges, friction between the approach slabs and the subgrade, and the infiltration of debris into the joints between approach slabs and bridge decks resulted in approach slabs gradually being pushed toward flexible pavement approaches or pressure relief joints and off of approach slab seats. Subsequently, drainage penetrated the joints, eroded abutment backfill, and caused settlement and faulting of the approach slabs. Installation of tie bars between approach slabs and abutments has eliminated this adverse behavior.

Currently, of the 20 transportation departments that use integral abutments on continuous bridges, 16 use reinforced concrete approach slabs. Of these 16, 8 tie the approach slabs to the abutments.

**Cycle Control Joints**

Figure 26 illustrates the design details being used by four states for pavement approaches to integral bridges. All of these designs accommodate the same functions, although the specific design details are substantially different. They each recognize cyclic movement of the bridge and, where the approach slab is attached to the bridge, cyclic movement of the approach slab as well. They each contain a reinforced concrete approach slab for spanning the abutment backfill, and they each provide for the growth and pressure that is generated in rigid approach pavements.

The initial appropriateness of these design details will be reflected in project costs and in the stress levels that will be developed in response to cycling of the bridges and growth of approach pavements. Ultimately, their actual effectiveness or success will be measured by their ability to function for long periods under traffic without periodic maintenance or major modifications. An evaluation of these designs and a background for that evaluation is given in Reference 37.

**SURVEY RESPONSES**

The survey for this synthesis elicited information about present integral construction practices. For the 20 of 30 transportation departments using integral bridges, only 11 questions out of 14 provoked other than random answers.

1. Unsymmetrical Bridges. No limitations have been established.
2. Abutment Settlement. Twelve departments have no limitations, two prohibit integral construction where settlement is expected, one limited settlement to \( \frac{1}{2} \) in. (13 mm) or less, and two mentioned the required use of piles to minimize settlement.
3. Pile Type. Ten departments used only steel "H" piles; three used steel "H" and prestressed concrete piles; one prohibited wood piles; the remaining six had no limitations.
4. Pressure Modification. Two departments mentioned the use of granular backfill, 2 bored holes for piles, 2 have used pressure attenuators of expanded polystyrene between abutment and backfill, 1 used a triangular wedge of concrete, and 13 responded that they do not attempt to modify passive pressure.
5. Approach Slabs. Sixteen departments use approach slabs and 8 out of 16 attached them to the abutments.
6. Deck Drainage. In addition to the use of approach slabs to protect abutment backfill from deck drainage, four departments used deck scuppers and drains adjacent to abutments. One department paved shoulders and one provided curbs and gutters. A number of departments also mentioned the use of pipe underdrains to collect and discharge water that penetrated the backfill.
8. Cycle Control Joints. Of the 16 departments using approach slabs, 7 had special details for joints between the approach slabs and the approach pavements (Figure 26).

9. Pier Foundation Pressures. All departments responded, that they use current national standard specifications for allowable foundation pressures for piers of integral bridges.

10. Superstructure Stressing. All departments except one neglected stresses induced because of the cycle restraint provided by integral abutments.

11. Curved Bridges. Seven departments had limitations on the use of integral construction for curved bridges. Four prohibited such use and three limited use to a 5 degree curvature or less.

A number of other questionnaires about integral bridge practices have been conducted in recent years (30, 31, 38). The responses reflect the policies, attitudes, and opinions of those engineers responsible for bridge design policies. They also show how some of those attitudes and opinions have changed during the last decade. Two reports (31, 38) also contain valuable bibliographies for those interested in a more in-depth study of current research on integral bridge behavior and abutment piling performance.

INTEGRAL CONVERSIONS (RETROFIT)

Following the trend toward the use of continuous construction and integral abutments, transportation departments are also beginning to convert existing multiple-span bridges from simple to continuous spans. This effort began in Wisconsin and Massachusetts in the 1960s, and 11 of 30 departments responding to the survey (about 35 percent) have converted one or more bridges from multiple simple spans to continuous spans. However, only two departments (Texas and Ontario) indicated that any conversions had been done in recent years. The Texas Department of Highways and Public Transportation responded:

In recent years, we have eliminated numerous intermediate joints. Generally, this is done while replacing the slab. We simply place the slab continuous across the bents. On a few occasions, we have removed only the joint and surrounding deck area, added reinforcing, and replaced that portion of the deck thus tying the adjacent spans together.

According to Wasserman (39), the Tennessee Department of Transportation has been actively converting simple-span bridges to continuous spans. To give this movement some direction, the FHWA has issued a Technical Advisory on the subject (40). That advisory in part recommends:

During the rehabilitation of bridge decks, it is recommended that the existing joints and structure layout be studied to determine which joints can be eliminated and what modifications are necessary to revamp those that remain to provide an adequate functional system. . . .

The Technical Advisory (40) states that design studies for joint elimination should consider the capability of the existing structural elements to properly function without joints, the grouping of several simple spans into a continuous unit by making the deck slab continuous through several spans or by making the stringers continuous, the replacement of obsolete and/or deteriorated bearings with elastomeric bearing devices, and, if the abutment is unrestrained, the development of a fixed integral condition full length for shorter bridges. An unrestrained abutment is assumed to be one that is free to rotate, such as a stub abutment on one row of piles or an abutment hinged at the footing.

Figure 27 appears to reflect the procedure described by Texas. Note that the detail shows that only the slab portion of the deck is being made continuous. The simply supported beams remain simply supported. For such construction, it would appear important to ensure that one or both of the adjacent bearings supporting the beams at a joint are capable of allowing horizontal movement. Providing for such movement will prevent horizontal forces from being imposed on the bearings by the rotation of the beams and continuity of the slab. As the Technical Advisory suggests (40): "...the joints and structure layout [should] be studied to determine...what modifications are necessary...to provide an adequate functional system."

Utah also has converted some simple-span bridges to continuous ones by using a design similar to the one shown in Figure 28. For deck slabs with a bituminous overlay, a waterproofing membrane can be used to waterproof the new slab section over the piers. With a design like this, it is understood that the deck slab would be exposed to longitudinal flexure caused by the rotation of the beam ends responding to the movement of vehicular traffic. However, for short- and medium-span bridges, the deck cracking associated with such behavior is judged by

---

FIGURE 27 Conversion of simple spans to continuous (Texas).

---

FIGURE 28 Conversion of simple spans to continuous (Utah).
some to be preferred over the long-term adverse consequences associated with an open joint or a poorly executed joint seal.

In new construction, the conversion of simple spans to continuous spans is rather commonplace. Figure 29 shows the design detail used by Wisconsin for prestressed I-beam bridges. A substantial concrete diaphragm is placed between the simply supported beams of adjacent spans and between the parallel beam lines. Then a continuously reinforced concrete deck is placed to complete the composite construction. This type of prestressed I-beam construction appears to be standard for many transportation departments.

Figure 30 shows the standard design detail used by Ohio to achieve continuity for simply supported prestressed box beams. These box beams are placed and transversely bolted together. Then, the continuity reinforcement is placed and the concrete closure placement is made.

Freyermuth (41) gives a rather complete description of the considerations necessary to achieve continuity in a bridge composed of a continuously reinforced concrete deck slab on simply supported precast prestressed beams. Conversion of existing bridges either by a complete deck replacement or by replacing portions of the deck adjacent to deck joints over piers can be accomplished by following the procedures developed for new structures. For existing bridges, creep effects will be negligible. Shrinkage effects for other than complete deck-slab replacements should also be negligible. Such continuous conversion not only eliminates troublesome deck joints, but the continuity achieved results in a slightly higher bridge load capacity because positive moments caused by live load and impact are reduced by continuous rather than simple-beam behavior.

The details and methods described above provide either partial or fully continuous behavior for live loads and superimposed dead load. If justified, continuity and composite behavior can be achieved for all loads by providing temporary intermediate supports for the beams; these supports are removed after all of the structural elements have been completed.

PROBLEMS

When considering the design of longer integral bridges, there are a number of problems that face the bridge engineer that have not been discussed or mentioned in current papers and reports on this subject. Because some of them can be solved or minimized by design details and/or construction procedures, they will be enumerated and briefly discussed below.

Bonding Concrete to Moving Members

When designing longer continuous bridges with integral abutments, especially steel beam or girder bridges, there is concern for the probability of a rapid drop in ambient temperature at the time when freshly placed abutment concrete is just beginning to set and bond to the beams of the bridge. There are several approaches that will allow this connection between the two to be made while the beams of the superstructure are moving in response to temperature changes and the foundations of the abutment are not.

a. Stage Construction. The abutment design by Tennessee for steel girder shown in Appendix B is one solution. First, the pile cap is placed up to the construction joint below the beams. Beam support bolts are cast in place in the pile cap and pass through slotted holes in the beam flanges but are not connected to the beams until after the pile cap has cured. After the cap has cured, the beams and cap are connected by tightening the nuts on the beam support bolts. Then, with the backwall forms fastened to the pile cap, the remainder of the abutment backwall can be placed. Also, the absence of backfill at this stage will allow the pile cap to respond more freely to movements of the superstructure.

b. Modify Thermal Responsiveness. Steel is much more responsive to rapid temperature changes than concrete. Consequently, by placing the concrete deck on the steel stringers of the bridge before the placement of backwall concrete, response of the composite deck is slowed, thereby providing more time for an initial set to be achieved when placing backwall concrete. This procedure would also eliminate pile bending stresses caused by placement of deck-slab concrete.

c. Control Placement Time. Because ambient temperatures reach a zenith during the afternoon and a nadir after midnight, placement of abutment backwall concrete can be timed so that placement is completed just before extreme bridge temperatures
when the length of the superstructure is relatively constant and the initial set is being achieved.

d. Control Deck Temperature. Probably the most effective way to ensure against damaging the fresh abutment backwall concrete is to specify placement of abutment backwall concrete while the deck is being sprayed with curing water. In some instances and in some localities, it may be desirable to combine all of the aforementioned details and procedures to ensure against damage to freshly placed concrete.

e. Movement Provisions. For the longer steel bridges, it may be desirable to consider a design that has an elastomeric bearing inserted between the integral backwall and the abutment foundation (Figure 23). Again, the abutment backwall is placed up to the construction joint below the steel girders and girder support bolts are cast in place. After the abutment support beam has cured, girders are attached to it by tightening support bolts and placing a nonshrinking grout between the support beam and bottom flange. The remainder of the backwall can then be placed without concern for the response of the bridge to rapid changes in ambient temperature.

f. Approach Slabs. Placement of approach slabs that are tied to the abutments of a long integral bridge presents a similar problem of connecting freshly cast members to a long structure responding to ambient temperature changes. As noted above for connecting the superstructure to abutment foundations, the deck can be placed and cooled with curing water. Also, the approach slabs can be placed before the day’s lowest temperature so that slab concrete is setting while the bridge length is relatively constant and beginning to expand. A smooth casting surface covered with polyethylene can be used to reduce friction between slabs and the subgrade. In addition, choosing an overcast day for casting approach slabs can further help to moderate temperature change effects.

Pile Fatigue

A number of research projects (30, 32) have demonstrated that steel piles of integral abutments are subjected to yield stresses caused by flexing of the piles in response to cyclic movement of superstructures. Extrapolating the results of fatigue research, which was conducted with stress ranges considerably below yield stress, it would appear that fatigue cracking should occur at a rather low number of stress cycles.

To protect against the chance of early fatigue damage, two design details have been used to minimize fatigue effects. As shown in Figure 24, a hinge can be installed in the abutment between the piles and superstructure; the flanges of the steel “H” piles can also be placed parallel to the direction of superstructure movement. In this position, piles are bent about their weak axis (Figure 31).

Placing a hinge in the abutment between the piles and the bridge deck will prevent bending of piles caused by movement of vehicular traffic, thereby reducing somewhat this aspect of pile flexural stressing. In addition, such a hinge placement moves the position of maximum bending moment down the piles to a position where the piles are laterally supported by confining subsoils.

Because each pile is being bent around its weak axis, fatigue cracking will probably commence at flange edges and propagate toward the web. The pile stresses are composed of an axial compressive component, P/A, and a flexural component Mb/2l; thus, fatigue cracking should occur only while the flexural component Mb/2l exceeds the axial component. Because the flexural component is proportional to the flange width “b,” cracking of the flange will reduce “b” and the flexural component, ultimately producing a hingelike condition where the axial compression component will protect the piles from further

![Fatigue of piles for integral abutments.](image)

**FIGURE 31** Fatigue of piles for integral abutments.
fatigue crack growth. At this stage, the remaining pile section (the web and root portion of the flanges) will have sufficient area and lateral support at point A to safely support the structure indefinitely.

Skewing of the structure will induce unsymmetrical flange cracking, but, again, a hinge should form while a substantial residual portion of the piles remains to support design loads.

Without the hinge, the cyclic movement of the bridge deck will force the piling into reverse bending, moving the point of maximum moment to the top of the piles adjacent to their connections with the abutment cap. Fatigue crack growth at this location would be accelerated due to additional flexural stresses induced by movement of vehicular traffic.

Obviously, the above discussion is a rationalization based on extrapolation of current research and experience. Much additional research is needed to describe pile performance in integral abutments and to determine if there are sufficient stress cycles associated with thermal movement of bridge decks to result in fatigue crack growth as described above. Without such research, it is questionable if current experience with short- and medium-length bridges unless safeguards are included in the designs to ensure adequate long-term performance.

Considerations such as those suggested above have also been partly responsible for the pile-type limitations that transportation departments have imposed on integral abutment bridges.

**Pile Durability**

The literature of integral abutments is mute concerning the subject of pile durability. However, a number of circumstances are present in the various designs (Appendix B) that suggest that this subject needs investigation and study.

To make integral abutments more responsive to cyclic movement of the bridge decks, the bottoms of such abutments have been moved up to within close proximity of the embankment surface—in one case, flush with the embankment surface. Various types and thicknesses of stone or rock surface protection, which are rather porous ground covers, are specified for the surfaces adjacent to abutment footings.

The tops of integral abutment piles can be exposed if there is substantial consolidation of abutment embankment. The pile tops will also be exposed by horizontal flexing of the piles within the penetrated soils.

As noted in the survey responses concerning the use of approach slabs, many departments do not use approach slabs on their integral bridges. Where they also use deicing chemicals on the roadway, roadway drainage containing these chemicals will penetrate the porous backfill and come in close proximity with supporting piles. Careful selection and protection of piles is required in such circumstances to ensure long-term durability.

**Frost Heave**

For abutments located close to the ground surface and considerably above frost depth, especially those supported by short piles in semipervious soils, the effects of frost heave also need consideration.
WATERPROOFED DECK JOINTS

INTRODUCTION

To protect the main structural elements of a bridge from the corrosive effects of deck drainage, especially drainage containing deicing chemicals, deck joints can be sealed or surfaces below the joints can be waterproofed. Sealed joints have been described and discussed in Chapter Two. Waterproofed joints will be described and discussed in this chapter.

ELASTOMERIC DRAINAGE TROUGHS

On the Fort Ancient bridge, which carries I-71 over the Little Miami River in Morrow County, Ohio, four major deck joints were used to accommodate the dynamic and cyclic movement of the superstructure. At two deck joints, located above pin-connected truss chord joints, overlapping sliding-plate joints were provided. At two other joints, located above truss hangers, open finger joints were provided. During the design of the structure in the early 1960s, it was decided to abandon the use of metal drainage troughs, which were commonly provided under open finger joints. This decision was provoked by the poor performance experienced with such troughs on similar bridges, which rapidly clogged and filled with sediment. Consequently, the Fort Ancient bridge was designed with a generous number of deck scuppers and connecting downspouts and open finger joints without drainage troughs.

By 1970, only five years after the bridge was opened to traffic, it was found that the paint system on structural steel under the deck joints had failed and truss chords and secondary members were beginning to corrode. It was also found that concrete under the bearings at the tops of the tall pier columns was beginning to deteriorate. Drainage that was penetrating the deck at the four major joints (located at the quarter points of the span) was flowing down the haunched bottom chords and washing around the bearings and down the pier columns. To prevent further concrete deterioration in these critical areas, it was decided to retrofit the structure with waterproof deck joints.

To accomplish this, elastomeric troughs were designed to completely enclose the bottom of the finger joints. With such enclosure, the troughs would continue to function even if they filled with debris. At the edge of the deck, the troughs were furnished with galvanized steel collectors and downspouts. These are accessible from above by the use of a truck-mounted boom and inspection platform. Figure 32 shows a schematic detail for these troughs.

For this structure, the enclosure of the finger joints by elastomeric troughs is so complete that a vacuuming effect is caused by the movement of vehicular traffic. The sides of the troughs flex back and forth with the passage of each automobile and slap together with the passage of trucks and trailers. Apparently, this continual trough flexing and vibration jostles sediment and debris toward the downspouts. Except for the removal of an occasional clump of brush, these troughs stay relatively clean. This bridge was one of the first in the United States to be retrofitted with prefabricated elastomeric drainage troughs (41).

The National Experimental and Evaluation Program for "Waterproof Bridge Deck Joint Seals" (NEEP Project No. 11) documents the experiences of 40 state transportation departments for a period of seven years. In the 1977 final report of that project (3), the summary contained in part the following:

Steel toothed and sliding plate joints, although not intended to be leakproof, have had long service life in bridge decks. A number of States have continued to use those joints but have incorporated full joint width neoprene troughs with and without downspouts to collect water and carry it off the structure. A common problem with any trough, however, is the tendency to fill up with dirt and debris. An active maintenance effort is needed to keep the troughs cleaned out. Based on a field review by FHWA Head-
When considering the use of elastomeric drainage troughs, three aspects—sheet types, elastomers, and trough details—are of primary concern.

**Sheet Types**

The Fort Ancient bridge was equipped with elastomeric troughs fabricated from 4.0 ft (1.2 m) wide, 1/8 in. (3 mm) thick sheets of nylon reinforced neoprene, elastomeric sheet material originally developed for use as pump diaphragms. Currently, elastomeric sheets being specified range from 1/32 in. to 1/4 in. (2.4 to 6 mm) thick, and are both unreinforced and reinforced. Reinforcement consists of nylon, polyester, or cotton fabric. Specified tensile strengths range from 1100 psi (7.6 MPa) to 800 lb/in. (140 kN/m). Some plans specify only that an elastomeric sheet shall be furnished. Others specify the elastomeric material, sheet thickness, type of reinforcement, and various physical properties. The more complete specifications include resistance tests to ensure long-term durability.

In a 1980 Technical Advisory (43), the FHWA suggests:

Elastomeric material for troughs should be low (50 or 60) durometer, synthetic fabric reinforced. Unreinforced sheets would be acceptable where experience has shown them to be adequate.

**Elastomers**

Following the first trough applications using neoprene sheets, numerous other elastomers were developed and used in similar applications. One of the major fabricators of elastomeric products for transportation applications said that there are probably as many as 20 or more different sheet materials now being used for trough applications. Based on conversations with a number of other manufacturers, it appears that these 20 different materials are composed of four basic elastomers. The original chloroprene elastomer (neoprene) has been supplemented by natural rubber, ethylene propylene diene monomer (EPDM), and nitrile butadiene (Buna N). The Lord Corporation has recently prepared and published a "Selection and Service Guide for Rubbers" chart (44). An excerpt from that chart is reproduced in Table 1. This excerpt provides a general evaluation of some of the more important properties of elastomers now being used in trough applications.

Like most other materials, the basic elastomers must be mixed or compounded with other materials to facilitate the manufacturing process and to achieve the desired attributes in the product for specific applications. Vulcanizers are added to convert the elastomer from a thermoplastic to a thermosetting material. Accelerators are added to speed up curing time. Plasticizers and softeners are added to reduce the time and power required to prepare the elastomer for shaping. Tougheners are added to increase wear resistance. Fillers or extenders are added to increase bulk and reduce the unit price. Finally, coloring agents are added to produce an appropriate appearance. Depending on the manufacturing processes and properties desired in the finished product, the mix of these constituent materials is altered or adjusted until the most appropriate compound is found that will furnish the desired properties in the finished product at the lowest cost to the manufacturer. For a product such as a neoprene sheet, the finished sheet may be composed of from 45 to 65 percent neoprene, 13 to 14 percent plasticizers, and 20 to 35 percent fillers. Vulcanizers, accelerators, tougheners, etc. make up the remainder or about 5 percent of the total mixture. The quality of an elastomeric compound appears to be directly related to the percentage of basic elastomer in the compound and inversely related to the percentage of filler.

Because of compounding differences from manufacturer to manufacturer, it should be realized that when an elastomeric sheet, neoprene for example, is specified, a sheet with neoprene as the basic elastomer will probably be furnished. However, the physical, mechanical, and environmental properties of the finished sheet can vary over a wide range, depending upon the number of manufacturers and the skill of their compounders and material processors.

It should also be realized that sheets received from any one manufacturer will be subject to some change from time to time because of variations in the manufacturing process, errors in the mixing of constituents, and changes in the original compound formula or processing procedures.

Finally, elastomeric products produced by some manufacturers may actually be composed of elastomeric blends. For example, a compound containing an elastomer consisting of 51 percent neoprene and 49 percent of some other elastomer may be called neoprene because the major portion is neoprene. For certain applications, an elastomeric blend may in fact furnish the most desirable performance characteristics at the lowest cost. Such a blend should be acceptable. However, where the performance characteristics have not been established and quantified, it may be prudent to limit initial purchases to compounds containing only one basic elastomer.

To ensure a minimum level of quality from the low-bid contractor, project plans preferably should contain a comprehensive performance specification for elastomeric sheets. Such a specification would permit various sheet manufacturers to choose the elastomer or elastomeric blend of their choice and the percentage of other constituents that will facilitate the production and fabrication processes and yet attain or exceed the specified test requirements.

Because such a performance specification is not yet available, transportation departments are currently purchasing elastomeric sheets for troughs using specifications that have been developed by various sheet manufacturers for these applications. Several specifications are given below to indicate the present variety in the specifications that are now being used to purchase elastomeric sheets for trough applications.

a. Elastomer Compound for trough and mat shall be in accordance with Table A or B of Article 718.20 of the Standard Specifications except the tensile strength shall be 1500 psi minimum or it shall be EPDM (ASTM D 2000, Line call-outs 3BA, 515, A14, B13, F17, C12, K21, Z1, Z2).
TABLE 1
SELECTION AND SERVICE GUIDE FOR ELASTOMERS (44)

<table>
<thead>
<tr>
<th>Chemical Type</th>
<th>Natural Rubber</th>
<th>Neoprene</th>
<th>Ethylene Propylene Diene Monomer</th>
<th>Nitrile or Buna N</th>
</tr>
</thead>
<tbody>
<tr>
<td>Common or Trade Name</td>
<td>Natural Polyisoprene</td>
<td>Chloroprene</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ASTM D1418 Designation</td>
<td>NR</td>
<td>CR</td>
<td>EPDM</td>
<td>NBR</td>
</tr>
</tbody>
</table>

1. **PHYSICAL**
   - Density (gm/cm³): 0.93, 1.24, 0.86, 1.00
   - Hardness range (Shore A): 30-100, 40-95, 30-90, 20-90
   - Bondability: A, A, B, B-A, B

2. **MECHANICAL**
   - Tensile strength (max psi): 4,500, 4,000, 3,000, 3,500
   - Abrasion resistance: A, B-A, B, B
   - Flex resistance: A, B, C, B
   - Tear resistance: A, B, B, C
   - Deformation capacity: A, A, B, B-A
   - Elasticity: A, A
   - Resilience: A

3. **THERMAL**
   - Recommended max temp (°C): 70, 100, 125, 100-125
   - Low-temp stiffening: B, B, B, B
   - Heat-aging resistance: B-C, B-A, B, B

4. **RESISTANCE TO:**
   - Weather: C-B, B, A, C-B
   - Oxygen: B, A, A, B
   - Ozone: C-D, B, A, C-D
   - Radiation: B, B, B, B
   - Water: A, B, B, A

A = Excellent  B = Good  C = Fair  D = Poor  NR = Not Recommended

1. The higher the density, the more rubber is required to make a given part. For example, compare neoprene and natural rubber. Even at the same price per pound, neoprene would be more expensive to use.

2. While tensile strength per se is not necessarily important, retention of strength at elevated temperatures suggests retention of other mechanical properties as well.

3. Abrasion-resistance ratings apply to a wide range of temperatures as well as type of abrasion (such as rubbing and impingement).

4. A high resistance to crack growth indicates good general durability—necessary where physical abuse is expected.

5. Tear resistance, along with crack-growth resistance, is desirable where physical abuse is expected.

6. Rubbers that strain-crystallize at extreme deformations are much more durable in impact than those that don't. Low-temperature flexibility also helps improve impact performance.

7. A high deformation capacity usually indicates a high fatigue resistance to flexing.

8. The lower the permanent set, the better the structural integrity and the better the retention of initial dimensions.

9. The higher the resilience, the less the degradative heat buildup in a flexing or dynamic situation.

10. Good low-temperature flexibility is a must for most shock absorbers. The first jolt is critical, regardless of subsequent softness.

### Physical Properties (EPDM)

<table>
<thead>
<tr>
<th>Property</th>
<th>Requirement</th>
<th>ASTM Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Durometer—Shore A</td>
<td>50 Min.</td>
<td>D 2240</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>1500 psi Min.</td>
<td>D 412</td>
</tr>
<tr>
<td>Ultimate Elongation</td>
<td>200% Min.</td>
<td>D 412</td>
</tr>
<tr>
<td>Compression Set 22 Hours @ 212°F</td>
<td>35% Max.</td>
<td>D 395</td>
</tr>
<tr>
<td>Compression Set 158°F @ 212°F</td>
<td>25% Max.</td>
<td></td>
</tr>
</tbody>
</table>

| Low Temperature Brittleness      | D 2137 Pass       |
|---                               |                   |
| Ozone Resistance Procedure      | D 1149 No Cracks  |
| Bond During Vulcanization       | D 429 80% R      |
b. Elastomeric troughs shall be shop fabricated from elastomeric sheets using vulcanized (with heat and pressure) joints. Hole spacing shall be based on templates furnished by the steel fabricator or on actual stud locations as measured in the field. Holes shall be cleanly made preferably by the use of a cutting die.

Elastomeric sheet shall be Dupont's Fairprene NN-0003, a \( \frac{1}{32} \) inch thick sheet of nylon-reinforced neoprene or a suitable alternate. The one ply material shall conform to ASTM D 751 and the following:

<table>
<thead>
<tr>
<th>Property</th>
<th>Requirement</th>
<th>ASTM Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hardness, durometer point</td>
<td>65 ±</td>
<td>D 2240</td>
</tr>
<tr>
<td>Tensile strength, mm.</td>
<td>1,200 psi</td>
<td>D 412</td>
</tr>
<tr>
<td>Elongation, mm.</td>
<td>350%</td>
<td>D 412</td>
</tr>
</tbody>
</table>

The specifications given above should not be considered representative of the elastomeric sheets being used by transportation departments. They are given here merely to indicate the wide disparity in specifications that are now being used to qualify the purchase of such materials. Specification A is the most complete in that it also contains ASTM tests to ensure long-term durability. Although the specification shown is for an EPDM elastomer, the reference to Tables A and B of the Standard Specifications is intended to give the contractor the option of furnishing neoprene, natural rubber, or EPDM.

### Trough Details

Figures 33 and 34 show elastomeric troughs used at abutments to waterproof open finger joints and sliding-plate joints, respectively. Figure 35 shows details for elastomeric troughs used at piers to waterproof open finger joints. In both figures, the structural steel portions of the joint designs have a long history of satisfactory service. That is one of the major attributes of a trough type of design and one of the main reasons why such designs are favored by so many transportation departments. They may have some negative characteristics that inhibit their use (debris retention, generally inaccessible structural surfaces, etc.), but they ensure long-term performance of the roadway surface without the risk of traffic-induced structural failures.

In all of these designs, elastomeric troughs are fastened from the outside to facilitate removal, repair, or maintenance from below. The designs are geometrically simple, even where the joint above is complicated by the presence of raised medians, barriers, curbs, and sidewalks.

With respect to the design of elastomeric troughs, the 1980 FHWA Technical Advisory, "Expansion Devices for Bridges" (43), suggests the following:

Elastomeric troughs should be continuous full width of the bridge including curb and parapet area when the joint is over a pier or at an abutment and elsewhere where a closed drainage system is required. Under toothed joints located where drainage freefall would be acceptable, the troughs need only be intermittently positioned to deflect drainage clear of supporting superstructure, including bearings.

For maintenance and/or replacement, troughs should be accessible from below. As-built contract plans should show developed layout with full dimensions, including size and locations of all holes for each segment of elastomeric trough.

Use stainless steel for bolts, studs, and washers as required to attach troughs.

Elastomeric troughs should be sloped at least 1 inch per foot with intermediate breaks and collectors as needed. [The] use of tapered side plates [is suggested] to provide required slope and maintain a constant elastomeric trough depth and sheet width.

The 1 inch per foot slope should work in most cases; however, if the normal rainfall is relatively low or the expected quantity

![FIGURE 33 Elastomeric trough under finger joint at abutment (Pennsylvania).](image-url)
of roadway debris would be high, such as near a sand and gravel, or similar, operation, steeper slopes would be in order. In all cases, the steepest slope possible should be used in an effort to gain the best self-cleaning potential. However, it should be remembered that at the upper end of a trough slope where drainage flow will be minimal, some debris will accumulate and may require maintenance every few years. A steeper slope at this location would likely be beneficial.

**METAL DRAINAGE TROUGHS**

In the early 1970s, the steel industry announced a new structural steel with high corrosion resistance that could be used bare in bridge applications. This steel, subsequently designated as ASTM A 588, has been widely used for both bridges and buildings. However, when used for drainage containment applications, where moisture (especially moisture containing deicing chemicals), sediment, and debris accumulate, the corrosion resistance does not develop and A 588 steel will corrode. Many drainage troughs were constructed using bare ASTM A 588 steel and all have had to be cleaned and painted or replaced.

Since that time, steel drainage troughs continue to be used but coating systems are usually specified. Newly developed paint systems are specified. Galvanized steel is used. For some applications, galvanized steel is given a top coat of bituminous material to enhance the coating durability. These bituminous coatings are similar to those being used to provide additional protection for galvanized steel culverts.

For the inaccessible galvanized steel surfaces of the trough shown in Figure 34, the following supplemental coating was specified:

Bituminous coating shall conform to the latest edition of Federal Specification WW-P-405b, Coating F, brush or trowel applied prior to its application. Surfaces shall be wiped with a suitable solvent using clean, dry cloths to remove contaminant traces. Surfaces shall be dry and warmer than 40°F. during coating application. Coating thickness shall be not less than 1/16 of an inch.

With respect to coating metal drainage troughs to protect them from deicing chemical corrosion, Ring (45) emphasizes the need to choose appropriate surface treatments and primers for metal substrates to achieve good adhesion and quality coatings to ensure long-term durability.

The high cost and limited durability of coated steel suggested the use of bare stainless steel for drainage troughs. Figure 36 shows a recent design that uses 3/16 in. (4.8 mm) thick stainless steel sheets for such an application. In this design, the engineer has specified a more expensive material but is assured of a durable surface, a surface that does not need to be applied and renewed periodically.

Figure 37 shows a drainage trough under an open finger joint. It is apparent that it was designed for the containment and control of water and not the sediment- and debris-laden deck drainage typical of most bridges. For these systems to be effective, generous sizes and slopes must be provided.

For bridges in urban areas, especially where troughs are readily accessible to vandals, a metal trough may be more appropriate than an elastomeric trough.
SURVEY RESULTS

Based on the responses to the survey for this synthesis, those departments that use drainage troughs currently favor the use of elastomeric troughs (Table 2). Galvanized steel troughs placed second with painted steel following a close third. Although not mentioned in the responses, stainless steel is indicated in the table because a number of departments are known to have used such steel for trough applications. Pennsylvania reported the use of both fiberglass and elastomeric troughs.

With respect to "performance," "durability," and "maintenance," the ratings of the troughs were consistent from department to department. Performance of all troughs was rated about the same (adequate); galvanized steel was given a slightly higher durability rating than the other materials; and maintenance received the lowest rating for all troughs.

One of the three departments that indicated the use of open joints without troughs commented that the substantial accumulations of snow and ice precluded the use of drainage troughs in their jurisdiction.

WATERPROOFING

For small bridges where it is impracticable to provide drainage troughs to contain and control deck drainage that penetrates unsealed deck joints, the structure surfaces below such joints can be waterproofed. Such waterproofing can be done in three ways: Concrete surfaces can be shaped to facilitate drainage, surfaces can be coated or sealed, and surfaces can be cleaned.

To protect bridge bearings and bearing seats of an abutment from drainage that leaks through the deck joint and flows down the abutment backwall, bearing seats and the bridge seat can be shaped to divert drainage away from bearing seats. Bearing seats can be elevated above the adjacent bridge seat by 1 or 2 in. (25 or 50 mm); a small channel, sloped to drain laterally, can be provided at the base of the backwall; and the bridge seat between bearing seats can be sloped toward the face of the abutment.

Figure 38 shows an old abutment where some of this shaping was provided. The beam bearing seats were not elevated and a lateral drainage groove was not furnished. However, the bridge seat between the bearing seats was sloped 3/4 in. (19 mm) toward the abutment face. Visible in the photograph is the result of 33 years of service. The bridge seat concrete is substantially deteriorated yet the photo suggests that partial shaping did offer some protection for the bearing seats because the most extensive deterioration is evident between the bearing seats. Note also that the backwall surface, which was exposed to the same drainage as the bridge seat, exhibits only moderate deterioration. This
suggests that the surfaces that retain debris and remain wet for longer periods of time will deteriorate at a faster rate. Shaping concrete surfaces between bearing seats did not prevent deterioration from occurring but it was effective in partially shielding the more important part of the bridge seat, the bearing seats, from similar damage.

In addition to shaping concrete surfaces to facilitate drainage, surfaces can be coated to make them more impervious to moisture absorption and chloride intrusion. There are numerous products available for this purpose. NCHRP Report 244 (46) identified the three most successful materials as a silane, a methyl methacrylate, and a high-solids epoxy. However, the evaluations were done on specific formulations, and extending the test results to an entire generic classification could be extremely dangerous.

Although at the time of this research there were numerous materials available to transportation departments for the protection of concrete surfaces, only a few of them were found that could be depended on to function as effective sealants against water absorption and chloride intrusion (46). Transportation departments are now using NCHRP Report 244 as a guide in the purchase of sealants by brand name alone, or test methods given in the report are used as the basis for the purchase of sealants regardless of brand name.

For those transportation departments that used open deck joints, the question “What type of coatings and sealers do you use to protect exposed surfaces from the effects of deck [drainage]?” elicited the following responses:

<table>
<thead>
<tr>
<th>Concrete Surface Sealants</th>
<th>No. of Departments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Epoxy</td>
<td>6</td>
</tr>
<tr>
<td>Epoxy or Silane</td>
<td>2</td>
</tr>
<tr>
<td>Epoxy, Chemtrate or Hydro 2030</td>
<td>2</td>
</tr>
<tr>
<td>Epoxy, Mod. Urethane, Polyurethane</td>
<td>1</td>
</tr>
<tr>
<td>Epoxy or Linseed Oil</td>
<td>1</td>
</tr>
<tr>
<td>Silanes or Linseed Oil</td>
<td>1</td>
</tr>
<tr>
<td>Silane</td>
<td>1</td>
</tr>
<tr>
<td>NCHRP Report 244</td>
<td>1</td>
</tr>
<tr>
<td>Linseed Oil</td>
<td>2</td>
</tr>
<tr>
<td>Two Coats of Chlorinated Rubber</td>
<td>1</td>
</tr>
<tr>
<td>None</td>
<td>5</td>
</tr>
<tr>
<td>Total</td>
<td>23</td>
</tr>
</tbody>
</table>

Surfaces of steel under and adjacent to unsealed deck joints should also be coated to protect them from corrosion. Substantial improvements have been made in the quality of steel coating systems developed within the last decade (47). In addition, considerable attention has been focused on the methods used by fabricators and field painters to clean steels before coating applications. Abrasive cleaning is routinely specified and, for some coating systems, the texture or profile of the abrasively cleaned surface must be controlled between rather narrow limits. Some states have provided steel surfaces with hot-dip galvanized coatings (48). Other states are looking at the metallocizing process to provide more corrosion-resistant surfaces.

With respect to coatings used to protect steel surfaces from corrosions, many transportation departments responded to the question “What type of coatings do you use to protect exposed surfaces from the effects of deck [drainage]?” in rather vague generalized language. Nevertheless, the responses received are somewhat informative:

<table>
<thead>
<tr>
<th>Steel Surface Coatings</th>
<th>No. of Departments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zinc rich paint</td>
<td>4</td>
</tr>
<tr>
<td>Inorganic zinc rich paint, epoxy</td>
<td>2</td>
</tr>
<tr>
<td>Inorganic zinc or galvanizing</td>
<td>1</td>
</tr>
<tr>
<td>Painting</td>
<td>3</td>
</tr>
<tr>
<td>Extra paint</td>
<td>1</td>
</tr>
<tr>
<td>Epoxy paint</td>
<td>1</td>
</tr>
<tr>
<td>Epoxy mastic paint, lead base paint</td>
<td>1</td>
</tr>
<tr>
<td>Epoxy mastic, H.D. galvanizing</td>
<td>1</td>
</tr>
<tr>
<td>Paint or galvanizing</td>
<td>2</td>
</tr>
<tr>
<td>None</td>
<td>1</td>
</tr>
<tr>
<td>No answer</td>
<td>9</td>
</tr>
<tr>
<td>Total</td>
<td>26</td>
</tr>
</tbody>
</table>

Finally, sediment- and debris-covered surfaces exposed to drainage appear to deteriorate at a much faster rate. Consequently, it would appear that periodic cleaning of horizontal surfaces once or twice a year would be a very cost-effective way to attain long-term durability.
CHAPTER FIVE

JOINT MOVEMENT FACTORS

INTRODUCTION

In an investigation of highway bridge movements by Moulton (49), data on 314 highway bridges distributed across 39 states and 4 Canadian provinces were tabulated, compared, and evaluated. The summary of that report concluded in part:

The results of this study have shown that, depending on type, length and stiffness of spans and the type of construction material, many highway bridges can tolerate significant magnitudes of total and differential vertical settlement without becoming seriously overstressed, sustaining serious structural damage, or suffering impaired riding quality. However, it was found that many of the bridges involved in this study were susceptible to structural damage, particularly to joints and bearings, from relatively small horizontal movements of abutments and piers, and the level of those damages was more severe when the horizontal movement was accompanied by vertical movement.

Moulton’s report is concerned with movements other than the thermal and dynamic movements usually associated with deck joints of bridges. It is concerned with movements that are rarely, if ever, mentioned in handbooks, considered during the design and detailing of bridge deck joints and joint seals, or considered in joint-seal surveys or bridge joint evaluation reports. However, the bridge movements that Moulton investigated are “real,” and consequently, the probability of such movements should be considered in all aspects of bridge design and especially in the design of bridge joints and bearings.

To understand the bridge joint experience, it is necessary to be aware of factors that have an appreciable effect on bridge movements and how the movements associated with these factors are reflected in the behavior of bridge deck joints. It is not possible in this synthesis to describe and discuss in any depth all of the various factors that affect the movement of bridge deck joints. However, a mention of some of the most important factors and a brief discussion of them seems appropriate.

ABUTMENT MOVEMENTS

Earth Pressure

Figure 39 shows a wall-type abutment supported on piling. For the design of such a structure, an engineer considers that the loads and forces above plane A-A are resisted by piling below plane A-A. Occasionally, passive earth pressure in the soil at the toe of the footing above plane A-A is included in the analysis as furnishing resistance to the horizontal forces above plane A-A. However, implicit in the design of an abutment such as the one shown in Figure 38 is that plane A-A is an absolutely rigid plane. Any movement of the abutment with respect to plane A-A is usually not considered and the deck joint size is based only on the joint movement produced by the response of the superstructure to temperature changes.

An abutment can be thought of as a vertical cantilever beam fixed at the footing, at plane A-A, resisting a triangular-shaped load caused by active earth pressure. Such a cantilever beam will be deflected horizontally at the top, $\Delta_A$, a deflection that may have an effect on the size and performance of the bearings and deck joints of the structure (Figure 40a).

In addition, horizontal movement parallel to plane A-A, $\Delta_B$, will be produced by the application of earth pressure (Figure 40b). Recognition of such movement is suggested by assumptions of active pressures being associated with expanding soils and passive pressures with soils that are being compressed.

Finally, there is the vertical movement, $\Delta_D$, and counterclockwise rotational movement, $\theta_C$, of the abutment with respect to plane A-A (Figure 40c). The piles are compressed by the application of a vertical load, resulting in vertical movement of the footing. Nonuniform pile compression caused by a resultant force that is eccentric with respect to the center of gravity of the pile foundation will result in rotation of the abutment footing. This vertical and rotational movement of the abutment...
footing is reflected in movement of the top of the abutment wall with respect to the bridge deck (Figure 40c). Consequently, these movements also have an effect on the size and performance of the bridge bearings and deck joints.

There is also the lateral and rotational movement of the abutment caused by the vertical consolidation and lateral movement of the subfoundation soils against the abutment piling, movements that are initiated and magnified by the surcharging effect of embankment construction adjacent to the abutment.

The abutment movements that are illustrated in Figure 40 cannot easily be quantified. Nevertheless, the bridge engineer responsible for a particular bridge design needs to be aware of these potential movements and provide for them in the design. Their effect on the structure, particularly the bridge's joints and bearings, can be minimized by controlling the construction sequence of the structure so that most of these movements occur before construction of abutment backwalls and before placing the superstructure bearings on the bridge seats. For example, if the abutment breastwall is constructed up to the bridge seat and the abutment backfill is then placed up to the subgrade elevation, most of the movements illustrated in Figure 40 will occur before placement of abutment bearings and deck joints.

Long-term consolidation of the embankment and subfoundation and creep of concrete associated with sustained earth pressure will induce additional movements of abutments. Those movements will mirror the initial movements but to a considerably lesser extent. The design engineer will provide for these additional movements by selecting a type of bearing and deck joint that will accommodate them, movements that will generally tend to close deck joints.

The movement of wall-type abutments caused by differential earth pressures associated with placement of embankments and abutment backfill has its counterpart in stub-type abutments constructed at the top of steep embankments. With respect to such construction, Moulton states (49):

The data suggests that more consideration needs to be directed to the potential effects of horizontal movements during the design stage, particularly for perched and spill-through abutments on fills and piers located near the toe of approach embankments.

In discussing effects of embankments on abutments he also states:

One of the most common causes of foundation movements revealed by this study was movement of underlying embankment materials and/or their foundations. In fact, this basic cause of movement was identified as being either totally or partially responsible for the movements of over 150 foundation elements. These data suggest that, when abutments or other substructure units are to be founded on embankments, ... whether on spread footing or piles, the embankment should be specifically designed to resist postconstruction deformation, either by settlement of embankment material, consolidation of underlying foundation soil, or sliding associated with slope or foundation instability. ...

The [use] of a "waiting period" after embankment construction, and before the construction of the foundation elements [including driving of piles], is highly recommended.

Without such an attempt to control the movement of perched or spill-through abutments, the post-construction movement of abutments not only has an adverse effect on superstructure stresses, it has a significantly adverse effect on the behavior and durability of bridge deck joints and joint seals.

If the joints and bearings of a bridge are to function for extended periods without distress, then the bridge design itself must ensure relatively static bridge seats so that an appropriate deck joint and joint seals can be chosen and designed. If it is impracticable to provide such bridge seat control, then the bearings and joints chosen for the structure should be such that they can tolerate substantial movement without distress and/or costly maintenance.

**Pavement Pressure and Growth**

As described in Appendix A, the growth and pressure associated with jointed rigid pavements should be considered in the design, maintenance, and repair of bridges, and in the evaluation of the performance of various types of bridges including those containing deck joints.

In providing new bridges and retrofitting old bridges with deck joints at abutments, pressure-relief joints are installed in approaches to protect the abutments and the deck joints from pavement pressure and growth.

Where the type of pressure-relief joint being used is not completely effective, the type of deck joint chosen should be one that can tolerate complete closure without distress. Obviously, compression seals or plank seals and the more complex modular seals are inappropriate for such applications.

Where pavement pressure and growth are probable and where the experience with relief joints has not been acceptable, then bridge engineers are giving more consideration to the use of integral bridges, which can withstand these pressures without appreciable distress or damage.

**Settlement**

Vertical abutment settlement has an adverse effect on the performance of some types of deck joints, especially sliding-plate joints like those illustrated in Figure 41. Bearing and joint sliding surfaces are in contact before settlement; after vertical settlement, the contact remains at the joint more so than at the bearing (Figure 41). This is especially true where structural parts of the joint are stiff enough and connections are strong enough to support the full dead- and live-load reactions of the superstructure.
Consequently, where substantial vertical settlement of an abutment takes place, the bridge superstructure will actually be suspended from the abutment backwall with the total dead- and live-load superstructure reaction being supported by the deck joint and the top front edge of the abutment backwall. For joints where there is not continuous contact between the joint sliding surfaces (and this is probably the case with most joints), the total superstructure reaction will be concentrated at those points along the top of the backwall where there is surface contact.

Generally, the sliding-plate deck joint, such as shown in Figure 8, has not been designed to withstand the reaction forces associated with vertical settlement. Consequently, such joints experience distress or localized anchor bolt failure when appreciable settlement does occur. Conversely, the deck joints shown in Figures 9b and 9c and other similar designs will tolerate substantial vertical settlement without distress.

Tilting of abutments caused by earth pressure, nonuniform consolidation of subfoundation soils, pavement growth, and erosion of foundation soils can also result in the condition illustrated in Figure 41. In each case, deck joints will be subjected to superstructure reactions they have not been designed to resist. Construction waiting periods are used to ensure that most embankment settlement and subsurface consolidation has occurred before abutment construction. Pressure-relief joints are installed in approach pavements to minimize pressure on abutments caused by the long-term growth of approach pavements. Curbs, gutters, paved shoulders, approach inlets, and other drainage-control devices are used to ensure adequate erosion control adjacent to abutment foundations.

Where such provisions cannot be made or provided and where the probability of vertical settlement and/or tilting of abutments is probable, then the design of the bridge's bearings and deck joints must provide for these conditions; otherwise, early joint maintenance and repair is probable.

SUPERSTRUCTURE MOVEMENTS

Longitudinal Movements

Thermal Changes, Moisture Changes, and Creep Shortening

The American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges (50) contains a number of general paragraphs about longitudinal bridge movements, bridge joints, and bridge bearings. Each transportation department has established its own practice and developed its own procedures for implementing AASHTO provisions.

The California Department of Transportation has developed comprehensive policy statements to control both design and construction of bridge deck joints. An excerpt from its design memorandum, No. 7-10, is given in Appendix C. With respect to longitudinal movements, a "Joint Movement Calculations" form (Form DS D129) is given, along with sample computations for a particular bridge application. As shown on this form, California uses thermal coefficients of 0.0000065 and 0.0000053 for steel and concrete superstructures, respectively, to calculate the thermal movement range for each deck joint. In addition, coefficients of 0.06, 0.12, and 0.50 are given to estimate the anticipated shortening (in./100 ft) caused by shrinkage, creep, and elastic effects for conventional concrete, pretensioned concrete, and post-tensioned concrete members.

A research report by Stewart (51) of the California Department of Transportation gives the results of bridge joint movement measurements for 230 bridge joints made over a period of three years. The report concludes in part:

If the widely used premise that 0.0000065 is the thermal coefficient of expansion for steel girder superstructures is accepted, then it must be concluded that 0.0000053 is the thermal coefficient of expansion for concrete girder superstructures.

In the application of these thermal coefficients, the yearly ambient temperature range of the bridge site is used. This conforms with the findings in a research study by Cosaboom and Kozlov (52) of the New Jersey Department of Transportation. Although this research was limited to the measurement of only two bridges, the results also agree with results obtained in other work (53). With respect to temperature measurements, the New Jersey report concludes in part (52):

Displacements at the joints of bridges are closely related to air temperature at the bridge site. Based on the environmental conditions encountered, for the types of bridges investigated in this study, the effects of solar radiation, precipitation or moisture, re-radiation, and other inherent or environmental parameters may be disregarded with respect to determining bridge movements. Air temperature may be assumed to be the "effective" temperature of a bridge for design purposes.

For the design of continuous bridges, including those with intermediate deck joints, the California memoranda (Appendix C) also contain a standard form for the computation of a structure's "neutral point" or point of no movement. Although based on certain simplifying assumptions, this form is a convenient guide that can be used for the estimation of bridge joint movements for other more complex structures.

For the design of cycle-control joints on the approaches to integral bridges, the California movement study also contained measurement data for such bridges (51). According to Figure
7 of that report, it appears that the movements of integral bridges are about 70 percent as much as comparable nonintegral bridges provided with standard joints and bearings.

Movement Tolerances

As suggested in California's Memo No. 7-10 (Appendix C):

Many variables affect the actual movement in an expansion joint. Some of these are:

1. The actual point of no movement may differ from the calculated point of no movement.
2. The actual shortening may differ from the estimated shortening.
3. The actual minimum and maximum structure temperatures may differ from the extremes listed in the preliminary report.

Therefore, the above factors, plus the fact that the Movement Rating [M.R.] test method does not provide a dependable safety factor explains the need for making conservative assumptions in design. For these reasons, the calculated M.R.'s shall be rounded up to the next half inch.

The movement computations given and discussed above are based on the premise that substructure members are built on static and secure foundations. Consequently, the probability of substructure movement, as described above under "Earth Pressure" and "Settlement," should also be considered when selecting the deck joint type and estimating the probable deck joint movements. Because compression seals and small size plank seals provide only limited amounts of movement for the sizes selected, they generally should not be used where uncontrolled movements are probable and where allowances for such movements have not been included in joint movement calculations.

Because compression and some plank seals have little tolerance for unanticipated bridge movements, the joint widths for these seals are adjusted during construction to account for installation temperatures other than normal. The difficulty of correctly accounting for the difference between bridge temperature and ambient temperature and for making appropriate adjustments during construction should also be considered when choosing construction tolerances and seal sizes for specific bridges and bridge lengths.

Growth

On older structures constructed with concrete-filled steel-grid floors, especially floors that have had deicing chemical applications applied to them for a period of 20 years or more, the high probability of floor growth should be considered when choosing the most appropriate type of deck joint and deck joint seal to use if it has been decided to retrofit such floors for new joints and joint seals. One recent paper on this subject contains valuable background for such consideration (54).

Rotational Movements

Most designers, when considering superstructure end rotations, assume that joint movements associated with these rotations are parallel to longitudinal movements induced by temperature, moisture, and creep effects. For square structures, this assumption is correct. For skewed structures, it is not. For skewed structures, joint movements caused by end rotation are more normal to the centerline of the joint and not parallel with the longitudinal centerline of the deck. For short-span bridges with moderate skews, the differences are of little consequence. But for large skews, especially for long-span structures, the end rotations or abnormal rotations are of significance.

Recent bridge bearing developments have provided compound bearings to accommodate the abnormal rotations of superstructures at skewed supports (55). However, some of the more complex modular joints do not adequately provide for superstructures that rotate about an axis that closely parallels the centerline of the joint.

To recognize these effects, designers of long-span structures can do three things to improve joint performance. They can:

1. Minimize structure skew,
2. Place deck slabs before installing deck joints, and
3. Choose the joint type best suited to accommodate abnormal rotations caused by live load.

In this way the effects of abnormal rotations are minimized and the performance and durability of the bridge's joints are improved.

BEARING-INDUCED MOVEMENTS

Not mentioned in design specifications and rarely mentioned in bridge joint evaluation reports is joint distress and damage caused by a lack of coordination of the movements of deck joints and bearings, especially movements of bearings adjacent to sliding-plate joints.

For many sliding-plate joints, designers have assumed that the bridging plate is continuously supported on both sides of the joint and that the contiguous sliding surfaces of the joint move parallel with each other. There are many conditions affecting the continuous contact of these surfaces that can cause these surfaces to move in other than parallel directions; thus, the premise of continuous support is not correct. Briefly discussed and illustrated below are several examples of uncoordinated bearing and joint movements and the effects of these movements on the integrity and durability of bridge deck joints.

Although binding and distress of sliding-plate joints is described and illustrated, similar binding and distress will be experienced by supported finger-plate joints because they have similar structural characteristics. Some of the smaller and larger modular elastomeric joints will also be similarly affected by steep roadway grades. However, the somewhat flexible construction of these joints will lessen the effects described below to an extent that will depend on the actual configuration of the structural components of the joints.

Nonparallelism

Many designers assume that expansion and contraction of the bridge deck is parallel to the surface of the deck and thus choose a joint design that has been developed to accommodate the magnitude of that movement.
For example, Figure 42 shows an abutment bridge seat and backwall detail for bridges located on steep grades. The detail specifies that the face of backwall should be built normal to the roadway surface and the joint armor for the backwall should be placed with the upper sliding surface of the armor parallel with the roadway surface. However, the drawing also shows horizontal bridge seat surfaces, which means that movement will take place in a horizontal direction.

Figure 43 is an enlarged joint detail showing both the abutment and superstructure parts of a deck joint. Consider the movement of point A, a point on the lower sliding surface of the superstructure angle. Because bridge bearings move on horizontal bridge seat surfaces when the bridge superstructure responds to changes in ambient temperature, the ends of the superstructure supported on the bearings and the superstructure part of the joint armor also move horizontally. The sliding surfaces of the abutment and superstructure armor are in contact at normal temperature and point A is in contact with the surface of the abutment armor angle. As the temperature increases, the superstructure and point A on the superstructure angle will tend to move a distance of $\Delta_{AH}$ to point B if the abutment angle was not in the way of such movement. However, with the abutment angle in the position shown, point A must move along the sliding surface toward point C, a movement that has a vertical component at C equal to $\Delta_v$. Such a movement would tend to lift the superstructure off of the bearings, causing the superstructure reaction at the abutment to pass through the sliding surfaces of the joint instead of through the abutment bearings. This reaction, in conjunction with the friction on the sliding surface, would tend to resist movement of point A toward point C. The horizontal frictional force developed on the sliding surface between point A and point C, a force equal to $Ru$ (where $R$ equals the coefficient of steel on steel), would be opposed by an approximately equal force on the fixed pier. Consequently, as the superstructure tends to expand an amount equal to $\Delta_{AH}$, point A will tend to move on the sliding surface toward point C, lifting the superstructure and developing frictional resistance. As frictional binding occurs, the remainder of the movement $\Delta_{AH}$ will occur by deflection of the fixed pier.

In summary, orienting the sliding surface of a deck joint in a plane that is not parallel with the moving surface of the adjacent bearings will cause binding of the joint and ultimately result in structure distress and potential damage. If the fixed pier is flexible enough, expansion movement will occur, but most of it will take place away from the joint.

With respect to the joint of Figure 43, binding as described above will be modified by clockwise rotation of the superstructure, including the attached portion of the deck joint, caused by the dynamic response of the superstructure to the movement of vehicular traffic. Such rotation will cause separation of sliding surfaces, remove frictional contact, and facilitate expansion of the superstructure at elevated temperatures. This, in turn, will minimize forces on and deflection of the fixed pier. Depending on the relative magnitude of the grade and dynamic rotation of the bridge superstructure, some joints will be able to accommodate full expansion of the superstructure, whereas others will bind after only slight expansion.

**Rotation**

Illustrated in Figure 44 is a rather commonplace type of bridge design where a sliding-plate deck joint is located between spans supported by structural rockers and bolsters. The live-load counterclockwise rotation of the left span will cause separation of joint surfaces. This will subject the bridging plate to cantilevered wheel loads, and for plates not designed as cantilever members, such loading can cause distress in bridging plate bolts, welds, and joint anchors.

During construction, there are beam erection and deck-slab placement procedures that can cause distress in such joints. For bridges where the joints are permanently attached to the bridge beams before deck-slab placement, clockwise rotation of the right span caused by placement of the deck-slab on that span will cause the bridging plate to be pried upward. Such behavior is usually not anticipated, and these rotations can easily fracture bridging plate bolts, welds, and deck-joint anchors.

To avoid such distress during erection, deck-slab construction joints are used adjacent to deck joints to allow slab concrete to be placed before installing the joints. To account for live-load rotations and separation of the contiguous sliding surfaces of
deck joints, bridging plates are designed as cantilever members and their attachments and joint anchorages are designed as cantilever supports.

To minimize rotational effects on deck joints, the horizontal distance between the centerline of joints and centerline of bearings is made as small as practicable.

Construction tolerances, lack of straightness of structural material, and live-load rotations associated with the longer bridge spans mean that the resulting space under the bridging plate will allow deck drainage to penetrate such joints.

The ultimate solution to intermediate deck-joint problems is the complete elimination of such joints by the use of continuous construction.

**Compressive Deformation**

Substitution of elastomeric pads for the bearings in Figure 44 would result in a deck joint that does not provide for vertical compression of the elastomeric bearings. Although a joint design can be made that could survive such an application, it is probably prudent to avoid the use of elastomeric bearings with joints of this type.

Depending on the relative amount of span rotation and bearing compression, placement of the deck slab on the left side span could result in the bridging plate being pried upward. Where such behavior was not anticipated in the design, this prying action can easily fracture bridging plate bolts, welds, and joint anchors.

In bridges employing elastomeric bearings, joint distress as described above can be avoided by using some type of elastomeric joint seal, a type that can accommodate differential vertical movement.

**Hanger Swing**

The use of the sliding-plate deck joint in conjunction with beam hangers, as illustrated in Figure 45, occasionally results in joint distress and damage as described above for other joint 2nd bearing combinations.

Because of the movement characteristics of the hanger and the position of the bridging plate, either expansion or contraction of the bridge causes lifting of the suspended span. Lifting of the span will cause the bridging plate to be pried upward, which can easily fracture bridging plate bolts, welds, and joint anchors. Also, the upward force on the bridging plate has its counterpart in an opposing or downward force on the hangers and pins, another aspect of this combination that could have serious consequences for the integrity of the bridge.

**SUMMARY**

As has been described and discussed, there are many situations and conditions that have an adverse effect on the performance of bridge deck joints. Many bridge engineers and others who have been charged with evaluating performance of bridge deck joints have recognized the adverse consequences of earth pressure, pavement pressure, settlement, and snowplows, but rarely mentioned are the adverse consequences associated with the improperly coordinated movement of deck joints and bearings. Many of these adverse conditions and situations have been described in this section because they help to explain why so many deck joints and deck joint seals have failed to perform as expected. Much of the joint distress being experienced today is caused by bridge design and joint design decisions over which bridge contractors and joint manufacturers have little or no control. Consequently, unless these aspects of bridge design and construction are given more consideration, much of the adverse bridge joint experience will continue to be repeated.
Possibly, it will come to be recognized that abutments of some bridges experience so much construction and post-construction movement that the use of rather fragile bridge deck joints and joint seals is not appropriate. On the other hand, if bridge deck joint seals are to be provided and their long-term integrity is deemed to be essential, then provisions will have to be made to control deck-joint movements. Waiting periods following embankment construction may have to be used to ensure that most vertical consolidation and some horizontal translation of foundation strata have occurred before abutment construction. Also, placing of joints and bearings may have to be delayed until after the abutment backfill has been completed and stabilized.
CHAPTER SIX

RESEARCH

INTRODUCTION

A significant amount of bridge-joint research has been accomplished in the last 20 years, with varying degrees of usefulness. Many technical reports of significant interest are somewhat obscure, generally unknown, or not readily accessible. Other significant research is of a proprietary nature and, as such, is not available to transportation department engineers to help ensure the quality of materials and quality construction. Consequently, many young engineers now being given responsibilities in the field of bridge joints will be hard pressed to become familiar with the best technical literature that has been produced.

There is a need for a comprehensive annotated bibliography of "significant" research on bridge joints and joint materials. Also, some well-directed basic and applied research would go a long way to improve the bridge-joint experience.

RESEARCH VARIABLES

Many bridge-joint evaluation reports are actually evaluations of bridge performance as that performance is manifested in the behavior of the joints. As such, they are more an evaluation of the functional effectiveness of a transportation department's bridges than an evaluation of specific types of bridge joints. This is so because of the variables involved in the behavior or performance of the specific bridge joints being evaluated.

Price (56) was particularly concerned about this problem and he devoted a considerable part of his report's introduction to a discussion of the "Factors Influencing Performance" of bridge deck joints. He says:

The investigations have identified a wide range of factors that influence the performance of joints. These differ between and within joint types and frequently it is a complex combination of factors which affect serviceability and not necessarily the same combination or sequence for all the joints. Often the action of one factor instigates another.

Price itemizes the factors under the topic headings of structural movement at the joints; thermal movements; traffic-induced movements; traffic over the joint; joint design; materials used; bond and anchorage; condition of substrate; detritus, foreign matter, and corrosion; weather and temperature; site preparation and workmanship; and performance of the bearings.

Price is not alone in his concern for the variables that affect the performance of bridge joints. Alexander (22), in reporting the result of a Kansas Task Force analysis and evaluation of bridge deck joints, qualified their bridge deck joint evaluation in part:

...failures have occurred which were not the fault of the Manufacturers or their devices. Due to the [lack of] attention to details by K.D.O.T., 12 percent of the joints have failed. Six joints have failed because the Maintenance Department failed to maintain adequate pavement pressure relief joints in the approach pavements. Nine joints have failed because the Construction Department failed to install joints within the 1/4" installation tolerance. ... Twenty two joints have failed because the Design Department failed to specify a device with adequate movement.

Recognizing the need to control variables in bridge-joint evaluations, Kazakavich and Massimilian (27) of the New York Department of Transportation installed six different modular deck joints in the Collar City bridge in Troy, New York. These joints were installed in the same year and under the same supervision. They will all be subject to the same traffic and environmental conditions, and each will be exposed to the same maintenance procedures and equipment. They will all be inspected at the same time and be evaluated by the same person. Not all variables have been controlled, but many of the major variables that affect the behavior of a bridge have been controlled so that the evaluation of the performance of these modular joints should result in a rather clear insight into the relative merits of these devices for applications similar to the one under consideration.

If the bridge-joint experience is to improve, then bridge-joint research should be under the supervision of engineers with extensive bridge experience and research variables should be controlled. If variables cannot be controlled, then resulting evaluations should be appropriately qualified.

RESEARCH EVALUATIONS

Even experienced researchers have difficulty evaluating the performance of bridge joints. Swanson (18) illustrated this problem when he reported:

None of the devices in Table B can be classified as acceptable or unacceptable at this date because the causes of failures have not been conclusively proven to be related to installation, snowplow, traffic, or simply the failure of the device.

In another case, an initial evaluation of a prefabricated elastomeric seal, which blamed snowplows for the rapid deterioration of nine plank joints in two Cleveland, Ohio, bridges, was later revised after a thorough investigation to include a list of
seven major flaws, of which snowplows were a minor contributor. It was ultimately determined that:

1. The product design was conceptually flawed.
2. The plank type seals were inadequately vulcanized.
3. Reinforcing was furnished but it was for the wrong joint model.
4. Crucial installation instructions were not furnished.
5. Joint movement range was incorrectly calculated.
6. Joint width was not properly controlled.
7. Snowplows did some superficial damage.

These joints were removed with less than one year of service life. The joint model was subsequently revised by the manufacturer and presented to other transportation departments as a new and improved model deserving their consideration.
CHAPTER SEVEN

RECOMMENDATIONS AND CONCLUSIONS

RECOMMENDATIONS FOR DESIGNS AND DETAILS TO AVOID

The recommendations herein are based primarily on a summary of comments received from the survey.

Designs

The following joint designs may cause problems and should be avoided:

1. Deck joints with little or no tolerance for unanticipated movements being used in bridges with insecure foundations.
2. Joints in bridges adjacent to jointed pavement not protected with adequate pressure-relief joints.
3. Joints not designed for movements that are highly probable.
4. Skew-sensitive joints used in large skew applications.
5. Sliding-plate joints used in applications where differential vertical movements (including rotational effects) are probable.
6. Joints that have not been subjected to successful load tests before their installation on highway bridges.
7. A bridging type of joint that cannot survive the application of substantial vehicular overload.
8. Joints with a wide expanse of elastomer on the roadway surface used in snowplow environments.
9. Sliding-plate or finger joints with sliding surfaces not placed parallel with the sliding surfaces of the bearings.
10. Joints with aluminum structural members exposed to the roadway surface and snowplows.
11. Joints with expansion anchor bolts used on bridge roadways.
12. Joints without substantial joint edge armor and substantial armor anchorage.
14. Sliding-plate joints that depend on countersunk structural bolts for their integrity.
15. Elastomeric joint seals that depend on adhesive bonding of transverse field splices for their integrity.
16. Joints with seals not installed in one continuous piece.
17. Joints with sharp changes in vertical alignment in curb and gutter areas.
18. Joint widths that are set before the deck slabs are placed.

Joint Details

The following details of joint design and construction are apt to cause problems and should be avoided:

1. Joint widths that are set before abutment backfill is completed.
2. Vents or bleeder holes not furnished in joint edge armor, which makes it extremely difficult to expel entrapped air during concrete placement.
3. Concrete under joint armor not hand packed to ensure elimination of voids under the armor.
4. Concrete buffer strip not finished above the joint edge armor, which will expose the armor to snowplow impacts.
5. Concrete buffer strip not used adjacent to joint edge armor. The wear and rutting of asphalt overlays will expose joint edge armor to snowplow impacts.
6. Anchor-bar location ineffective in anchoring legs of armor angles. Stud-weld area furnished not sufficient to resist snowplow impacts.
7. Anchor-bar location improved but area of connecting weld not sufficient to resist snowplow impacts.
8. Length of armor anchors not sufficient to engage concrete reinforcement.
9. Width of joint armor not sufficient to achieve adequate anchor-bar attachment.
10. Construction joint not provided to allow superstructure dead-load rotation before final joint adjustment.
11. Construction joint not provided in abutment backwall in lieu of construction joint in superstructure to allow joint adjustment after superstructure dead load placement.
12. Segmented joint seals with bonded splices.
14. Joint armor for wide bridges not provided with rigid splices.
15. Compression seals not set low enough in the joint to avoid protruding above the roadway surface when the joint closes and compresses the seals.
17. Joint widths that are set before the deck slabs are placed. On long steel superstructures exposed to direct sunlight, the deck slab will insulate the deck and retard thermal movement and it will also integrate the stringers and minimize differential stringer movement at the joints.
18. Joint widths for unarmored compression seals cut without a gang saw to ensure a constant joint width.
19. Joints with unnecessarily complex curb geometry.
20. Subdrains not provided for asphalt wearing surface.
21. Armor not thick enough to avoid welding distortion.

CONCLUSIONS

This synthesis has considered three aspects of deck joints and other movement provisions for bridges. First, a brief and gen-
eralized description of various types of bridge deck joints and
deck joint seals was given along with the evaluations of de-
partment of transportation engineers of their own success in
achieving design objectives. The second aspect discussed was
the development of continuous bridges and bridges with integral
abutments, which are efforts by bridge engineers to solve deck
joint problems by constructing bridges without joints. Last, the
synthesis examines the methods currently being used to water-
proof open deck joints from below or to waterproof the struc-
tural elements below unsealed deck joints. Other information
has been included in the synthesis to establish a context or
background upon which the relative success or failure of these
can be considered and understood.

The last several decades have seen a growing concern with
deicing chemical deterioration of bridges. One of the successful
efforts in this area has been the elimination of deck joints
through the use of continuous bridges and bridges with integral
abutments. Such designs have gradually evolved during a trial
or development period of 50 years or more. The early successful
performance of integral details and the durability of short- and
medium-span bridges containing those details served to en-
courage subsequent similar applications for longer bridges. On
the other hand, only limited success has been achieved with the
multiplicity of essentially proprietary joint devices, which, ex-
cept for elastomeric compression seals, have been evolving dur-
dring a development period of less than 20 years. In 1965, Johnson
(60) warned new engineering graduates about the probability of
incomplete engineering:

There is real danger when new development in construction is
introduced with the appearance of advanced design and novel
shapes, but with neither a background of successful usage in
prior construction nor the imaginative foresight that the expe-
renced engineer gives to every type of load and structural re-
sponse that may be involved in the development of a new
structural form. Such incomplete engineering is leading to an
increasing number of structural failures, both during and after
construction.

Although Johnson used generalities in speaking about “new
developments,” he foresaw the failures experienced by the bridge
engineering profession in adapting for bridges many of the new
and novel devices conceived by product distributors and man-
ufacturers.

The poor joint performance and many joint failures experi-
enced by transportation departments cannot be used as a general
condemnation of the joint devices themselves. Their quality,
mechanical efficiency, material integrity, strength, toughness,
and durability depend on the individual who conceived them,
the specialist who developed them, the individual who promoted
them, the person who selected them for use, the designer who
prepared the project plans, the contractor that fabricated and
installed them, and all of the inspectors who approved them
and their installation. The success or failure of a particular
design is directly related to the expertise of all of the individuals
who contributed to the design’s conception, development, pro-
duction, adaptation, fabrication, testing, and installation. In
other words, it is the level of ability and experience that each
of these individuals brings to a particular design that determines
the success or failure of that design. Consequently, when con-
sidering the performance of these designs, the focus should be
on the human element. Improving the performance of the human
element appears to be one of the most effective ways to achieve
improved concepts, details, specifications, policies, and, ulti-
mately, bridge longevity.

If the experience of transportation departments with bridge
deck joints and deck joint seals is to improve, the individuals
having responsibility for conceiving and adapting bridge deck
joint details for bridges must be experienced bridge engineers
who are thoroughly familiar with bridge behavior and structural
design. Otherwise, the experience of the last two decades sug-
gests that the quality and durability of bridges and bridge deck
joints may improve only marginally.
REFERENCES


*This advisory was canceled on June 30, 1982, and is no longer mandatory for projects requiring FHWA approval.
APPENDIX A

PAVEMENT PRESSURE AND BRIDGE DISTRESS

PAVEMENT PRESSURE

Pavement blowups are a clear indication of the high pressures that can be generated in restrained rigid pavement. However, pavement blowups may be indications of localized high pressures and not indications of generalized longitudinally oriented compressive stresses existing throughout extensive lengths of pavement and distributed both laterally and vertically throughout the pavement cross section. A brief and simplified explanation should help to illustrate that extremely high compressive stresses, distributed throughout the pavement cross section, are probably the rule rather than the exception.

Figure A-1 illustrates the cyclic movement that occurs at pavement contraction joints. This movement is caused by response of the pavement to changes in the pavement’s moisture content and temperature. Also illustrated is the effect that incompressible debris has on this cyclic movement.

In drying, following a wet curing period, concrete shrinks a maximum of about 0.0005 of its length. This is the well-established average value of the total free shrinkage from a saturated to a dry condition. Most shrinkage can be recovered by a thorough rewetting. Because concrete pavement in contact with a subgrade probably retains a substantial amount of moisture, a coefficient of 0.0003 may be used to represent the initial shrinkage of the average pavement.

After being cast, concrete pavement responding to a loss of moisture, to cooling after the heat of hydration, and to a lowering of the ambient temperature tends to shorten. This shortening is resisted by the tensile strength of the concrete. Ultimately, the tensile strength is exceeded and the pavement cracks at precut contraction joints. Ignoring effects of concrete hydration, pavement reinforcement, and subgrade friction, etc., the initial cracking, as illustrated in Figure A-1(a), may be assumed to be equal to about 0.0003L, where L equals the length of pavement between contraction joints.

Responding to changes in ambient temperature, initial shrinkage cracks open wider at temperatures lower than normal, Figure A-1(b), and close at temperatures higher than normal, Figure A-1(c). With daily fluctuations in temperature, and with magnifications of these fluctuations caused by seasonal temperature extremes, this movement at contraction joints continues to cycle. However, because these joints are surface sealed, initial infiltration of debris commences at the open ends and open bottoms. This infiltration is facilitated by the movement of water, which penetrates the pavement and shoulder joints from above, and the groundwater, which seeps through the shoulders and migrates along the base below. As joint seals begin to fail because of a combination of age degradation, low temperature stiffening, traffic abrasion, neglect, etc., debris infiltration accelerates both from above and below.

Cyclic movement at contraction joints is restrained by compression of the debris and by the restrained expansion (compression) of the pavement, Δcp [compare Figure A-1(d) with A-1(c)].

Because stress is proportional to strain, (f = Eε), the stress induced by this restrained expansion (compression) can be estimated by assuming a value for the strain associated with the condition illustrated in Figure A-1(d). By assuming Δcp to be about equal to the original pavement shrinkage, Δs, equal to 0.0003L, the unit strain, ε, = 0.0003.

With the weight of concrete, W, equal to 145 pounds per cubic foot (pcf); the 28-day cylinder strength of concrete, f′, equal to 4000 pounds per square inch (psi); and the unit strain, ε, equal to 0.0003; the concrete compression stress, f, equals about 1000 psi.

![Figure A-1 Cyclic movement at contraction joints.](image-url)
\[ f_c = E_c \varepsilon \]
\[ E_c = W_{c1.5} (33) \sqrt{f_c} \]
\[ E_c = 145^{1.5} (33) / 4,000 \]
\[ E_c = 3.64 \times 10^6 \]
\[ f_c = (3.64 \times 10^6) (0.0003) \]
\[ f_c \approx 1,000 \text{ psi} \]

This is the stress associated with a pavement compression \( \Delta p \) about equal to the original shrinkage crack width, \( \Delta \). Obviously, other assumptions will yield other stresses, but any reasonable assumption will yield stresses of similar magnitudes.

Pressures of these magnitudes have been measured by Richards (56), who describes the application of rock mechanics techniques to the measurement of pavement pressures.

Essentially, the process consists of drilling a 1 1/2 in. diameter hole in pavement suspected of being compressed and bonding strain gauges to the concave surfaces of the hole. Then, at the same location, the pavement is over-cored with the hole located at the center of the core. After the core is removed, the changes in the strain gauges mounted within the core indicate the magnitude of the pressures that were compressing the core before its removal. Of 13 locations sampled in various Ohio counties, three cores indicated pressures in excess of 900 psi; two of these three cores were removed from pavement on bridge approaches. Other cores removed by Richards indicated a broad spectrum of stresses from about 70 psi up to and including 1064 psi.

The generation of such pressures may be visualized as suggested in Figure A-2. Illustrated is an idealized chart of the maximum compressive stress in a pavement, \( f_c \), as it is generated over time. Initially, the pressure is insignificant because the joints are relatively clean and the joint seals are intact and functioning. However, as time passes and joints begin to fill with debris, pressure increases at an increasing rate. As joints continue to fill, the somewhat compressible debris functions to minimize infiltration of additional material, slowing the rate of joint infiltration and pressure generation. Somewhere along this hypothesized pressure generation curve, the pavement fractures adjacent to a joint, relieving some of the pressure; or the pavement buckles, relieving all of the pressure at the location of the buckle. Illustrated in Figure A-2 is the pressure generation curve for one particular pavement. Because of the many factors that affect the performance of pavement joints, innumerable stress-time curves could be illustrated in this chart. This suggests that the fracturing could occur at an earlier or later time, depending on the number of factors that combine to affect the behavior of such joints.

From observations of various projects throughout Ohio, it appears that the major factors that contribute to joint infiltration and consequently to pavement pressure generation include the following:

1. Subgrade drainage
2. Sealant quality and durability
3. Temperature range and duration
4. Pavement moisture content
5. Deicing applications (including grit)
6. Traffic volume
7. Joint spacing
8. Pavement reinforcement
9. Sealant maintenance

It is apparent by examining this list of factors that care should be exercised in the original design and construction to ensure the best functioning of pavement contraction joints by careful and thoughtful attention to Items 1, 2, 7, and 8. However, after a project has been constructed, the most efficient functioning of the contraction joints can only be influenced by maintenance attention to Item 9. Where modification and repair can be justified, Items 1 and 2 can effectively be improved. But as Figure A-2 and the list of factors above would suggest, a pavement that has experienced a broad temperature range (requiring deicing applications) and that has good subgrade drainage, high-quality sealants, modest traffic volumes, and a good joint maintenance program, should be expected to survive 25 to 30 years before pressure generation reaches a point where pavement distress becomes evident. On the other hand, a similar pavement that has experienced the same temperature range and deicing applications, but with poor subgrade drainage, high traffic volumes, and minimal or no joint maintenance, should be expected to exhibit pavement distress and substantial bridge damage within 10 years.

**BRIDGE DISTRESS**

In Ohio, the nonintegral type of bridge was generally provided with a 3 in. wide open joint between the bridge deck and the abutment backwall and a 2 in. wide sliding joint in the end dam (Figure A-3). During construction, the superstructure and abutment parts of the end dam were bolted together while the concrete in the abutment backwall was placed, assuring that, as cast, the joint in the end dam would be 2 in. wide at concrete placement ambient temperatures. In other words, no adjustment was made in the width of the end dam joint for various ambient temperatures.

Figure A-4 shows a view of such an end dam and the top of an abutment backwall of a bridge on Route 77 in Summit County, Ohio, a nonintegral type of bridge constructed along the lines of the abutment illustrated in Figure A-3. In Figure A-4, the bridge deck is shown on the right, the approach slab in the upper left-hand corner, with the shoulder edge to the approach slab coinciding with the white striping. The fractured
concrete to the left of the structural steel end dam in the foreground is the top of the abutment backwall. Of significance in Figure A-4 is the opening in the end dam, which is nearly 2 in. wide in the foreground but entirely closed adjacent to the approach slab. This abutment, which is supported on short steel H piles driven to bedrock, is apparently sufficiently resistant to horizontal forces to prevent the abutment proper (footer and bridge seat) from being jammed against the bridge deck by the longitudinal thrust of the approach pavement. Consequently, the abutment near the curbs has remained intact and essentially unmoved by the thrust of the approach slab. This is evidenced by the 2 in. wide joint in the end dam visible in the foreground of Figure A-4. However, in the roadway area, the approach slab has sheared completely through the abutment backwall and closed the 2 in. wide joint. Incidentally, this bridge was less than 10 years old when this photograph was taken, illustrating that in some projects, pavement pressure generation commences early and pressure accumulates quickly.

Evidence from other similar bridges shows similar backwall fracturing. In addition, after the end dam has been closed (2 in. movement), the generating pavement pressure, supplemented by the pressure of an expanding bridge deck, is sufficient to thrust the backwall portion of the end dam under the superstructure end dam angle, completely lifting the superstructure off of the bearings. Then with the superstructure supported on the backwall, the reaction of the superstructure supplemented by the weight of the vehicular traffic continues the backwall fracturing until the backwall is completely fractured or until the pavement is released by cutting pressure relief joints in the bridge approach pavements.

**PAVEMENT GROWTH**

At the Stanley Avenue Bridge in Dayton, Ohio, the concrete approach pavements were cut transversely so that 3 ft wide bituminous-filled pressure relief joints could be installed. The need for these relief joints became necessary when the deck joints at the bridge abutments were found closed and evidence of substantial longitudinal pressure was evident.

Periodic observations of these relief joints were made over a five-year period. The cutting of pavements and release of pressure was followed by a gradual and progressive closure of the joints. At one of the relief joints, for instance, the movement of the approach pavement into the joint amounted to $7 \frac{1}{2}$ in. This movement occurred over a five-year period, for an average movement rate of $1 \frac{1}{2}$ in. per year.

The approaches to this bridge consist of a pair of two 12 ft wide pavements with a separately cast 4 ft wide raised median. When the pavements were constructed, the contraction joints were sawed to coincide with the vertical joints in the median curbs. The longitudinal movement of the pavement was manifested not only by the movement of the pavement into the relief joints, but also by a differential movement of the pavement joints with respect to the median curb joints. The joints closest to the bridge showed the greatest differential movement, $7 \frac{1}{2}$ in., whereas the joints further removed from the bridge showed progressively less and less movement, with the joints located approximately 1000 ft from the bridge showing no appreciable movement.

Consequently, based on the behavior of the pavement approaches to the Stanley Avenue Bridge and to similar pavements of many other bridges, it appears that up to 1000 feet or more of pavement can contribute to the movement of pavement at pressure relief joints. Because these pavement movements are both progressive and accumulative and because a substantial length of pavement contributes to this movement, this type of movement has come to be called "growth" to distinguish it from "expansion," the term usually used to refer to the minor component of cyclic movement.

The distinction between the terms "growth" and "expansion" is important. Many standard joint details have been designed...
to facilitate the expansion component of cyclic movement and have been named "expansion joints." These have mistakenly been selected to accommodate the "growth" of bridge approach pavements primarily because the individual making the selection associated the movement of pavements with the term "expansion." The use of name or label more indicative of the behavior being considered has been found to aid in a more appropriate selection of standard designs for particular applications.

The generation of pavement pressure or the generation of pavement growth appear to be two sides of the same coin or two major aspects of the same problem. The debris infiltration of contraction joints will result in pressure generation where the pavements are restrained (no relief joints or bridge joints) or growth generation where the pavements are not restrained (with relief joints or bridge joints, etc.). In many instances, growth will take place until all available space has been consumed. (All available space refers to space provided in pavement expansion joints, space available in pressure-relief joints by compression of the filler, and space provided in bridge joints to facilitate the cyclic movement of bridge decks.) Then, as the pavements are restrained from further growth, pressure generation commences along the pressure generation curve illustrated in Figure A-2. Because both pressure and growth generation appear to be directly related to the debris infiltration of contraction joints, the factors that have a significant effect on pressure generation have a similar effect on growth generation. "Ideally," the solution to this problem is simple. All that is needed is a pavement joint design that would completely seal the joint against the intrusion

FIGURE A-5 Pressure-relief joints.
of all foreign materials. Designs somewhat less than ideal would be suitable because a reasonable life-cycle could be attained. However, it should be clear that present technology is not sufficiently developed to provide a cost-effective solution to the significant problem of debris infiltration at contraction joints.

For bridges furnished with deck joints between superstructure and abutments, the joints can be protected from the adverse effects of pavement growth and pressure. To accomplish this, effective pressure relief joints, such as those shown in Figure A-5, can be installed in bridge approaches. In addition, because some of the pavement pressure can be transmitted through relief joints, abutments and their foundations can be designed to resist such pressures, unless the relief joint itself provides for such pressures to be resisted by the pavement subgrade.

For integral bridges, similar relief joints can be installed in bridge approaches. However, for longer integral bridges, especially those with curbs, barriers, etc., cycle control joints are also provided in bridge approaches not only to facilitate the constant cycling of the bridges, but also to protect the subgrade from the adverse effects of roadway drainage.
APPENDIX B

TYPICAL DESIGNS FOR INTEGRAL ABUTMENTS

CALIFORNIA

Bridge length "L"

Span

3'-0"

6'

Box girder

Constr. joint

Abut.

Underdrain and permeable material

Reinforcement not all shown

8" CSP

L-3'

L-3'

LIMITATIONS

Approach Slab: Yes No
Long. Move. Max. 1" 1/2"

TENNESSEE

Bridge length "L"

Span

Steel girder

Aggregate

Varies 2'-0" Min.

Clean well drained aggregate

LIMITATIONS

Skew Max. "L" 45° 400'

OHIO

BP 10 x 42 Piles

Constr. joint

Porous backfill

Rock channel protection

LIMITATIONS

Skew Max. "L" 30° 300'

PENNSYLVANIA

Prestressed concrete beams

Prestressed deck panels

NOTE:

Turnback wings each supported by a steel-H pile.

AASHTO No. 57 course aggregate

Geotextile material

LIMITATIONS

Not Established
II. DESIGN POLICY

1. Location of Expansion Joints:

(a) Expansion joints shall be located so that all structural members will satisfy AASHTO Article 1.2.22. Use the factors set forth in Article 1.2.15 and the procedures for calculating the points of no movement set forth in this memo to calculate thermal stresses in supporting members.

Do not use the factors in Article 1.2.15 to calculate joint movement ratings. Use the factors set forth in this memo.

(b) Expansion joints should be avoided in spans over public roadways, pedestrian crossings, railroads, or other locations that may be subject to public access.

(c) Consideration should be given to minimize the total number of expansion joints. This may require the designer to make additional trial calculations to find the most efficient location for expansion joints.

Points of no movement may be calculated by the approximate method. This method is shown in the example and is accurate enough for most cases. Abutment diaphragms on spread footings should be checked for sliding when the stiffness of the diaphragm is used to calculate the points of no movement. Use the following stiffness values to calculate the resistance of abutment diaphragms on piles.

### PROPERTIES/PILES

<table>
<thead>
<tr>
<th>Pile Class</th>
<th>I(Ft^4)</th>
<th>E(psi)</th>
<th>Equivalent Length</th>
<th>End Condition</th>
<th>Lateral Resistance K/in. Defl.</th>
</tr>
</thead>
<tbody>
<tr>
<td>45-1, 70T, 16&quot; CIDH, 10x57#, 12x53#</td>
<td>0.115</td>
<td>4.0x10^6</td>
<td>5.50'</td>
<td></td>
<td>100/Pile</td>
</tr>
<tr>
<td>45-2, 10x42#, 8x40#</td>
<td>0.080</td>
<td>4.0x10^6</td>
<td>5.35</td>
<td></td>
<td>75/Pile</td>
</tr>
</tbody>
</table>
2. **Movement Rating (M.R.) Calculations:**

The Joint Movements Calculations sheet is to be used to calculate the M.R. for all bridge expansion joints. This sheet is to be placed in the preliminary report for forwarding to Construction. The example illustrates the necessary information to be completed by Design.

The M.R. is equal to the total anticipated movement from widest to narrowest opening of a joint. This is equal to the total thermal movement plus any anticipated shortening. The factors used to calculate the thermal movement are those published in Research and Development's Annual Movement Study of Bridge Deck Expansion Joints and are commonly accepted values. The temperature range is the extreme ambient temperature range of the area which is given in the preliminary report. Research and Development has found that the structures temperature range is very close to the ambient range, therefore, no adjustment will be made to the ambient range. The factors used to calculate the anticipated shortening are about one-half the total expected creep and shrinkage. The reason is that approximately one-half of total anticipated shortening should be out of the structure at the time the joint groove widths are determined.

<table>
<thead>
<tr>
<th>TYPE OF STRUCTURE</th>
<th>Coefficient of Expansion: Movement/Unit Length/Degree Fahrenheit</th>
<th>Anticipated Shortening: inches/100 ft Contributory Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>.0000065</td>
<td>0.00</td>
</tr>
<tr>
<td>Concrete (conventional)</td>
<td>.0000053</td>
<td>0.06</td>
</tr>
<tr>
<td>Concrete (pretensioned)</td>
<td>.0000053</td>
<td>0.12</td>
</tr>
<tr>
<td>Concrete (post tensioned)</td>
<td>.0000053</td>
<td>0.50</td>
</tr>
</tbody>
</table>

Note that post tensioned concrete structures are expected to initially shorten about 0.50 in./100 ft due to stressing. The total long-term shortening is anticipated to be 1.00 in./100 ft. The difference between the long-term shortening (1.00 inch) and the initial shortening is equal to 0.50 in./100 ft. This is the value shown on the "Joint Movements Calculations" form as "Anticipated Shortening for Post Tensioned Concrete Structures."
Many variables affect the actual movement in an expansion joint. Some of these are:

1. The actual point of no movement may differ from the calculated point of no movement.
2. The actual shortening may differ from the estimated shortening;
3. The actual minimum and maximum structure temperatures may differ from the extremes listed in the preliminary report.

Therefore, the above factors, plus the fact that the Movement Rating test method does not provide a dependable safety factor explains the need for making conservative assumptions in design. For these reasons, the calculated M.R.'s shall be rounded up to the next half inch.

The M.R. of all joints must be shown on the plans. Examples of the correct notation on the plans are as follows:

Open Joint use: Open Joint (MR = 2")
Type A, use: Joint Seal (MR = 1/2")
Type B, use: Joint Seal (MR = 2")
Type Seal Assembly, use: Joint Seal Assembly (MR = 4")
Longitudinal Joint, use: Joint Seal (Type AL)

3. Open Joints:
   a. Open joints shall be considered first for all expansion joints. Designers are directed to use open joints whenever conditions are appropriate for their use.
   b. A special design will be required for an open joint at hinges for all structures except possibly steel structures. Designers are encouraged to participate in the design of a self-cleaning or drop through type hinge. Abutments on elastomeric bearing pads, rockers, or rollers can be designed to permit the use of open joints. Page 6-95 of the Bridge Design Details is an example.
Joint openings should be detailed for a width of $\frac{1}{2}''$ at the maximum temperature. Use the temperatures shown in the Preliminary Report to determine the movement rating. The Structure Representative will determine the exact width of the opening from the Joint Movements Calculations sheet.

The maximum movement rating for an open joint is $4\frac{1}{2}''$ inches. Do not use open joints on structures with sidewalks.

c. When open joints are used, drainage must be planned to avoid erosion or serious maintenance problems. In some cases water may be dropped directly through the joints while in others it may be necessary to collect it into a drainage system. An access gallery is required for seat abutments to provide for the removal of dirt and trash. See page 6-74 and page 6-95 of the Bridge Design Details Manual.

4. Sealed Joints

Sealed joints in this movement range should be noted on the plans and referenced to Standard Plan B6-21. No further information is necessary on the plans. The Standard Specifications specify that the type A pourable seal shall be used for a M.R. = $\frac{1}{2}''$ and that the type B seal shall be used for a M.R. of 1'', 1\frac{1}{2}'' and 2''. See attachment no. 4 for dimensions of the type B compression seals for the maximum groove width $W_1$ and the saw cut depth.

b. Joint Seal Assemblies (M.R. = 4'' maximum).
Sealed joints in this movement range should be noted on the plans and referenced to Standard Detail sheet XS-12-59, Joint Seal Assemblies, which is to be inserted in the plans. See page 20-77 of the Bridge Design Details Manual.

Complete the information required on the Joint Seal Assembly sheet for each joint concerning the location, M.R., skew, and "a" dimension by adding lines to the table.

Unless otherwise permitted by the plans or specifications, falsework supporting the hinge spans
should not be removed until the closure pour concrete adjacent to the joint seal assembly has attained the required strength.

The Special Provisions will list alternative joint seal assemblies which the Contractor may use in lieu of the joint seal assembly detailed on the Contract Plans. Attachment No. 5 shows details of alternative joint seal assemblies. If the Contractor selects an approved alternative joint seal, the Structures Representative may send two copies of the initially submitted working drawings to Structures Design for an informal review by the Joint Seal Committee and by Structures Design.

Requests for substitutions should be referred to the chairman of the Joint Seal Committee. The Specifications Section will revise the special provisions when new or improved products are approved.

Where water tightness is important to meet commitments such as leased air space under the structure, Design should notify Specifications to delete trade name alternatives that do not meet that criteria. Notification should be given to specifications when the plans and quantities (P&Q) are submitted.

c. Sealed Joints (M.R. greater than 4").
   For joints having a M.R. greater than 4" the designer should consider finger joints and possibly relocate expansion joints to permit finger joints at the abutments. If this is not feasible, the designer should check with the chairman of the Joint Seal Committee during the early design stage for selection of a watertight modular trade name system.

5. Longitudinal Joints

The M.R. should be calculated much the same as transverse joints and shown on the plans as Joint Seal (MR = ________) or Open Joint (MR = ________).

Longitudinal joints are to be avoided in areas that may be subject to vehicle traffic, due to the hazards of tires tracking the joint. When they cannot be avoided and there is no appreciable movement, specify "joint seal (Type AL)" and reference Standard Plan B6-21.
6. **Expansion Joint Fillers** *(Sealed Joints)*

M.R. \( \leq 2" \): The expansion joint width is sized by the filler width "a" as shown on Standard Plan B6-21. Do not dimension on the plans.

M.R. \( \geq 2" \): Expansion joint filler thickness for expanded polystyrene shall be calculated as follows:

- **Summer**  
  \[ "a" = \frac{1}{2} (\text{M.R. less anticipated shortening}) \]

- **Spring and Fall**  
  \[ "a" = \frac{3}{4} (\text{M.R. less anticipated shortening}) \]

- **Winter**  
  \[ "a" = \text{M.R. less anticipated shortening.} \]

Where "a" = Thickness of expanded polystyrene. The anticipated shortening should be calculated by the factors shown on the Joint Movements Calculations sheet.

The thickness of the expanded polystyrene must be shown on the plans for larger than 2-inch movement rating joints (no reference to Standard Plan B6-21).

7. **Waterstop**

The waterstop, in addition to resisting leaking, provides a membrane that stops silt, sand, concrete chippings, etc. from falling into a joint below an easily accessible level. Debris is especially a problem during construction before a joint is sealed and even after construction if there is a joint seal failure. Waterstop should be specified at all sealed joints at hinges and seat type abutments where Standard Plan B6-21 is used.

Waterstop, when required, should be shown on the plans and referenced to Standard Plan BO-3.

Do not use waterstop with open joints, longitudinal expansion joints, paving notch joints for end diaphragm abutments with approach slabs, and joints with a M.R. greater than 2".

8. **Quantity Estimate**:

Estimate the linear feet of each M.R. joint seal called for on the plans. Waterstop is a separate item and should be estimated in linear feet.
9. Other Considerations

It is the designer's responsibility to check the movement capacity of all standard details. This includes items such as hinge details, rollers, rockers, elastomeric pads, abutment diaphragms on piles or spread footings, earthquake restrainers and any other details which may be affected by too much movement. In all cases the movement capacity of these items should be greater than the calculated movement, or as recommended by other Memos to Designers.

Designers are encouraged to coordinate with the Joint Seal and other related committees to make suggestions for improving procedures and details involving joint seals and expansion joints.

Following are some of the items which are presently being considered for improving expansion joints:

1. Eliminating all intermediate joints in curved structures with a small radius and high columns.

2. Designing open, drop through hinges for concrete structures.

3. Evaluating large M.R. joint seals such as those available on the open market and those designed by our personnel.

4. Developing an effective debris barrier used in conjunction with a joint seal that will replace our present waterstop.

George A. Hood

Guy D. Mancarti

Attachment
# Joint Movements Calculations

## Design

<table>
<thead>
<tr>
<th>Design</th>
<th>Construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Designer: ____________________ Date: ____________</td>
<td>4. R.E. or Sr: Complete &amp; Return to structure construction with JT Seal report.</td>
</tr>
<tr>
<td>2. Checked: ____________________ Date: ____________</td>
<td>5. Joint Seal Committee Chairman - Review</td>
</tr>
<tr>
<td>3. Project Designer: Send to R.E. or SR with preliminary report</td>
<td>6. Maintenance file</td>
</tr>
</tbody>
</table>

### Expense Authorization

<table>
<thead>
<tr>
<th>Dist.</th>
<th>County</th>
<th>Route</th>
<th>PM</th>
<th>Bridge Name &amp; No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>03</td>
<td>Sac</td>
<td>5</td>
<td>0.2'2.4</td>
<td>Dry Creek O.C. 29-000</td>
</tr>
</tbody>
</table>

### Joint Structure

<table>
<thead>
<tr>
<th>Type</th>
<th>Abutment</th>
<th>Expansion Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced Concrete, Box &amp; CIPP/S</td>
<td>A1 - 70T Piles</td>
<td>2' Elasto Pads, etc.</td>
</tr>
<tr>
<td></td>
<td>A2 - Spd. Ftg.</td>
<td>Bent 5 = 1.5 Steel Hangers</td>
</tr>
</tbody>
</table>

### Temperature Extremes

- Max: **110°F**
- Min: **23°F**

### Type of Structure

<table>
<thead>
<tr>
<th>Steel</th>
<th>Concrete (Conventional)</th>
<th>Concrete (Pretensioned)</th>
<th>Concrete (Post tensioned)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>87°F</strong></td>
<td><strong>0.000053x1200</strong></td>
<td><strong>0.000053x1200</strong></td>
<td><strong>0.000053x1200</strong></td>
</tr>
</tbody>
</table>

### Thermal Movement

<table>
<thead>
<tr>
<th>Type</th>
<th>Movement</th>
<th>Shortening</th>
<th>Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td><strong>0.55</strong></td>
<td><strong>0.12</strong></td>
<td><strong>1.05</strong></td>
</tr>
</tbody>
</table>

### Temperature Calculations

<table>
<thead>
<tr>
<th>Type</th>
<th>Movement</th>
<th>Shortening</th>
<th>Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete (Conventional)</td>
<td><strong>0.55</strong></td>
<td><strong>0.12</strong></td>
<td><strong>1.05</strong></td>
</tr>
<tr>
<td>Concrete (Pretensioned)</td>
<td><strong>0.55</strong></td>
<td><strong>0.12</strong></td>
<td><strong>1.05</strong></td>
</tr>
</tbody>
</table>

### Seal Width Limits

<table>
<thead>
<tr>
<th>Catalog Number</th>
<th>W1</th>
<th>W2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>A</strong></td>
<td><strong>½</strong></td>
<td><strong>1.05</strong></td>
</tr>
<tr>
<td><strong>B</strong></td>
<td><strong>1.01</strong></td>
<td><strong>2.31</strong></td>
</tr>
<tr>
<td><strong>C</strong></td>
<td><strong>3.32</strong></td>
<td><strong>6.11</strong></td>
</tr>
</tbody>
</table>

### Seal Width Installation

**SHOW LINE DRAWING OF STRUCTURE ON REVERSE SIDE, SHOW POINTS OF NO MOVEMENT AND CONTRIBUTORY LENGTHS. RETAIN COPY FOR DESIGN CALCULATIONS FILE.**

**TYPE B INFORMATION FROM TRANSLAB REPORTS**

**GROOVE WIDTH ADJUSTMENT BASED ON Δ = (MAX TEMP EXTREME) - (SUPERSTRUCTURE TEMPERATURE).**

**MEASURE SUPERSTRUCTURE TEMPERATURE BY PLACING BULB OF CONCRETE THERMOMETER 6 ± INTO EXPANSION JOINT.**
CALCULATION OF POINTS OF NO MOVEMENT

<table>
<thead>
<tr>
<th>L (Ft)</th>
<th>5.50</th>
<th>35.0</th>
<th>40.0</th>
<th>Sum</th>
<th>40.0</th>
<th>40.0</th>
<th>Sum</th>
<th>40.0</th>
<th>40.0</th>
<th>Sum</th>
</tr>
</thead>
<tbody>
<tr>
<td>P (kips) @ 1' side sway</td>
<td>1200</td>
<td>610</td>
<td>615</td>
<td>2,233</td>
<td>0</td>
<td>415</td>
<td>415</td>
<td>830</td>
<td>359</td>
<td>slide + 600 = 959</td>
</tr>
<tr>
<td>D (dist. from 1st member of frame)</td>
<td>0</td>
<td>90</td>
<td>210</td>
<td>0</td>
<td>160</td>
<td>0</td>
<td>90</td>
<td>0</td>
<td>540</td>
<td>540</td>
</tr>
<tr>
<td>P x D / 100</td>
<td>0</td>
<td>565</td>
<td>870</td>
<td>1,426</td>
<td>0</td>
<td>664</td>
<td>664</td>
<td>0</td>
<td>664</td>
<td>664</td>
</tr>
<tr>
<td>X = ( \frac{\Sigma (P \times D)}{100} )</td>
<td>1426</td>
<td>2233</td>
<td>64'</td>
<td>664</td>
<td>830</td>
<td>(100) = 80'</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P (Col) = 12 EL ( \frac{A}{L^2} )</td>
<td>D.W. Abut 5 = 600k (assume linear up to 1' defl.)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>@ 1' defl. = ( \frac{432 L^3}{10^3} )</td>
<td>I (abut) = ( \frac{78}{12} \times (2.5)^3 )</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>X = Point of No Movement</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes:
- Width Str. = 76'
- Dia. Col. = 5'0''
- K / Pile @ 1' defl. = 100 kips
- X = Point of No Movement

Assumptions:
1. Super str. inf. rigid
2. Col's fixed top & bottom
3. Abut. fig. will slide @ a force equal to D.W.
4. E (piles) = 4 x 10^6 psi
   E (columns) = 3 x 10^6 psi
THE TRANSPORTATION RESEARCH BOARD is a unit of the National Research Council, which serves the National Academy of Sciences and the National Academy of Engineering. It evolved in 1974 from the Highway Research Board which was established in 1920. 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. 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 National Academy of Sciences is a private, nonprofit, self-perpetuating society of distinguished scholars engaged in scientific and engineering research, dedicated to the furtherance of science and technology and to their use for the general welfare. Upon the authority of the charter granted to it by the Congress in 1863, the Academy has a mandate that requires it to advise the federal government on scientific and technical matters. Dr. Frank Press is president of the National Academy of Sciences.

The National Academy of Engineering was established in 1964, under the charter of the National Academy of Sciences, as a parallel organization of outstanding engineers. It is autonomous in its administration and in the selection of its members, sharing with the National Academy of Sciences the responsibility for advising the federal government. The National Academy of Engineering also sponsors engineering programs aimed at meeting national needs, encourages education and research, and recognizes the superior achievements of engineers. Dr. Robert M. White is president of the National Academy of Engineering.

The Institute of Medicine was established in 1970 by the National Academy of Sciences to secure the services of eminent members of appropriate professions in the examination of policy matters pertaining to the health of the public. The Institute acts under the responsibility given to the National Academy of Sciences by its congressional charter to be an adviser to the federal government and, upon its own initiative, to identify issues of medical care, research, and education. Dr. Samuel O. Thier is president of the Institute of Medicine.

The National Research Council was organized by the National Academy of Sciences in 1916 to associate the broad community of science and technology with the Academy's purpose of furthering knowledge and advising the federal government. Functioning in accordance with general policies determined by the Academy, the Council has become the principal operating agency of both the National Academy of Sciences and the National Academy of Engineering in providing services to the government, the public, and the scientific and engineering communities. The Council is administered jointly by both Academies and the Institute of Medicine. Dr. Frank Press and Dr. Robert M. White are chairman and vice chairman, respectively, of the National Research Council.