NATIONAL COOPERATIVE
HIGHWAY RESEARCH PROGRAM REPORT **243**

REHABILITATION AND REPLACEMENT OF BRIDGES ON SECONDARY HIGHWAYS AND LOCAL ROADS

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL

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REHABILITATION AND REPLACEMENT OF BRIDGES ON SECONDARY HIGHWAYS AND LOCAL ROADS

University of Virginia Civil Engineering Department
Virginia Highway and Transportation Research Council
Charlottesville, Virginia
AND
Virginia Department of Highways and Transportation
Richmond, Virginia

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AREAS OF INTEREST:
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TRANSPORTATION RESEARCH BOARD
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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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

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.

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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.

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FOREWORD

By Staff
Transportation Research Board

This report contains the findings of an evaluation of repair and rehabilitation procedures and replacement systems used to correct common bridge deficiencies. These findings are immediately applicable and will be of interest to engineers concerned with the design, construction, and maintenance of bridges, in particular those on secondary highways and local roads.

Many bridges on secondary highways and local roads are in need of replacement or major structural repair. Although most are not frequently traversed by heavily loaded vehicles, these structures are vital to the efficient movement of agricultural and other commodities and provide an important transportation link for rural America's population with centralized educational centers. The National bridge inventory shows that, of the more than 500,000 bridges in the United States, 98,000 are structurally weak or unsound and another 102,000 are functionally obsolete because of inadequate alignment, widths, clearances, or load carrying capacities. Under the severe fiscal constraints that currently exist at the local level, most of these bridges cannot be replaced in the foreseeable future. Until recently, considerable effort had been devoted to the analysis and design of new structures, but little attention was given to problems associated with rehabilitation of older structures on the secondary and local road systems. Therefore, local agencies responsible for inspection, maintenance, and repair are required to make decisions without benefit of supporting information. Under these conditions, an urgent need exists for research that will provide tools for engineers to reach and carry out cost-effective decisions.

This report contains the findings of Phase II of NCHRP Project 12-20. The findings of Phase I were published in March 1980 in *NCHRP Report 222,* "Bridges on Secondary Highways and Local Roads—Rehabilitation and Replacement," which contains information that local highway agencies can apply immediately to the repair, improvement, or replacement of deficient bridges on secondary and local road systems. Project 12-20 had four major objectives: (1) to identify the common deficiencies found on bridges on secondary highways and local roads throughout the United States, (2) to evaluate feasible corrective procedures that have been successfully employed for these deficiences, (3) to evaluate economical replacement systems for bridge structures for which repair or rehabilitation is not feasible, and (4) to develop a simple procedure to assist engineers in making decisions involving repair or replacement.

The major portion of *NCHRP Report 222* consisted of a manual of recommended practice—comprising 34 procedures for repair, rehabilitation, and retrofit of bridges and 27 systems that are available for use in replacing bridge components or complete structures. The manual is intended to be used by engineers responsible for bridges on secondary highways and local roads. In preparing the manual it was recognized that many of these engineers are not bridge specialists; therefore, the goal was to provide enough information to alert the engineer of his options in dealing with certain bridge deficiencies and to direct him to the proper sources for more detailed information required for a final design.

Phase II of Project 12-20 was initiated in June 1980 with the purpose of expanding the manual to include procedures directed at the problems of fatigue cracking of steel bridge members, scour, bridge deck deterioration, seismic damage, and damage due to accidental impacts.

This report contains useful information both on repair and rehabilitation procedures that can be applied to bridges with such problems and on replacement systems that are also available for immediate application.

CONTENTS

ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 12-20 jointly by the Civil Engineering Department, University of Virginia; the Virginia Department of Highways and Transportation; and the Virginia Highway and Transportation Research Council. Henry L. Kinnier and Wallace T. McKeel, Jr., served as co-principal investigators. Kinnier, a professor of civil engineering for the University of Virginia and a faculty research scientist for the Research Council, prepared the section on Selected Retrofit Procedures Designed to Prevent Seismic Damage. McKeel, a research scientist for the Research Council, was in charge of the preparation of the five sections: (1) Evaluation, Repair and Replacement of Concrete Bridge Decks, (2) Repair of Accidental Damage, (3) Repair of Fatigue Damage in Steel Members, (4) New York Thruway Deck Replacement Systems, and (5) Method Used for Replacement of the Deck on the George Washington Bridge. Marvin H. Hilton, Research Scientist, with the Research Council, prepared the section on Scour at Bridge Sites; and M. M. Sprinkel, Research Scientist, also of the Research Council, prepared the sections on Replacement Systems. J. E. Andrews, Bridge Design Engineer Supervisor, and L. L. Misenheimer, District Bridge Engineer, both of the Virginia Department of Highways and Transportation, reviewed the drafts of the several sections. M. D. Leinbach and J. P. Gomez III, graduate civil engineering students of the University of Virginia, carried out literature searches for the report and prepared the drawings.

REHABILITATION AND REPLACEMENT OF BRIDGES ON SECONDARY HIGHWAYS AND LOCAL ROADS

SUMMARY

This report presents the results of Phase II of NCHRP Project 12-20, "Bridges on Secondary Highways and Local Roads—Rehabilitation and Replacement" (results of Phase I were published in *NCHRP Report 222,* May 1980), and addresses the following topics concerned with bridge repair, maintenance, and replacement that were not included in the first study.

1. Evaluation, repair, and replacement of concrete bridge decks.
2. Repair of accidental damage.
3. Repair of fatigue damage in steel members.
4. New York Thruway Deck Replacement System.
5. Method used for replacement of the deck on the George Washington Bridge.
6. Scour at bridge sites.
7. Selected retrofit procedures designed to prevent seismic damage.
8. Replacement systems—innovative concepts for the use of prefabricated bridges and bridge components; short-span segmental construction; field-connected beams; single and multiple culverts of aluminum, concrete, and steel; modular construction.

As was the case in the first phase of this project, the purpose of the second phase is to review repair and rehabilitation procedures, newly developed replacement systems, and other successful bridge operations that have been used and may be of interest and use to engineers responsible for the maintenance of bridges on secondary highways and local roads. The intention is not to develop new methods or extend the state of the art, but rather to assemble from several sources information that will be useful to engineers who are not necessarily specialized in all the relevant areas of structural engineering.

Similarly to *NCHRP Report 222,* the organization of this research differs from that of the usual NCHRP report. The material provided in subsequent chapters pertains to each of the eight topics previously named in this summary. A number of references are included at the conclusion of each chapter in which additional detailed information is available on areas related to the subject discussed. Replacement systems are prepared in the same format used in the Phase I *NCHRP Report 222.*

CHAPTER ONE

INTRODUCTION AND BACKGROUND

This report presents the results of the second phase of a two-phase effort. The results of Phase I were published in May 1980 as *NCHRP Report 222*. That phase of this study addressed four major objectives: (1) to identify the common deficiencies found on bridges on secondary highways and local roads throughout the United States, (2) to evaluate feasible corrective procedures that have been successfully employed for these deficiencies, (3) to evaluate economical replacement systems for bridge structures for which repair or rehabilitation is not feasible, and (4) to develop a simple procedure to assist engineers in making decisions involving repair or replacement. In Phase I, 34 repair, rehabilitation, and retrofitting procedures were found to correct a high percentage of the common bridge deficiencies, and 27 highway bridge replacement systems that have been successfully used in the United States were identified.

A number of important topics concerned with bridge maintenance, repair, and rehabilitation, however, were not addressed in the first phase. Further, a few, newly developed replacement systems were not included. A number of these were deliberate omissions and resulted from the fact that major studies in the omitted areas were being conducted concurrently with Phase I. It was considered advantageous to defer evaluation and comments on several areas until certain relevant studies were completed.

Phase II of NCHRP Project 12-20 was anticipated at the conclusion of the first phase to provide information on certain of the topics not addressed in Phase I. Two additional topics are included in this second phase report that are deemed to be of current interest to bridge engineers which came to the attention of the researchers and the projct panel during the course of the first phase; namely, the New York Thruway Deck Replacement System and the method used for replacement of the deck on the George Washington Bridge. Consequently, a total of eight topics considered to be of interest to engineers responsible for bridges on secondary highways are addressed in this report.

The intent of the research was not to develop new methods or extend the state of the art, but rather to assemble information from several sources in a format that would be useful to engineers responsible for bridge maintenance, rehabilitation, and replacement.

Only U.S. customary units are used in this report; however, to obtain values in S.I. units, the following six basic conversions are needed:

1 foot (ft) = 0.3048 meter (m)
1 inch (in.) = 25.4 millimeters (mm)
1 ounce (oz) = 0.2780 Newtons (N)
1 pound (lb) = 4.448 Newtons (N)
1 pound per square inch (psi) = 6895 Pascals (Pa)
1 mil = 0.0254 millimeters (mm)

CHAPTER TWO

EVALUATION, REPAIR, AND REPLACEMENT OF CONCRETE BRIDGE DECKS

The deterioration of concrete bridge decks has long been recognized as a major problem on all classes of highways. Assessment of the extent of the problem and possible preventive measures have been developed and reported in numerous studies, and no attempt has been made to duplicate that information here. Rather, the information has been summarized for use by engineers in the field. Selected references are provided at the end of this chapter. A complete bibiliography is contained in Ref. *(1)*.

An earlier NCHRP synthesis, published in 1970 and now out of print *(2)*, defined the various types of distress and provided information on protective measures. Subsequently, much research sponsored by FHWA, NCHRP, and various state agencies has been devoted to the deterioration of bridge decks, with most of the effort being concentrated on the problems associated with spalling, which is the separation of the surface concrete, often at the level of the top mat of reinforcement. Studies by FHWA have defined the mechanism through which reinforcing steel corrodes in the presence of chloride ions and have expanded on several earlier studies by California in defining areas of active corrosion. These studies have also developed analytical procedures for determining the amount of chloride in concrete *(3,4)*. Other studies, funded under NCHRP, have concentrated on the use of waterproof membranes and cathodic protection as protective systems and the influence of repair techniques on the subsequent corrosion of the reinforcing steel *(5,6)*. A great number of studies conducted by transportation agencies throughout the United States and in Canada are in the comprehensive list of references published in Ref. *(1)*.

Problems with concrete bridge decks include cracking, wear and polishing, scaling, and delamination—the last mentioned being by far the most serious in limiting the service life of the deck.

Cracking of bridge decks can be caused by tensile forces in the structure in excess of the capacity of the concrete to sustain or by shrinkage during construction. Structural cracking may be a characteristic of the design, as in continuous beams, but its occurrence can be minimized by controlling the strength of the concrete and by care during consolidation of the concrete around the reinforcement. Shrinkage cracking can be lessened by proper quality control and construction procedures, particularly the timely initiation of curing. Cracking does play a role in the onset of corrosion of the reinforcement, but other factors, such as the depth and quality of the surface concrete, are more important. Cracking due to corrosion of the underlying steel is the first sign of serious distress.

Wear and polishing of a bridge deck are important factors in its service life because of the accompanying loss of skid resistance. The loss of skid resistance due to the wearing of even relatively rough surface textures has caused some state agencies to specify surfaces with transverse grooves 1/8 in. in width by 1/8 to 3/16 in. in depth. The grooves allow for the escape of water from beneath the tires, and thus prevent hydroplaning. The loss of skid resistance due to polishing can be prevented by specifying that a nonpolishing fine aggregate, such as a silica sand, be used in the concrete surface.

Scaling has been defined as local flaking or peeling away of the near-surface portion of hardened concrete or mortar caused by frost action and aggravated by the presence of deicing salts *(1)*. The most effective defense against scaling is the use of properly air-entrained concrete. Air-entrained concrete is generally produced through the use of the appropriate amount of an air-entraining admixture to yield the desired air content. Recommended air contents for bridge deck concrete are given, as follows, in an excerpt from the table "Recommended Air Contents for Frost-Resistant Concrete" prepared by A.C.I. Committee 201 (Table 1.4.3, Ref. *(7)*).

NOMINAL MAXIMUM AGGREGATE SIZE (IN.)	AVERAGE AIR CONTENT, PERCENT FOR SEVERE EXPOSURE (BRIDGE DECKS)*
3/8	7½
½	7
¾	6
1½	5½

* A reasonable tolerance for air content in field construction is ± 1½%.

Spalling is the separation and removal of the surface concrete caused by the corrosion of embedded reinforcing steel *(1)*. Corrosion of the steel occurs when sufficient chloride ions from deicing salts permeate the concrete and overcome the natural passivity of the steel. The corrosion product occupied a greater volume than the original reinforcing bar, and the resulting pressure causes separation of the concrete at the level of the steel.

The complete discussion of the corrosion reaction provided in the *NCHRP Synthesis of Highway Practice 57* is summarized as follows. Corrosion, an electrochemical reaction, requires an anode, a cathode, an electrolyte, and the presence of oxygen. Moisture in the concrete serves as the electrolyte and oxygen is generally available. Points on the reinforcing steel serve as the anode, where ions are discharged and corrosion occurs, and the cathode, where ions are received. Anodic and cathodic areas form at points on a reinforcing bar where there are differences in surface conditions or in the environment around the bar. Various aspects of the corrosion process enter into the evaluation, repair, and reconstruction procedures for spalled bridge decks.

EVALUATION OF SPALLED BRIDGE DECKS

Wearing, scaling, and cracking of bridge decks can generally be evaluated visually, but the assessment of the condition of spalled bridge decks requires special techniques.

Spalls in a bridge deck, those areas where the concrete overlying the reinforcement has been removed, are, of course, immediately apparent. However, areas of delamination where the overlying concrete has not been dislodged are often present as well. A complete survey of the deck should define those areas in which the steel is actively corroding but separation has not begun or is so minimal that it is not apparent. Finally, the chloride ion concentration in the concrete, an indicator of the ability to support active corrosion should be determined. In summary, each of the following three evaluations, fully described in Ref. *(1)*, are essential to a complete deck survey:

1. Visual inspection and sounding of the bridge deck.
2. Measurement of half-cell potentials at points on the deck surface to define areas of active corrosion.
3. Sampling of the concrete for subsequent chemical analysis to determine the chloride ion concentration.

All three steps are necessary because the affected areas do not necessarily coincide. The repair of only spalls and separated areas is often only a stop-gap solution. Additional patches are needed in a short time as areas of active corrosion progress and new corrosion cells develop between steel in the patch and that in the adjacent chloride-contaminated concrete. Long-term repair can be assured only when all concrete having sufficient chloride concentration to sustain corrosion is removed or its effects nullified through the application of a cathodic protection system.

REPAIR OF BRIDGE DECKS

Wear and Polishing

Low skid resistance because of traffic wear can require corrective action to eliminate hazardous conditions. In some instances, where the cover over the reinforcing steel is not critically shallow and the concrete is sound and uncontaminated, the skid resistance is improved by sawing transverse grooves, described earlier, into the deck surface. Otherwise, an overlay of bituminous concrete, polymer concrete, or plain or modified portland cement concrete is generally required. The various types of overlays will be discussed more fully under repairs of spalled concrete. It is important to note, however, that a bituminous concrete overlay should not be used alone on the deck. Properly designed bituminous concrete does not prevent the passage of chloride into the deck, so a waterproof membrane must be first applied to the deck surface. Used alone, a bituminous concrete overlay can conceal the progressive deterioration of the concrete until the full depth of the slab is affected.

The choice of type of overlay must be dictated by the capacity of the bridge to carry additional dead load. A bituminous concrete overlay is usually placed with a 1½ to 2-in. minimum thickness, while a polymer concrete overlay, which is composed only of polymer and fine aggregate, can have a thickness of only ⅜ in.

Cracking

Cracking of bridge decks is not always repaired. Should repair be warranted, load-induced cracks, which open and close with the passage of traffic, must be filled with a flexible joint-sealing compound, generally after routing. Filling with a rigid adhesive, such as an epoxy resin system, often results in the forming of a new crack at or adjacent to the bond line. Extensive cracking may also be sealed by applying a polymer concrete overlay or a bituminous concrete overlay with a waterproof membrane capable of bridging the cracks. The cracks will reflect through a conventional portland cement concrete overlay.

Nonstructural cracking, related to delaminations in the deck caused by corrosion of the reinforcement, can be repaired with epoxy resins and other adhesives.

Scaling

On relatively new bridge decks light-to-medium scaling, limited to the loss of surface mortar with minor exposure of the coarse aggregate, may be indicative of insufficient entrained air. A common protective system is two applications of a 50/50 mixture of boiled linseed oil and either mineral spirits or kerosene. For estimating materials, rates of 40 yd²/gal for the first coat and 65 yd²/gal for the second coat are assumed *(7)*. The actual rate should be determined on a test section of the deck. Overlays may be used instead and are generally required for more severe scaling. A complete survey of the deck, including the tests described earlier, is advisable prior to the installation of an overlay.

Spalls

An evaluation of the condition of the bridge deck through the application of the testing techniques described earlier is an essential first step in repairing spalls. Knowledge of the chloride content of the concrete, the extent of delaminated areas in it, and the presence of active corrosion of the reinforcement is required in determining the extent of the repair and choosing the appropriate procedure. A discussion of the various factors in the planning process is provided in Ref. *(1)*, which also more fully describes the options available to the engineer.

The amount of concrete to be removed from the deck during repairs is indicated by the survey, although overruns are common. The Minnesota Department of Transportation, as one example, uses the guidelines below to relate concrete removal to the percentage of unsound areas in the deck surface.

CATEGORY	DECK CONDITION	PROCEDURE
I.	0–5% Unsound (Slight deterioration)	Scarify, spot removal, and 2-in. low slump concrete overlay or 1½-in. latex modified concrete overlay.
II.	5–29% Unsound (Moderate)	Scarify, spot removal and 2-in. low slump concrete overlay of 1½ in. Latex modified concrete overlay.
III.	20–40% Unsound (Severe) (20–60% Unsound on highways with less than 10,000 ADT and with the bottom of the slab sound.)	100% removal of surface down to reinforcing bars and minimal spot removal below reinforcing bars. Overlay with 3 in. of low slump concrete.
IV.	40+% Unsound (Critical) 60+% Unsound on highways with less than 10,000 ADT	Schedule new deck after usable life of inplace deck is expended.

As mentioned earlier, unless cathodic protection is employed, a permanent repair can be assured only if all concrete in areas having chloride contents sufficient to sustain corrosion is removed to a depth of ¼ in. plus the maximum size of the aggregate beneath the top mat of reinforcing steel *(1)*. The actual corrosion threshold can be as low as 1.3 lb of Cl^- per cu yd of a typical deck concrete with a cement factor of 7 sacks per cu yd, but a value of 2 lb Cl^- per cu yd is commonly accepted as the level beyond which removal of the concrete is warranted *(3)*. Unless the contaminated concrete is removed, differences in the surface conditions and environment around points on the reinforcing bar may cause the formation of anodic and cathodic areas and a resumption of the corrosion process. It should be noted, however, that removal of concrete below the reinforcing steel is extremely costly, and some agencies have found complete removal and replacement of the deck to be more economical. Patching of the deck followed by the installation of a protective overlay or membrane is a less costly and often used alternative.

Patching

Consideration of the corrosion process indicates that patches cannot be considered permanent repairs, and field experience verifies this conclusion. Newly delaminated areas are often found adjacent to areas patched months before. Nevertheless, patching can be an appropriate holding action until more extensive restoration is performed, and it can provide substantial service with the subsequent installation of a waterproof membrane. (Davis, R. E., J. B. Poppe, J. H. Gates, and R. C. Cassano, "CALTRANS Research—An Overview," unpublished paper prepared for the Federally Coordinated Projects Conference, Williamsburg, Virginia, December 3-4, 1979, California Department of Transportation, Sacramento, California.)

The area to be patched is defined on the deck by sounding, which sometimes is supplemented by taking potential readings. The boundaries of the area are sawed to a depth of at least 1 in., and the concrete is removed. Any exposed reinforcing steel is cleaned, a bonding agent is applied if required, and the repair material is placed and cured.

A wide variety of materials has been used for patching bridge decks. While conventional portland cement concrete is often used, most of the other materials have been developed to provide rapid strength development and allow early opening of the deck to traffic. Among these are concrete containing accelerators, fast-setting cements, polymer compounds, and polymer concrete composed of polymer and aggregate. Most of the polymer concrete are proprietary formulations, and it is essential that the manufacturers' requirements for mixing, placing, and curing be rigidly followed.

Bonding compounds vary with the repair materials. Usually a grout is brushed into the clean, sound surface of the underlying concrete, although some of the polymer-modified concretes develop sufficient adherence that a bonding agent is not required. A coating of the epoxy or other polymer is generally used to prime the surface of the underlying concrete before a polymer concrete is placed.

Epoxy Injection

An alternative to removing and patching a delaminated area in a bridge deck is the injection of an epoxy resin adhesive into the crack to bond the separated section to the deck *(1,8,9)*. As is the case with conventional patching, the repair cannot be considered permanent, but injection can be a cost-effective technique for extending the service life of a deck.

Holes are drilled through the delaminated area and between the reinforcing bars to a depth below the delamination, usually 2 to 3 in. An appropriate epoxy system capable of bonding to wet surfaces is injected into the entry hole under pressure. A pumping system in which the two components of the epoxy are mixed at the injection nozzle is usually used. Polymers other than epoxies have been used with only mixed success *(1)*.

Injection repairs require more expertise than conventional patching, particularly in selecting the epoxy resin. Although experience with the application of injection to deck repairs is limited, advice is generally available from epoxy resin formulators, and reports listed in the bibliographies of Refs. *(1, 9,* and *12)* provide additional information.

Cathodic Protection

As mentioned earlier, corrosion requires an anode, a point on the reinforcing steel where ions are released with resulting corrosion, and a cathode, another point on the steel where ions are received. Cathodic protection is the application of direct current in such a manner that the reinforcing steel becomes cathodic to anodes located on the deck. In most of the systems installed to date a direct current from an external power source is impressed on the circuit between the anodes, which are placed in a conductive bituminous layer on the deck surface, and the top reinforcing mat. The anodes are commonly high-silicon cast iron or graphite, and the conductive layer, a mixture of coke aggregate, asphalt, stone and sand, is needed to minimize the number of anodes required to protect the deck. A DC rectifier operating on AC line voltage and a control panel to regulate the voltage and current of each anode are located nearby, generally beneath the bridge. Periodic monitoring of the instruments in the control box is required to ensure that the polarized potential of the steel is maintained within limits established in an earlier NCHRP study *(6)*.

Disadvantages of the cathodic protection system include the need for periodic adjustment, a continuing though minimal power requirement, possible disbonding of the overlay, and the need for expertise in design and construction. At least on present systems, there is a further requirement that the structure be capable of carrying a bituminous overlay having a minimum thickness of 3 in. The coke-aggregate conductive layer, usually 1½ in. in thickness, does not possess the requirements of a wearing course, so an additional 1½-in. bituminous concrete wearing course has been required. Current research is investigating potential conductive materials with better wearing qualities, new anode configurations, and the use of galvanic cathodic protection, as opposed to impressed current systems, so it is likely that some of the disadvantages will be lessened in the future.

The significant advantage of cathodic protection lies in its ability to halt the progress of corrosion without the removal of chloride-contaminated concrete. Consultants are also available to assist in installing the system if desired. Decks under consideration for cathodic protection systems should not be repaired by epoxy injection, because this insulates the steel. Polymer bonding agents and polymer concrete patches should not be used in repairing the deck before the cathodic protection system is installed.

Waterproof Membranes

Some state agencies routinely apply waterproof membranes and bituminous concrete wearing courses on all newly constructed bridge decks. Recently developed protective systems, such as coated reinforcing steel, offer alternatives for new construction that are more attractive to many bridge engineers, but membranes and wearing courses are widely used in rehabilitation. There is, however, a wide difference of opinion among agencies of the desirability and effectiveness of membranes, and there are variable reports of the success of field installations.

Early membranes were built up on the deck in layers of bituminous material and reinforcement such as fiberglass cloth. Membranes using epoxy resin systems, usually with wearing courses, appeared in the 1960s. Newer developments include preformed membranes of sealer and reinforcement, which are unrolled and lapped on the deck surface, and both hot- and cold-applied liquid systems, most of which offer better stress-relief and crack-bridging characteristics than the epoxy systems. These characteristics simplify surface preparation; thorough cleaning is required, but sandblasting is not always necessary.

Most of the membranes require a 1½- to 2-in. thick bituminous concrete wearing course to provide durability under traffic, and the structure must have sufficient reserve strength to carry this additional deadload. Slippage between the deck, the membrane, and the wearing course limits the grade on which membrane systems should be used to less than 4 percent *(1)*. The completed membrane wearing course conceals further deterioration of the deck. Despite these disadvantages, the collective experience of various state agencies with waterproof membranes indicates satisfaction with at least some of the products *(10)*. An informed choice of the proper system is necessary, but membranes do have a place in the repair and rehabilitation of decks on secondary and local highway bridges.

Overlays

When such factors as the grade of the deck (more than 4 percent) limit the utility of a waterproof membrane, or when a more permanent repair is desired, an overlay can be an effective, although more expensive, alternative. Materials that have been used successfully in this capacity include the following:

Portland cement concrete overlays, usually in thicknesses of about 2 in., offer compatibility with the original deck. A low-slump concrete having a water-cement ratio of about 0.32 and a slump of 1 in. or less is often specified to provide impermeability and minimize shrinkage. Problems may be encountered in consolidating and finishing the overlay; the engineer must be sure that the screed can properly finish low-slump concrete.

Polymer-modified concrete is a portland cement concrete with a polymer added during mixing. The most widely used system contains polymeric latexes, an expensive system but one that offers superior resistance to the penetration of chloride ions *(11)*. For this reason, latex-modified concrete overlays may be thinner than those of conventional concrete.

Internally sealed concrete is a form of polymer-modified concrete that contains small wax spheres added during mixing and melted after the concrete has cured. The overlay has a minimum thickness of 2 in., and heating to 185 C is required to melt the wax to that depth. External heating is provided by an infrared heater or an electric blanket system, with deck temperatures being measured by thermocouples in the concrete. Although internally sealed overlays promise good protection for the reinforcement, they are still considered experimental and problems such as the expense of heating and cracking of the decks during heating make other types of overlays more attractive at this time.

Polymer concrete is a mixture of a fine aggregate and a monomer with initiator and promotor that is presently being used on an experimental basis. Four layers of the monomer with aggregate broadcast on it are placed and rolled with a pneumatic-tired roller. Polymerization occurs in-place in a few hours, and the final overlay is thin, about ⅜ to ½ in. thick, and apparently impervious. Although such overlays must be considered experimental, a description is included because of the advantages of their minimal added weight and the limited time that the lane must be closed.

Installation details vary with the materials. Surface preparation may be limited to a light sandblasting to clean the deck and remove laitence, but usually some concrete is removed. The depth of concrete removed can vary from ¼ in., using a scarifier, to complete removal to a level below the reinforcing steel, the only way to obtain a permanent repair in the case of chloride-contaminated decks. Removal of the concrete to a level equal to the thickness of the overlay eliminates the problem of additional dead load. Grout is an acceptable bonding agent for those overlays using portland cement concrete, and the initial layer of monomer bonds a polymer concrete overlay. Curing methods and times should be tailored to the overlay material used.

REDECKING AND WIDENING

In cases of severe deterioration it may be necessary to replace the entire deck, and replacement may at times be more economical than removing all of the concrete to a level below the top reinforcing mat. The report on the first phase of this study presented several replacement systems, including precast deck slabs, timber and steel grid decks, and permanent deck forms that may have application *(12)*. A later section of this second phase report includes descriptions of approaches in redecking the George Washington Bridge and bridges on the New York Thruway. These approaches may be helpful if very rapid replacement under heavy traffic is required.

Durability is a prime consideration in the design or repair of a deck. It is attained through the placement of concrete with low absorption, and it is not directly related to the commonly measured strength of the concrete. (Unpublished research performed by H. H. Newlon, Jr., at the Virginia Highway and Transportation Research Council indicates that the absorption should be less than 4.5 percent.) The following factors are critical.

1. There must be sufficient cover, at least 2-in. clear cover, over the reinforcing steel.
2. The water–cement ratio must be low. A value of 0.40 (0.45 with 2½-in. minimum cover) has been recommended *(8)*, and lower values are used for low-slump and high-density mixes.
3. The proper degree of aid-entrainment is required.
4. Proper consolidation is necessary.
5. The concrete must be given proper and timely curing.

The attainment of all of these goals in a bridge deck requires considerable skill, and some of the qualities, notably the water–cement ratio and degree of consolidation, are difficult to evaluate during placement. For these reasons, other protective features are often included in the deck design.

Membrane waterproofing systems were the earliest protective systems used. However, because of some problems in their installation, their relatively short service life, and their concealment of the condition of the deck, they are less popular today on new construction.

The most popular system at present is probably the use of epoxy-coated reinforcing steel in the top mat. The coatings, which are electrostatically applied in powder form on a heated bar, cure to form a durable coating that has no adverse effect on the bond characteristics of the bar. The extra care required of the contractor is not burdensome, and the additional cost of coating the bars in the top mat is a low percentage of the cost of the structure.

Concrete having low permeability is required only in that portion of the deck above the top reinforcement. The remainder of the deck must be of good quality concrete with the strength required to perform its structural function. One approach to meeting these needs is the use of two-course deck construction in which the structural slab is placed and roughly screeded at a level just above the top steel. A wearing course, or overlay, of high quality portland cement concrete or some of the special impervious concretes is added when the slab has sufficient strength. The obvious advantage to this approach lies in minimizing the quantity of the costly materials in the top course. Beyond this, quality control of the small quantity is easier, and proper cover is more easily obtained because most of the dead load deflection has occurred and the overlay load is on the composite section. If locally available fine aggregates do not provide adequate skid resistance, as is the case with crushed limestones, the needed quantity of imported fines is reduced.

In the rehabilitation or widening of decks it often is necessary to place concrete while traffic uses adjacent lanes. There has been considerable concern over the effects of traffic-induced vibration on the quality of the concrete and its bond to existing concrete and its embedded reinforcing steel. A synthesis in preparation for the NCHRP reviews a considerable amount of information pertinent to this concern *(13)*. Essentially, it was found there there was no evidence that the maintenance of traffic on a structure being repaired had an adverse effect on its service life, provided that good engineering practice and high standards of construction and inspection were followed. There was no evidence that detouring of traffic off the structure or temporary shoring of the span was required. Use of an existing detour within the right-of-way, if available, or keeping trucks out of the adjacent lane by voluntary compliance is advised, but restrictions on speed and weight or closure of the deck should be made primarily for reasons of safety. The most effective way to reduce the amplitude of traffic-induced vibrations is to maintain a smooth riding surface.

Several other recommendations are put forth in the synthesis. Among those of particular interest to the engineer contemplating an overlay or widening project are the following:

> Widenings should be attached to the existing structure. Moment transfer should be provided through the joint between the new and existing portions of the deck. Lapped reinforcing bars are preferable to dowels. The laps should be tied securely or welded. Dowels and reinforcing steel should be straight. A concrete keyway is not necessary.
>
> A closure pour is recommended to achieve a smooth surface when an overlay will not be placed on the deck. Where the fascia beam of the existing structure differs from the other beams in either section or camber, it should be removed and used as the fascia beam in the new deck.

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CHAPTER THREE

METHOD USED FOR REPLACEMENT OF THE DECK ON THE GEORGE WASHINGTON BRIDGE

In 1974 a study by the Port Authority of New York and New Jersey concluded that the upper level of the George Washington Bridge had deteriorated to a point that extensive corrective measures were necessary. After several alternatives were investigated, it was decided to totally replace the reinforced concrete roadway slab with an orthotropic steel plate deck.

PLACEMENT OF THE STEEL DECK

To minimize the undesirable effects of this project on peak-hour traffic flow, a system utilizing off-hour, nighttime installation of prefabricated panels was adopted. These panels, see Figures 1 and 2, measuring 11 ft in width and 60 ft in length, were constructed in modular form. One section of four panels was to be installed each night in order to meet the 2-year deadline for completing the project. Prefabrication was complete with a 1½-in. asphaltic concrete wearing surface so as to minimize the on-bridge construction time.

The time allocated for installation of the panels was the critical factor and required adherence to a rigid sequence of activities.

While prefabrication was being completed at a plant in nearby New Jersey, a crew prepared the bridge for receiving the new panels. The first task was the erection of scaffolding under the particular section of roadway to be replaced. Next, the crew removed the old sections in accordance with the following schedule (see Fig. 3):

1. Remove the rivets connecting the existing concrete deck slab to the floor beams. (Note "Existing Rivet Holes" in Fig. 3.)
2. Saw and jackhammer the slab into easy-to-handle sections.
3. Remove these 11-ft x 20-ft or 11-ft x 40-ft sections by cranes and flatbed trucks.
4. Ready the existing floor beams by cleaning and painting.
5. Truck the panels onto the bridge and seat them in place, using a specially designed, 4-point pick-up system that leaves no holes in the deck.
6. Install bolts through the strap plates with plate washers and through the holes left after removal of the old rivets.

With four panels bolted into place and the joints filled with a coat of tar epoxy mortar, the new 44-ft x 60-ft section of roadway was ready for the morning's rush-hour traffic. Figure 4 is an isometric sketch of the completed floor system.

ADVANTAGES AND DISADVANTAGES

Two significant advantages resulted from use of the orthotropic deck. First, and foremost, was the ability to maintain all eight lanes of traffic during the peak hours (6:00 a.m. to 8:00 p.m.). Four lanes were left open during the actual installation.

The second advantage was a significant reduction in the deadweight of the deck as compared to that of the reinforced concrete deck. The dead load of the new decking is 61 lb/ft²; a 43.0 percent reduction from the 107 lb/ft² of the original one. The new deck resulted in a 10.1 percent reduction in the overall dead load of the structure. This, in turn, provides an increased live load capacity and a reserve structural strength should general degradation of the structure occur.

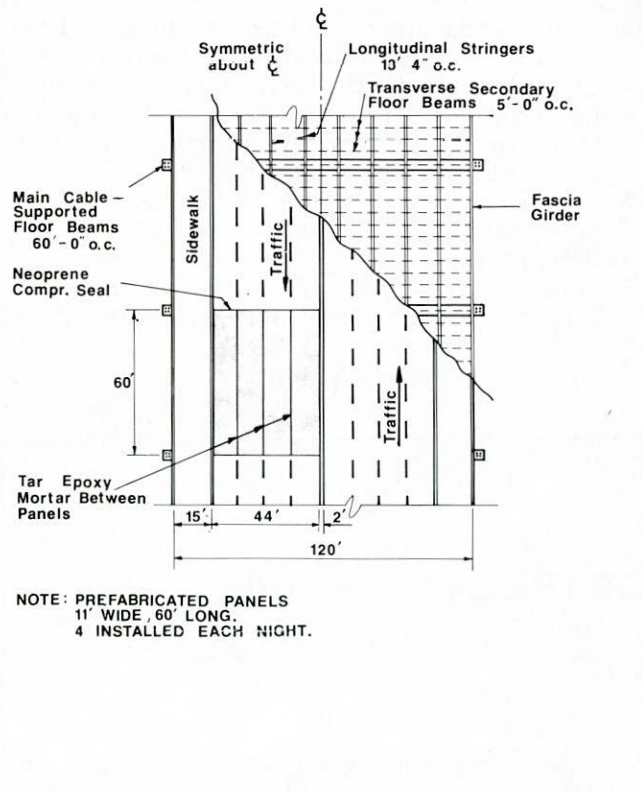

FIGURE 1. Partial Plan View of Bridge Deck System

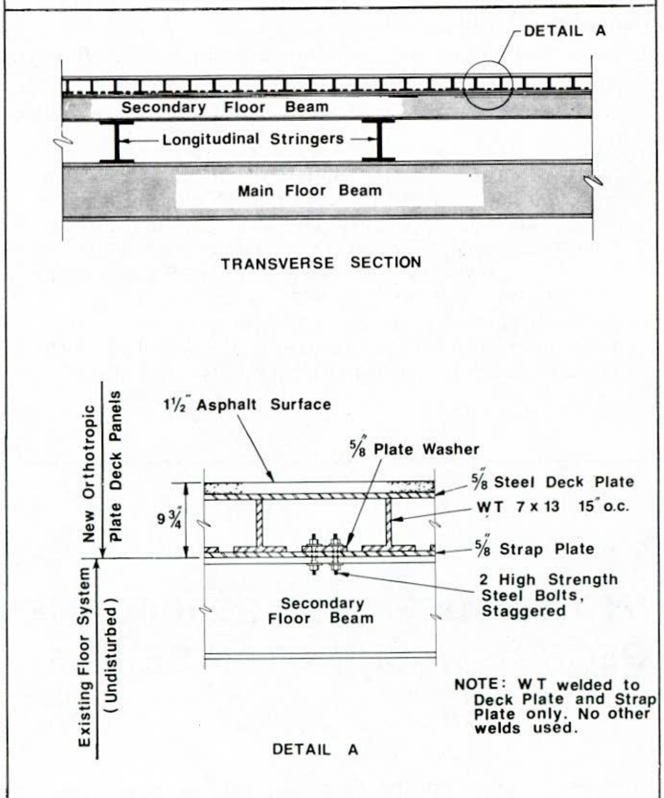

FIGURE 2. Transverse Section Through Deck

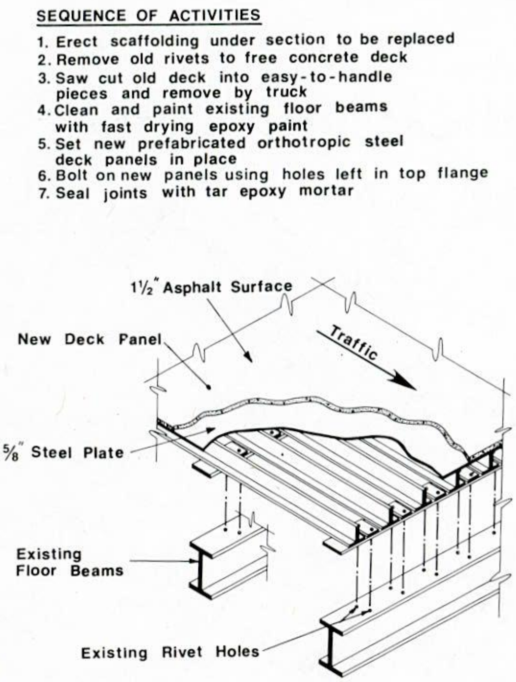

FIGURE 3. Installation Sequencing Scheme

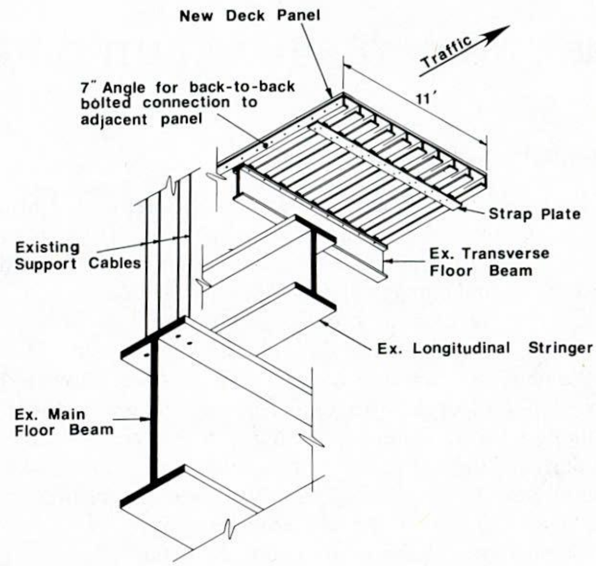

FIGURE 4. Orientation of Floor System Members

The principal disadvantage to using an orthotropic steel deck system is the cost. Costs associated with the steel itself, prefabrication, and installation can be prohibitive. Relative cost comparisons depend on specific applications.

APPLICABILITY

Because the initial cost of orthotropic steel decks is comparatively high, their use is limited to situations which impose the advantages of this structural system as requirements for construction. Applicability would include, but not be limited to, situations similar to that of the George Washington Bridge where peak traffic flow could not be disturbed. Although this situation is expected to be largely confined to urban structures carrying large volumes of traffic, the deck system may be applicable for structures on which the weight savings justify the cost. The use of standard orthotropic steel plate deck sections was described in *NCHRP Report 222*.

Also, the system might be feasible for structures from which traffic cannot be detoured because of their original importance. An example of such a structure may be one which is the sole crossing of some feature, as a waterway, in a wide area. Finally, the system would be feasible for the repair of structures that serve large numbers of emergency vehicles.

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CHAPTER FOUR

NEW YORK THRUWAY AUTHORITY DECK REPLACEMENT SYSTEM

DESCRIPTION OF SYSTEM

Faced with a substantial number of deteriorated bridge decks, the New York Thruway decided that a systems approach to deck replacement would minimize costs, needed resources, and patron delay. Working with a consultant, the Authority developed a design using precast, full-depth, structural slab elements that required little or no on-site placement of concrete (*1, 2*). The system allowed the scheduling of deck replacement operations during times of minimum traffic volumes, and it can be adapted for use on a bridge carrying traffic with minimal delays and no impedance during peak hours. Since 1973, the system has been used to replace all or part of the decks on three bridges.

Components of the system were essentially described as System S3 in the earlier NCHRP report on the rehabilitation and repair of secondary highway and local road bridges from which Figure 1 is taken (*3*). Full-depth, conventionally reinforced concrete slab sections are placed transversely across the stringers after the removal of the deteriorated deck. The dimensions are tailored to the geometric and structural requirements of each bridge and the lifting capacity of available equipment. On the Thruway bridges, the 7½ or 8-in. thick sections were 4 to 5 ft in width and up to approximately 40 ft in length.

Epoxy mortar is placed in the female-female keyways shown in Figure 1 to join the transverse edges of the slab sections. On a relatively narrow 2-lane interchange bridge the sections extend completely across the deck. A 3-ft wide longitudinal closure pour at the crown of the roadway (Fig. 2) has been used to connect 2 rows of sections on wider main-line structures. The closure pour forms the peak of the crown and develops the bond strength of reinforcing bars extending from the sections. Composite action of the slab and beam is usually provided by welding stud shear connectors inside blocked-out pockets in the sections and filling the pockets with epoxy mortar, as shown in Figure 1. A bed of epoxy mortar is also placed on the top flange of the beam before the slab section is positioned. An alternate bolted connection (Fig. 3) was used in some sections on the first installation to provide a "dry" (epoxy free) connection. The holes are drilled in the top flange of the beam using the sleeves formed in the slab section as guides. Epoxy mortar is used in the transverse joints between sections to transfer loads and ensure compatibility of deflections.

The precast slabs have the capability of supporting the erection equipment for the installation of later sections once the epoxy has cured. Cure times were 1 to 2 hours for the mortar in the shear connector pockets and about 5 hours for that in the transverse keyways, probably because the latter had less mass of adhesive (*1*). Cure times vary greatly with the ambient temperature and can be much longer in cold weather. Allowance for the cure time can be made by installing the sections from alternate ends of the bridge. For a 3-span structure the following erection sequence was used (*4*):

1. Erect 50 percent of west-end span with crane placed on west approach.
2. Erect 50 percent of east-end span with crane placed on east apporach.
3. When epoxy reaches design strength, place crane on west-end span and erect remainder of west-end span panels.
4. Place crane on east-end span and erect remainder of east-end span panels.
5. Place crane on west-end span to erect center-span panels.

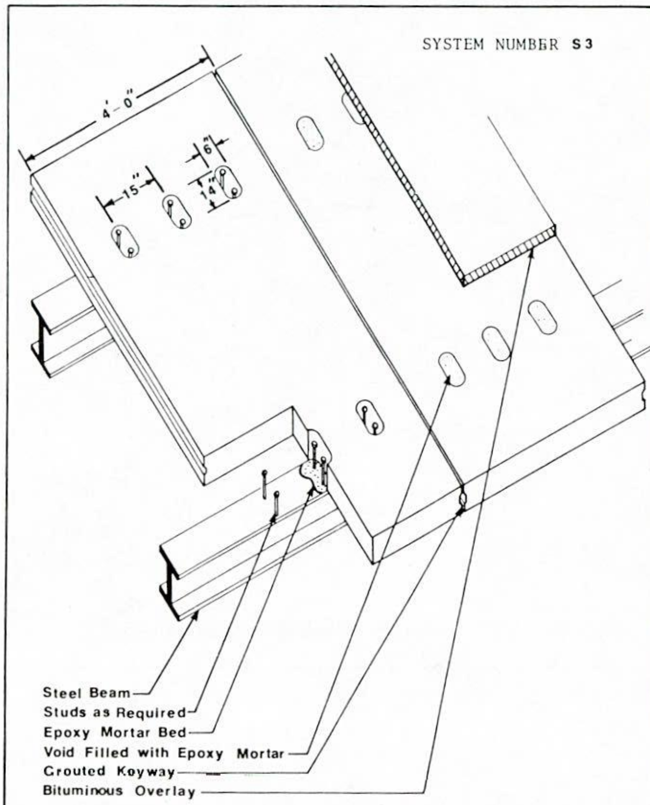

FIGURE 1. Precast Slab Sections as Used by New York Thruway Authority, with Epoxy Mortar Joint to Beam.

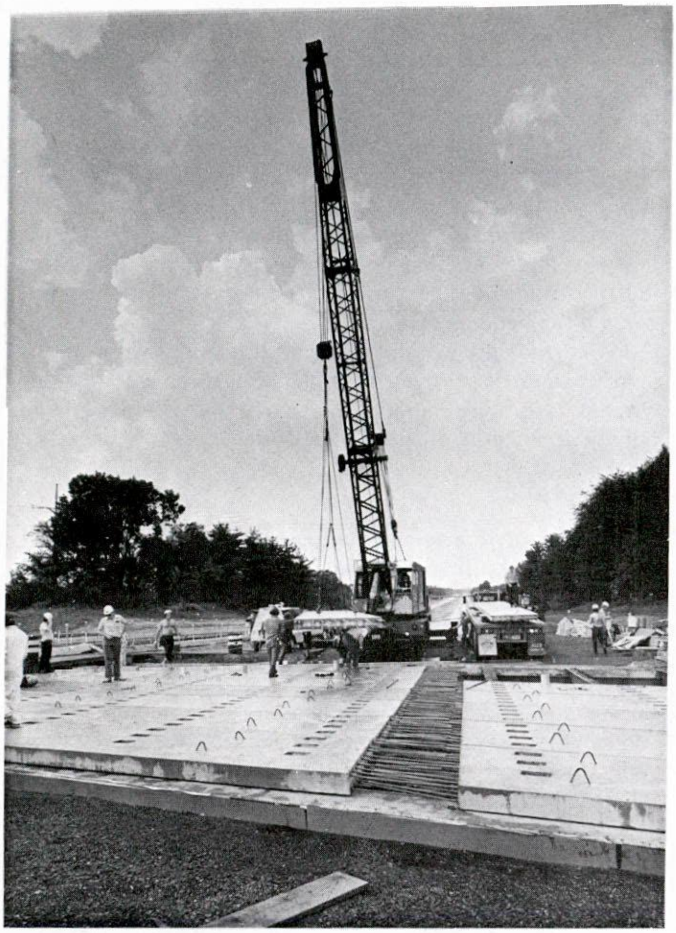

FIGURE 2. Placement of Precast Deck Slabs
(Photo by N.Y.S. Thruway Authority.)

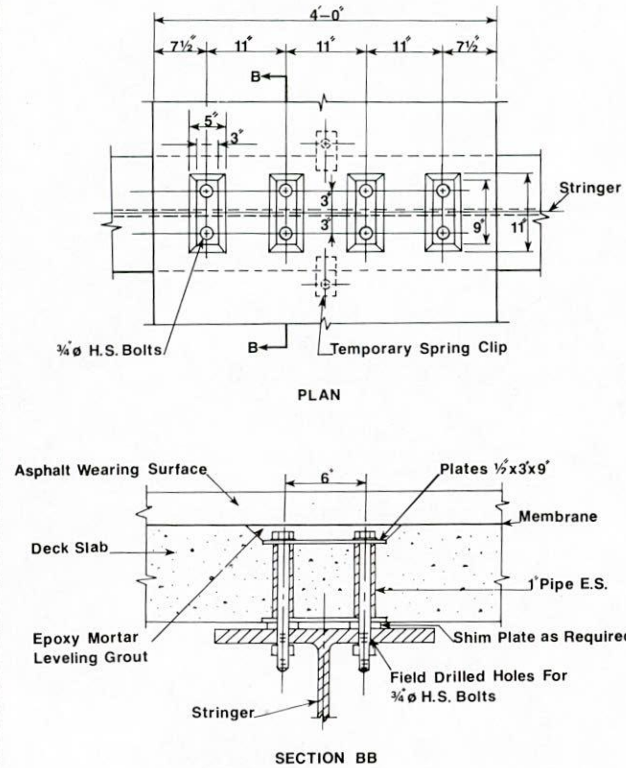

FIGURE 3. Alternate Bolted Connection

A waterproof membrane and bituminous concrete pavement was placed on the slab sections to attain the vertical profile of the roadway. Although traffic could use the sections without the payment, ramping of the sections would have been required in some cases to match the profile.

Detours have been available to carry traffic in the three instances in which the precast slab sections have been used on the New York Thruway. However, the original design contained provisions for installing the deck under traffic with only minimal delay. Traffic is supported on a set of steel plates attached by hinges to a rectangular steel tube as shown in Figure 4. One set of plates is required for each lane. The sequence of erection, keyed to the numbers on Figure 4, is as follows:

1. Cut a slot having the width of the steel tube into the deck, one lane at a time, and place the tube in the slot. The plates are attached to hinges on the tube and allowed to rest on the old deck.
2. While maintaining traffic in one lane, remove the existing deck under the plates in the other lane, raising individual plates as required. With traffic supported on the steel plates, remove the remainder of the deck. Any existing shear connectors are removed with the deck.
3. Move the hinged plate assembly forward, leaving an opening in the deck.
4. Place epoxy mortar on the stringer surfaces, drop the precast unit into place, and make connections.

Traffic would be stopped, in the example shown, only during the preparation of the stringer and placement of the new section, and the use of separate slab sections for each lane would eliminate the need for stopping traffic. At any time during the removal of the deck, all of the steel plates could be closed to accommodate peak traffic. The tube and plate assembly is composed of a number of plates for ease in handling and transporting and to allow flexibility in the width of opened roadway.

COST EXPERIENCE

The first use of the precast slab system, in 1973-74, was on a span 59 ft in length and 45 ft wide on an interchange bridge at Amsterdam, New York. The remainder of the 4-span bridge was repaired by conventional means. Maintenance forces fabricated the slabs in an open-air casting yard adjacent to the bridge site. Although the installation appeared economical, no cost data are available.

Precast slabs were next used in redecking the 1-span bridge, 45 ft in length by 123 ft in width, carrying the Thruway over Krumkill Road at Albany, New York, in 1977. Thruway maintenance forces demolished the existing deck, and the precasting company fabricated, delivered, and

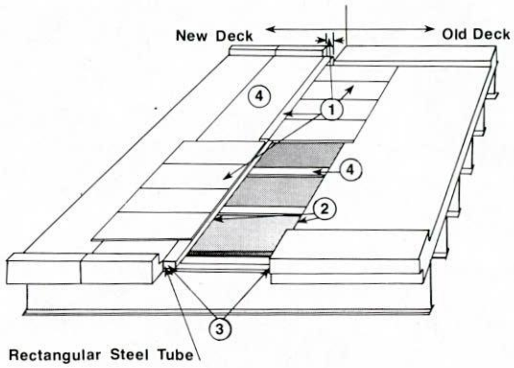

FIGURE 4. Use of Steel Plate Assemblies in Deck Replacement Under Traffic

erected the slabs and installed the shear connectors for a cost of $39,777.

The latest application of the precast slab system was in redecking a 3-span ramp bridge at the Harriman Interchange. The cost of 8,990 ft² of slab panel units was $15/ft². Epoxies, stud shear connectors, pavement, and other items were paid for separately.

PROBLEMS

Only minor problems have been encountered in the use of the precast slab sections on the three bridges mentioned previously (*4*).

1. Fine cracking has been noticed at the time of erection in the panels fabricated for the last two bridges. Possible causes include difficulty in casting around congested reinforcement and problems with mix design, curing, or handling. At this writing the cracking has not progressed.
2. Difficulties in applying the bedding epoxy on the top surface of the steel stringers resulted in many voids at the interface with the top flange. Because of this problem, it is now recommended that neoprene edge strips be placed on the top flanges of the stringers to support the panels and contain the bedding epoxy.
3. Scheduling and sequencing of the epoxy work during cold weather has proved difficult because of the extended cure times for the compounds. Use of a special cold weather polymeric material is being considered.

PERFORMANCE

The precast slab sections are a proven approach to deck replacement, with the potential for use under traffic. Inspections since the erection of the panels, representing as much as 6 year's experience, has disclosed no separation in the joints between the abutting sections or between the sections and the stringers. No damage beyond that experienced during construction and installation has been noted. Although the Thruway has not realized the degree of cost reduction and construction speed originally expected, its experience has continually improved. (Further information can be obtained from Mr. R. C. Donnaruma, P.E., Assistant Superintendent of Thruway Maintenance, New York State Thruway Authority, P.O. Box 189, Albany, New York 12201.)

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CHAPTER FIVE

REPAIR OF ACCIDENTAL DAMAGE

Most of the accidental damage to highway bridges is caused by the collision of overheight vehicles. Structures affected include both overpass bridges and truss spans with low vertical clearances. Truss spans are also occasionally damaged by the shifting of loads carried on crossing vehicles. Impact by debris during periods of high water can cause similar damage. Although less frequent, fires on or beneath a bridge cause serious distress that can be difficult to evaluate and correct. In any event, the engineer must face the problems of assessing the extent of the damage and determining the proper course of repair.

Although guidelines for the use of various repair techniques and materials are available, most agencies rely heavily on the judgment of experienced personnel. Two studies on the repair of bridge structural members have been conducted under NCHRP sponsorship: "Evaluation of Repair Techniques for Damaged Steel Bridge Members" (*1*) and "Damage Evaluation and Repair for Prestressed Concrete Bridge Members" (*2*). Both of these publications provide reviews of the state of the art in damage assessment and repair. The September 1980 issue of *Concrete International*, published by the American Concrete Institute, had as its theme concrete repair and restoration. Several articles describe procedures adaptable to the repair of both superstructure and substructure elements. Other publications by the ACI describe the use of epoxy resin compounds (*3*) and pneumatically applied mortar (*4, 5*).

The most effective means of countering impact damage is through prevention by increasing vertical clearances. The first-phase report of the present study describes techniques for modifying the portal bracing on through trusses (Repair Procedure G-1) and elevating the superstructure of an overpass bridge (Repair Procedure G-2) (*6*). Other repair procedures are applicable to structures, particularly metal trusses, that have received impact damage.

INSPECTION AND ASSESSMENT OF DAMAGE

The inspection of a damaged structure must be comprehensive and be performed by trained, observant personnel. Where there has been a significant alteration of the section of the structural member, the inspection must be made by a structural engineer. A brief initial inspection may be required to estimate the ability of the bridge to safely carry traffic and to establish traffic control. The more time-consuming, comprehensive inspection should concentrate on the collection of detailed factual data on the state of the bridge, and a set of original plans should be available. It has been recommended that the inspection and engineering assessment of damage be regarded as separate functions (*2*). The assessment phase would be performed off-site using data from the inspection with appropriate computations to determine the most effective corrective action. Often the member is repaired in-place instead of being replaced. Both functions, inspection and assessment, can, of course, be performed simultaneously if required to minimize the time before the repair of a critically important bridge is begun. In such a case the personnel performing the inspection and assessment must be extremely well qualified.

When inspecting concrete bridges, various agencies use a pachometer to locate and determine the cover over reinforcing steel, a hammer for sounding, a measuring microscope or feeler gages to determine crack width, pulse velocity testing equipment, and coring equipment. Useful equipment also includes a flashlight, mirror, and magnifying glass. The detection of cracks in steel bridges may require magnetic particle, dye penetrant, or ultrasonic testing in addition to a visual inspection.

SELECTION OF REPAIR PROCEDURE

The appropriate repair procedure is often clearly dictated by the nature and severity of the damage to the bridge. However, particularly in the case of concrete members, there may be alternative procedures. A choice between the repair of the member and its complete replacement must always be made.

Major factors considered in such decisions include the effectiveness of the repair in restoring the structural capacity and the service life expected after repair. Systematic approaches to integrating a number of factors, listed as follows, into the decision process have been provided in earlier NCHRP reports (*2, 6*). Among the factors that may be considered are the following:

1. Required structural capacity.
2. Durability.
3. Anticipated future use of structure.
4. Cost.
5. Speed of repairs.
6. Inconvenience to users.
7. Experience.
8. Available contractors.
9. Material availability.
10. Labor requirements.
11. Environmental priorities.
12. Esthetics.

Both earlier studies cited used a value engineering approach, in which weights are assigned to the chosen criteria and points are credited to the repair schemes in proportion to their adequacy in meeting each criterion. Such factors as service load capacity, ultimate load capacity, overload capacity, and fatigue life can be averaged to provide the rating for structural adequacy.

Certain considerations depend on the material in the structural member. Repair materials should respond similarly to changes in temperature and to the applied loading, and they should blend in appearance (*7*). The type of steel in a steel beam will determine its weldability and suitability for mechanical straightening. Replacement operations should utilize identical members if possible, but always the same material.

REPAIR OF CONCRETE MEMBERS

For the purposes of organizing repair strategies, the NCHRP-sponsored study on the repair of prestressed concrete beams recognizes four degrees of damage discussed below (*2*). Although the information was developed for prestressed members, much of it is applicable to the repair of conventional reinforced concrete beams and to general concrete repair.

Minor Damage

Minor damage is damage to only the concrete portions of the beams. There may be extensive spalled areas, but no reinforcing bars or prestressing strands are exposed. Cracks emanating from damaged areas are less than 3 mils in width.

Minor damage is essentially cosmetic and is usually repaired by patching with an epoxy mortar. As is the case with all repairs, preparation of the surface is of critical importance. All unsound concrete must be removed, usually by hand tools, and the surface must be cleaned of any dust, oil, or other contaminants. The epoxy resin mortar, which usually contains 4–7 parts of clean silica sand to 1 part resin by weight, must be formulated in strict accordance with the manufacturer's recommendations (*3, 7*). A bonding coating of neat epoxy resin is used to prime the surface. The depth of the patch is ½ to 1 in. minimum at its edge to avoid feather-edging. Patching restores the depth of cover over the steel to provide corrosion protection.

Moderate Damage

Moderate damage is that limited to concrete portions of the beam, although reinforcing bars or tendons may be exposed. Cracks may be 3 mils or wider, but they are closed below the surface damage. No bars or tendons are severed.

Patching is usually done with an epoxy mortar or with a special concrete mix. The depth of the patch is usually 1 in. minimum, and welded wire or other reinforcement is used as required. A bonding coat of epoxy resin or cement grout is used. More detailed information on patching can be obtained from most suppliers of materials or from the literature (*3, 7*).

Cracks 3 mils to 0.25 in. in width, as measured by a feeler gage or measuring microscope, can be sealed by injecting epoxy resin into them (*3, 7*). Wider cracks can be filled with a system containing a mineral filler. The chosen epoxy resin system should be compatible with wet conditions for field applications. Although some cracks extending downward from nearly horizontal surfaces can be filled by the force of gravity, pressure injection is generally used, with the two components of the resin system being mixed at the injection nozzle. The crack is routed at the surface of the beam and blown clean. Entry ports, which may be short sections of ¼-in. tubing, are spaced along the crack and secured with epoxy. The remainder of the routed crack length is sealed with an epoxy system until flush with the beam surface. The spacing of the entry ports is a function of the desired depth of penetration, based on the assumption that the resin travels as far into the crack as along its face. The location and spacing of the ports are also affected by the crack width and the depth or thickness of the member, if it is fully cracked. The injection of the epoxy begins at the bottom of a vertical crack or at one end of a horizontal crack, and it continues until the resin flows from the port spaced at the desired depth of penetration. The injection nozzle is then moved to the next port and the port used is closed. Intermediate ports at spacings closer than the depth of penetration are occasionally used to monitor the flow of epoxy, and these are closed when epoxy appears without moving the nozzle. Some epoxy compounds are injected through suction fittings eliminating the need for tubing ports.

Epoxy injection cannot be used to repair cracks that open under normal loads, but it is widely used to repair collision damage. The technique is well documented (*2, 3*), and advice on the suitability of epoxy injection and the details of the application are available from a number of epoxy resin suppliers.

Severe Damage

Severe damage affects both the concrete and the reinforcement or tendons in the beam. The damage may consist of one or more of the following:

1. Cracks extending across the width of the bottom flange but closed below the surface damage.
2. Major or total loss of concrete section in the bottom flange.
3. Major loss of concrete section in the web, but not occurring at the same location as loss of concrete section in the bottom flange.
4. Severed prestressing strands or strands that are visibly deformed.
5. Horizontal misalignment of the bottom flange in excess of the allowed standard tolerance and vertical misalignment not exceeding the normal allowable. If horizontal misalignment of the bottom flange is within permissible limits, and the girder web is not cracked at the interface with the top flange, web misalignment can normally be considered within allowable limits.

Severe damage requires extensive repair supported by structural analysis. It is often necessary, in the case of severe or critical damage, to restrict the flow of traffic until a preliminary structural assessment is performed and any needed temporary shoring or strengthening is installed. Patching and epoxy injection techniques similar to those described earlier are used, often in conjunction with preloading to restore the live-load capacity. Splicing may be required where tendons have been damaged.

Preloading is the application of a temporary vertical load during the repair of a damaged prestressed member. The loading is applied through the use of a heavy vehicle or by jacking against a longitudinal beam over the damaged member. Preloading is used when a substantial portion of the concrete has been lost, but the strands remain intact, to reduce excessive compressive stress in the concrete or to restore partial or full prestress to the repaired area to reduce or eliminate tensile stresses under live load. Because the repair of severe damage to a prestressed beam will invariably require an engineering analysis, it is advisable to include an investigation of the applicability of preloading. Performed prior to epoxy injection and patching, it can result in the repair beam having a live load capacity equal to that of the undamaged member. Sample calculations and examples of the use of preloading are provided in Ref. (*2*).

Also provided in Ref. (*2*) are calculations and examples of splices to restore prestressed beams in which tendons have been severed. One such splice was also described in the first phase of the present study as repair procedure C-6 (*6*). Splices are shown which can restore the strength of practically any number of broken strands in a standard AASHTO I-beam. Application to other sectional shapes would not be difficult.

Critical Damage

Critical damage affects both the concrete and the steel or prestressing tendons, as described by one or more of the following conditions:

1. Cracks extending across the bottom flange or in the web directly above the bottom flange damage and not closed below the surface damage. (This indicates that the prestressing strands have exceeded yield strength.)
2. An abrupt lateral offset as measured along the bottom flange or lateral distortion of exposed prestressing strands. (This also indicates that the prestressing strands have exceeded yield strength.)

3. Loss of prestress force to the extent that calculations show that repairs cannot be made.
4. Vertical misalignment in excess of the normal allowable.
5. Longitudinal cracks at the interface of the web and the top flange that are not substantially closed below the surface damage. (This indicates permanent deformation of stirrups.)

Critical damage requires the replacement of the member. Girders are commonly replaced by cutting the deck slab and connecting diaphragms and lifting the damaged girder out of the superstructure. Responses from state highway and transportation departments indicated that the longitudinal cut line is either at the centerline of the adjacent girder left in place or approximately at the quarter point of the slab span between girders nearest the girder left in place (*2*). Sufficient reinforcing steel is left projecting from the slab and diaphragms to lap with steel in the replacement section. Sawcuts ½ to ¾ in. deep are made in the deck to provide a neat appearance at the cut lines. The Minnesota Department of Transportation has developed a procedure for removing the girder from beneath the bridge. Longitudinal steel beams support the damaged girder. The slab steel is left in place during this replacement scheme. Details of this procedure and those developed by other agencies are provided in Ref. (*2*).

Fire Damage to Concrete Members

The repair of fire-damaged concrete is similar to the repair of impact damage, and the procedures described earlier are generally applicable. However, the inspection and assessment of the damage, particularly a determination of the length of time of exposure to the fire and the temperature levels attained, can be most difficult. It is generally advisable to sandblast the structure to remove smoke and fire stains and loose particles and to expose any cracking. Sounding with a hammer can disclose loose concrete remaining after sandblasting, and petrographic analysis of cores can be used if necessary to determine the extent of the damage.

Changes in the coloration of the coarse aggregate can provide an indication of the temperature levels, which can then be related to an assumed strength loss of the concrete. Pulse velocity tests have also been used to evaluate the concrete strength, and tensile tests of sections of the reinforcing steel indicate its strength.

Pneumatically applied mortar, or shotcrete, has been used successfully to restore beams, slabs, pier caps, and columns when a large expanse of concrete has required variable-depth repair. The dry mix shotcrete process is most widely used in the restoration of bridges. The preparation is similar to that for other patching, except that the area to be repaired must have a shape that does not entrap rebounding particles. An epoxy resin bonding agent can be used to prime the surface if bond is questionable, but the application of a cement grout, or mortar bonding course, should be avoided. Anchor bolts may be used to provide mechanical bond, and reinforcement is often placed in deep patches. Finishing of the repair surface should be minimal. Specialty contractors are usually employed to assure the use of proper application techniques. Information on the use of shotcreting is available in Refs. (*4*) and (*5*).

REPAIR OF STEEL MEMBERS

The repair of steel members requires considerable experience because of the relationship of the repair techniques to the properties of the metal. For this reason, every effort must be made to determine the type of steel used in the member. The planning of repair operations must include considerations such as the welding and straightening procedures that may be used and the temperature during heating. In the case of a bent member, the sequence in which the straightening forces will be supplied can be as important as the location of the forces.

The NCHRP-sponsored study of the repair of steel bridge members assembled the practices of many state agencies and provided available information on the effect of the repairs on the properties of various steels (*1*). A bibliography of articles and reports providing background information is also provided. The following repair techniques are discussed.

1. Welding of minor defects such as small, shallow dents, nicks, gouges, and cracks.
2. Removal of a more severely damaged portion of a member and welding a replacement section in place.
3. Mechanical straightening of bent members either cold or with the application of heat to make the metal more workable.
4. Flame straightening of bent members.

Complete replacement of the member can be considered a final alternative, and this is the general practice of a few states. Most agencies, however, will repair a member in place when possible.

Repair of Defects

The repair of such defects as dents, nicks, and gouges involves grinding the defect to an acceptable contour and filling the area with weld metal. Nicks less than ¼ in. deep and gouges can be repaired by simply grinding the area to blend the defect smoothly into the surrounding material. When welding is required, the sharp root of the defect is ground to provide a generous radius and the area is inspected, as described in the following, to ensure that no cracks are present. The cavity is filled with weld metal, generally using the shield metal-arc (covered electrode) process. The welding procedure must be qualified, and the new welding should be ground in the direction of stress to restore the original shape of the member.

Repair of Cracking

The examination of the structure for cracking is an important part of the initial inspection of an impact-damaged bridge. Visual inspection with a magnifying glass of the cleaned surface of a member for the existence of cracking may be sufficient, but many agencies supplement this by magnetic particle, dye penetrant, and ultrasonic inspection. The use of these procedures is advisable if the area is covered with scale or rust, and the first two procedures are more easily performed and less expensive than the last. The examination of the structure for cracking should not be limited to the immediate area of impact. Cracking can occur at points well away from the impact, as the shock is transmitted through the structure. Spalling of the paint or scale on the structure can indicate cracking.

Holes are drilled at the extreme ends of the crack to prevent its progression through the member, and the crack is removed by grinding or by arc or flame gouging followed by grinding. As shown in Figure 2 of Chapter 6, the recommended location of the center of the hole is beyond the apparent tip of the crack. The actual crack can extend beyond the apparent tip on the surface of the member. It is most important that the crack be completely removed prior to filling the groove with weld metal. The welding procedure and welder must be qualified, and radiographic inspection of the weld may be desirable. This technique is not useful for repairing tears in steel members. Because of associated distortion of the edges, tears are more appropriately repaired by replacing the damaged portion, as described in the following section.

Partial Replacement of Member

When damage to a portion of the member does not lend itself to repair by welding or straightening, the damaged area is commonly cut out and replaced. Rectangular cut-outs with horizontal and vertical sides are generally made, with rounded intersections. Great care is advisable in measuring the dimensions of the cut-out to obtain a good fit of the replacement section. In some instances the replacement section is fabricated oversize and match-marked in the field for an improved fit. Welding procedures and welders should be qualified, and radiographic or other nondestructive testing of the welds is necessary. The welds may be ground flush with the member surface if desired, but grinding must be done parallel with the direction of stress. Imperfections, such as grinding marks perpendicular to the direction of applied stress, can lead to the development of fatigue cracks.

Mechanical Straightening

Mechanical straightening is the use of external loads, applied by hydraulic jacks or mechanical winches, to bend a member into alignment. It is useful when the damaged area is plastically distorted but of uniform section. The straightening can be done with steel at ambient temperature or hot. Heat is applied to increase the workability of the steel so that less force is required and there are no cold-hardening effects on the steel. Cold mechanical straightening can adversely affect the toughness properties of certain classes of steels, and use of the procedure is not recommended on low strength (A7 and A36) and high strength (A242, A441, A572 and A588) steels (*1*). Hot mechanical straightening, with a temperature limit of 1200 F, is recommended instead. Mechanical straightening of sharp bends or kinks is prohibited by the AASHTO bridge specifications. Judgment is required in determining the application of the load. Loading schemes and a complete discussion of the effects of both mechanical and flame straightening on the properties of the steel are given in Ref. (*1*).

Flame Straightening

Flame straightening, less widely used than mechanical straightening, is recommended only for low strength (A7 and A36) steels, as problems may be encountered with other classes (*1*). In the process, the distorted portion of the member is restored to alignment by controlled thermal expansion and contraction. The heat is applied, usually by means of an acetylene torch, to an area of carefully controlled size, shape, and location on the convex side of the bend. In Figure 1, for example, the heat is applied in a wedge-shaped pattern. Longitudinal expansion of the metal in the heated area is constrained by the surrounding cold metal, so that expansion occurs primarily in the thickness of the piece. Upon cooling, longitudinal contraction occurs, and the member straightens. Auxiliary forces are often applied to increase the effectiveness of the operation by further restraining the longitudinal expansion or by tending to straighten the member.

As indicated in Figure 1, heating begins at the apex of the wedge and the torch is moved along a serpentine path as the area under it reaches the desired temperature, 1200 F. The temperature level is often gaged visually by the welder, but temperature-indicating crayons are available.

A comprehensive discussion of flame straightening is provided in Ref. (*1*). Equipment needs and the application of the process to the more complex shapes of common bridge members are described. Responses of state agencies to an inquiry made during the NCHRP study indicate that 6 states use flame straightening extensively, considering it a successful and cost-effective procedure, and 12 states use the technique occasionally. Wider use is apparently limited by concerns for the effect of heating on the properties of the steel and for residual stresses that may be induced in the member. Mastery of flame straightening does require considerable experience, but the technique offers significant advantages in applicability, rapidity, and economy.

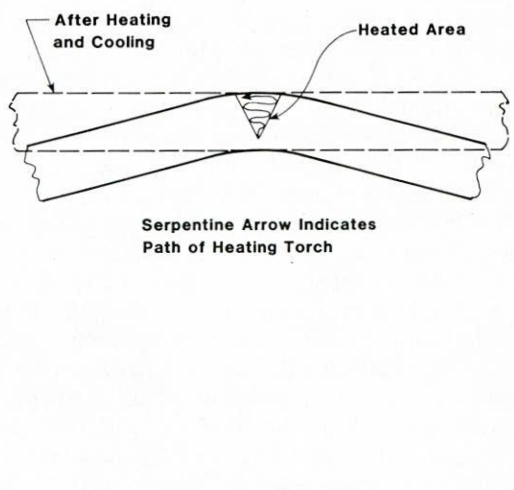

FIGURE 1. Flame Straightening of a Bent Piece. (From Reference 1.)

Total Replacement of Members

Total replacement is required if the distortion of a steel beam or girder is judged to be beyond correction, and a very few agencies replace all damaged members (*1*). The procedure for replacing a steel beam is essentially the same as that used for concrete members; the deck concrete above the member is removed, leaving a sufficient length of reinforcement at each side for bond development. The procedure requires a great deal of careful chipping to avoid damaging the steel, and it is a particularly difficult operation in the case of a composite beam. The report on the first phase of the present study describes a procedure (repair procedure S-5) in which removal of the deck is avoided by cutting the web of the damaged member just below the top flange (*6*). The bottom of the top flange is then ground smooth, and a new member having a narrower flange width is connected to the flange with continuous fillet welds.

Fire Damage to Steel Members

As is the case with concrete members, the most difficult task in restoring steel members damaged by fire is evaluating the temperature to which the metal has been exposed. Research on flame straightening has indicated that exposure to temperatures below 1200 F does not adversely affect the properties of common bridge steel. Distortions are commonly repaired using the procedures described previously. Bent components have been straightened with vertical stiffeners being added in some cases. Severe local distortions have been removed by cutting and a replacement section attached by welding. Some repair to the concrete underdeck or adjacent substructure elements is usually required.

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CHAPTER SIX

REPAIR OF FATIGUE DAMAGE IN STEEL BEAMS

Responses to a questionnaire on bridge deficiencies distributed during the first phase of the subject study indicated that fatigue damage was seen as a relatively minor and infrequent problem on secondary highways (*1*). The rarity of occurrence of fatigue cracking can be ascribed to the fact that, in general, secondary highways and most local roads carry low volumes of heavy trucks. Yet, fatigue must be considered a potential problem. Older bridges, designed for lighter vehicles under specifications without fatigue provisions, may have members with details that are now regarded as fatigue-critical exposed to high stress ranges with the passage of today's heavy trucks. Further, a review of repair procedures during the earlier phase of the study disclosed several instances in which field welding and cutting of members were done with no consideration of fatigue. Welding is often done to expedite completion of the repair, but inadvisable details can greatly shorten the fatigue life of the bridge. The current *AASHTO Standard Specifications for Highway Bridges* gives permissible stress ranges for common types of welded details exposed to a range of load repetitions (*2*). This specification, which incorporates information developed in research sponsored by the NCHRP, should be consulted not only during the design of a bridge but in planning repair operations and selecting a repair technique. This section summarizes recent work on fatigue to provide information on locating fatigue distress and repairing the structure. Types of details to be avoided in rehabilitating bridges are also briefly reviewed.

Comprehensive fatigue provisions were included in the 1965 AASHTO (then AASHO) bridge specifications. These included general mathematical relationships to be included in bridge design, but additional experimental data were desirable to define the effects of the many related variables. In 1966, the NCHRP initiated a series of studies conducted by Lehigh University and the United States Steel Corporation. An experimental study of 374 steel members with various welded details by Lehigh University found the stress range to which the detail is subjected to be the dominant variable in its fatigue life (*3*). Suggested revisions were adopted in the 1971 AASHTO Interim Specifications. A continuation of the effort, also performed by Lehigh University, reaffirmed the stress ranges as the dominant factor in the fatigue life of 157 specimens (*4*). The framework for the current categories of details was developed, and the comprehensive revision was adopted in the 1974 AASHTO Interim Specifications.

Research at U.S. Steel has included studies of fatigue crack growth behavior of a variety of bridge steels (*5*) and the fatigue life of bridge members under realistic variable amplitude, random sequence loadings (*6*). The latter study provides a method for computing the remaining life of an existing structure using estimated truck volumes and the fatigue characteristics of various details. The procedure may be useful in evaluating an old structure designed for light loads or one with undesirable design details.

Later research at Lehigh University has been published in *NCHRP Report 206*, "Detection and Repair of Fatigue Damage in Welded Highway Bridges" (*7*), which is particularly useful to the engineer involved in bridge rehabilitation. The report discusses the repair of fatigue cracking by several means, and the appendixes discuss nondestructive methods for detecting cracks and the classification of welded details. A complete discussion of the various techniques for detecting fatigue cracks is also appended to the report of the FHWA's development of an acoustic crack detector and magnetic crack definer (*8*). A more recent study by Lehigh University, "Fatigue Behavior of Full-Scale Welded Bridge Attachments" (*9*), contains additional information on retrofit techniques.

FATIGUE DESIGN

Fatigue design is not within the scope of the present study. However, a brief review of the AASHTO fatigue provisions may be helpful in interpreting the research on the detection of fatigue cracks and the retrofitting of damaged members.

The fatigue life of a given welded detail, the number of cycles of loading that it sustains before failure, is predominantly determined by the range of stresses to which the detail is subjected during loading (*3*). In the *AASHTO Standard Specifications for Highway Bridges* fatigue design is handled by specifying allowable ranges of stress for seven categories of welded connections for various numbers of stress cycles (*2*). There are tables for both redundant and nonredundant load path structures, the latter being those where failure of a single element could cause collapse. The number of stress cycles, from 100,000 to over 2,000,000, is chosen on the basis of the average daily truck traffic and the class of highway or street. The categories of details, A through F with two E categories, are described in the text of the specification and by illustrations. The use of a stress range simplifies the design computations and reflects the results of the experimental work described earlier.

Category A represents the fatigue life of rolled shapes and plate without a welded detail. On the basis of tests of rolled beams, it provides the upper limit to the fatigue strength of any detail.

Category B, which applies to a variety of welded beam details, is based on tests of welded, built-up beams. The fatigue strength is about 75 percent of that of rolled sections.

Category C covers stiffeners and attachments with a minimum length (up to 2 in.) in the direction of the applied bending stress. Fatigue cracks grow from the toes of the terminating welds, limiting the fatigue strength to 50 percent to 55 percent of that of a rolled section.

Category D covers fillet- or groove-welded flange or web attachments having lengths of 2 to 4 in. in the direction of the applied stress. The fatigue strength of these details is 40 percent to 45 percent of that of a rolled section.

Category E, which places the most severe limitations on fatigue strength, covers a wide range of fillet- and groove-welded details. The fatigue strength of details with attachment lengths over 4 in. rapidly approaches that of partial-length cover plates, which have only 10 percent to 30 percent of the fatigue strength of a rolled section. Category E' further limits the fatigue strength of partial-length cover plates on beams with a flange thickness greater than 0.8 in.

Category F, which applies to the shear stress acting on the throat of fillet welds, seldom controls a design. The shear stresses in the welds are generally low, and fatigue cracks form instead at the weld toe termination (*7*).

Several examples of category E details, taken from Ref. (*7*), are shown in Figure 1. These and similar details should be avoided, if at all possible, both in the design and rehabilitation of a bridge. The small double-headed arrows on the details indicate both the direction of the stress field and the critically stressed point in the base metal adjacent to the weld. The full text of the AASHTO fatigue provisions should be consulted when planning operations to repair or strengthen a bridge (*2*).

DETECTION OF FATIGUE CRACKS

Nondestructive Test Methods

As with any type of damage to structural members, the first step in repair is to define the extent of the distress. There are several potential methods for locating and defining the extent of fatigue cracks, but not all of the procedures are presently suitable for use in the field. Descriptions and comparisons of the test methods are provided as background for the FHWA's development of an acoustic crack detector and magnetic crack definer (ACD and MCD) (*8*). A more selective survey of those methods suitable for in-situ testing to locate cracks on existing bridges was also performed during the NCHRP's series of studies (*7*). The latter review was limited to five methods, 1/m's; X-ray, ultrasonic, dye penetrant, magnetic particle, and eddy current 1/m's, all of which were believed applicable or sufficiently promising to warrant evaluation. Descriptions of the techniques and detailed comparisons of their effectiveness are provided.

It was concluded that ultrasonic examination, which can detect both surface and internal defects and is capable of defining the shape and depth of the crack, would be the most effective method if only one could be used. However, a combination of ultrasonic and magnetic particle testing was regarded as the best approach. The latter procedure was regarded as superior to the use of a dye penetrant test, which is the other common and relatively inexpensive method. Both the magnetic particle and dye penetrant tests are limited to defining surface defects, but magnetic particle examination will define tighter cracks. Although it was acknowledged that X-ray examination, properly performed under controlled conditions, should yield satisfactory results, the method was judged less than ideal for detecting fatigue cracks. The use of the eddy current technique was found promising but in need of further development. Further development also appeared needed in the case of the ACD and MCD instruments developed for the FHWA.

In ultrasonic testing, high-frequency sound waves passing through the metal are reflected by a defect, such as a cracked surface, or by an external surface. Thus, the technique can detect cracks in depth. One principal disadvantage is the need for a skilled operator. Conversely, magnetic particle testing is limited to the detection of surface or near-surface

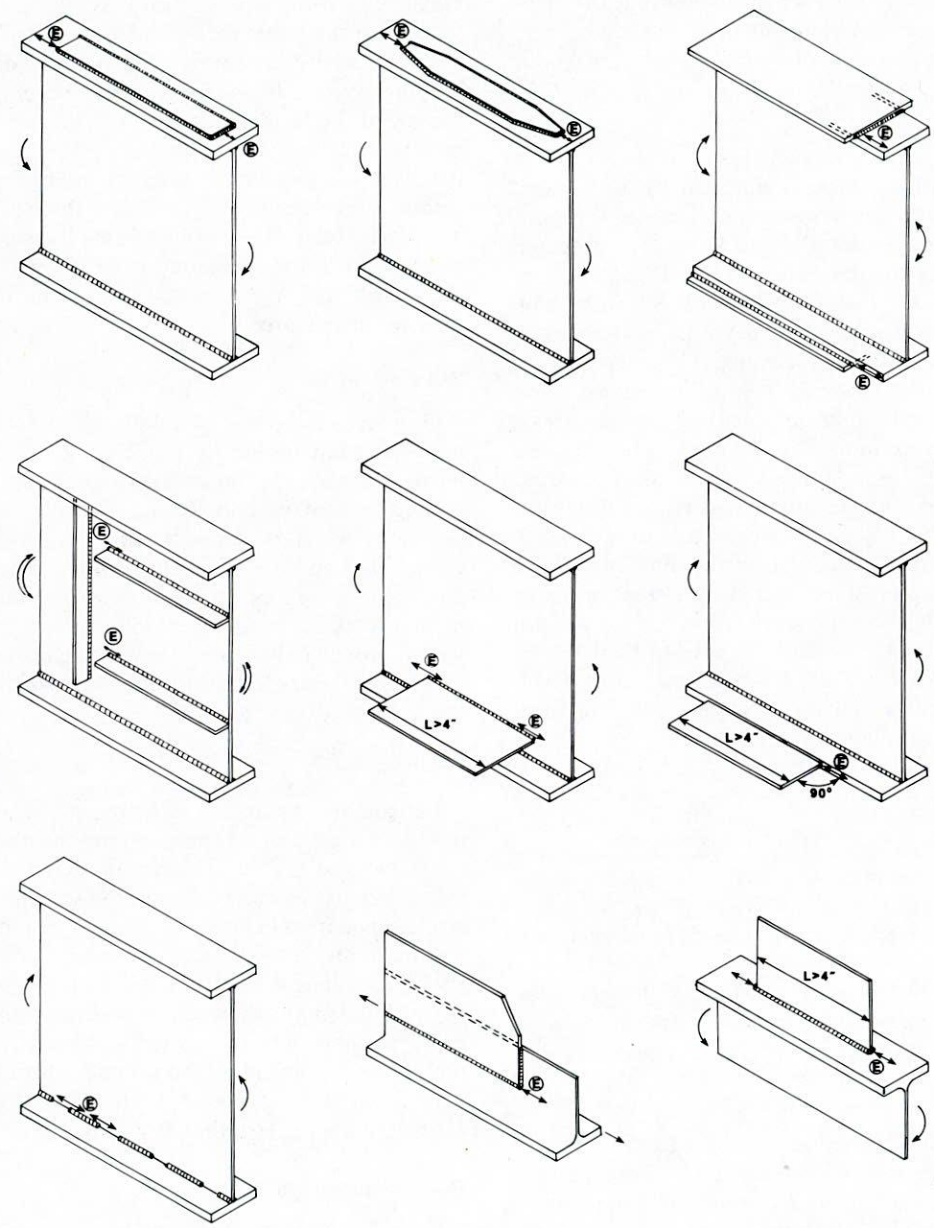

FIGURE 1. Examples of Welded Details in Category E. (From Reference 7.)

defects, but it can detect fine cracks without relying on a highly skilled operator.

Laboratory fatigue tests have shown that crack growth at the toe of terminating fillet welds can be detected at about the same time by ultrasonic inspection and visual inspection with 10X magnification (*7*). Small cracks could also be found in the field with 10X magnification if the paint, dirt, and oxide on the member surface were removed by blast cleaning within 24 hours of the inspection. However, field experience on structures has demonstrated that inexperienced inspectors may fail to discern cracks of significant length, although the cracks can be clearly seen when pointed out (*8*). This weakness of visual examination indicates the advisability of supplementing visual inspection with, at least, magnetic particle or dye penetrant testing.

Fatigue Crack Locations

The detection of fatigue cracks is enhanced by the fact that the critical locations for cracking are predictable. A report, "Inspecting Steel Bridges for Fatigue Damage," prepared by Lehigh University for the Pennsylvania Department of Transportation and the Federal Highway Administration (*10*), illustrates in considerable detail the primary locations at which fatigue cracking is likely to develop. Critical points are often located at the toes of welds in specific regions, those with high stress concentration and residual tension stress, and at which there is an initial discontinuity. Those details included in category E, as in Figure 1, are the most susceptible to fatigue cracking (*10*). Because stress ranges are generally low, details in other categories are unlikely to experience crack growth (*7*). Fatigue cracking is further limited to those portions of the structure subject to tensile stress ranges or stress reversal; details in compressive stress regions will not present problems. Discussions during a visit to Lehigh University, made as part of the FHWA's previously cited development study (*8*), generated the following order of most probable locations for fatigue cracks:

ORDER FOR PROBABLE CRACK LOCATION	TYPE OF WELDED JOINT
First	End of cover plate
Second	Other joint configurations which can experience crack growth from weld toes, i.e., attachments, etc.
Third	End of stiffener welded to tension flange
Fourth	End of stiffener not welded to tension flange
Fifth	Tension flange fillet weld on welded beams

Figure 1 used double-headed arrows to indicate the direction of the stress range and the critical point for fatigue cracking. Cracking initiates at a microdiscontinuity in the toe of a weldment when the applied stresses are perpendicular to the weld toe. Large internal discontinuities perpendicular to the applied stresses occur only at the termination of longitudinal welds and at transverse welds.

The initial discontinuity can be a gas pocket, blow hole, slag inclusion, undercut, or similar flaw. The crack grows in a semieliptical shape until it penetrates the thickness of the load-carrying element. Several cracks may form individually at several points along the toe of a critical weld and eventually join to form a long, shallow crack (*8*).

RETROFITTING FATIGUE-DAMAGED MEMBERS

An important aspect of the NCHRP-sponsored research has been evaluating the effectiveness of a promising method for retrofitting fatigue-damaged members to allow them to remain in service (*7, 9*). Earlier retrofitting methods included installing bolted splice plates across the crack to carry the load and reduce the effective stress range at the crack tip or completely removing the crack and filling the gap with weld metal. The former procedure was effective but expensive; failures of the latter repair had been noted after only a few years of additional service. Methods evaluated in the research program included grinding, peening, and remelting the toe of the weld to improve the fatigue strength of members with critical details or subcritical cracking. Complete details of the procedures used, including equipment requirements, are given in Ref. (*7*). These three retrofit procedures are applicable to the repair of small surface cracks. Large cracks and those emanating from internal flaws will likely require a bolted splice across the crack location, as described later in this chapter.

Grinding

Grinding of the toe of a weld is an accepted method of improving fatigue life by reducing stress concentrations at intrusions and other discontinuities. The toe of the weld is ground until it is smooth and free of visible defects and cracks using a tree-shaped burring bit. Care must be taken to ensure that no grinding marks are left perpendicular to the direction of applied stress. However, during the experimental work the results were inconsistent and the fatigue life was improved only at lower stress range levels. Grinding was eliminated for use in retrofitting fatigue-damaged members in the field.

Peening

Fatigue life of a detail can be improved by peening the toe of the weld with an air hammer until the metal is smooth and no cracks are visible. The plastic deformation of the weld induces compressive residual stresses that prevent the full stress range from being developed under loading. Both crack initiation and growth are favorably influenced. Peening should be done with the dead load in place, if possible, because the application of the minimum stress after peening removes some of the compressive residual stress. Peening is the most economical method for retrofitting a member with very small cracks, those less than 2 in. in length and with estimated depths less than 0.125 in.

Gas Tungsten Arc Remelt

Remelting of the weld toes removes nonmetallic intrusions and fuses discontinuities and cracks. The metal surface adjacent to the weld is first sandblasted to remove all mill scale to prevent undercutting during the remelt passes. A tungsten electrode is moved along the toe at a constant rate sufficient to melt a small volume of the weld and base metal. Proper equipment, skilled operators, and extreme care are necessary to assure adequate penetration. It is recommended that penetration be verified by test plates. Details of the welding equipment and attendant variables are provided in the literature (*7*). The gas tungsten arc remelt procedure is relatively difficult to execute in the field, but it can be used effectively for cracks up to 3 in. long and 0.180 in. in depth.

Applicability to Field Operations

Both the peening and gas tungsten arc remelt procedures have been used under field conditions to retrofit full-scale beams of A36 and 588 steel (*7, 9*). Both operations can be performed overhead, and it is not necessary to interrupt normal traffic. Improvements of the fatigue life by one design category can be expected of either method, regardless of the type of steel. A greater improvement is not possible because neither procedure can prevent the initiation of cracks at discontinuities in the root of the weld.

Often a fatigue crack on an in-service bridge progresses to a size that is too large for retrofitting by either peening or remelting before it is noticed. In such a case the only course of action is the installation of a bolted splice across the crack location designed to transfer the load around the crack. If the crack is in the web, holes should be drilled at the crack tips to prevent further crack growth. It is imperative that the crack tip not extend beyond the perimeter of the hole to prevent further growth. To avoid this situation, it is recommended that the hole be located with its perimeter at the apparent tip of the crack as shown in Figure 2 (*9*). Hole diameters of 0.75 and 1.00 in. were used successfully in the research to limit crack growth in the webs of members (*7, 9*). Even if the fracture is confined to the flange, it may be desirable to slot the web to prevent the crack from propagating into it.

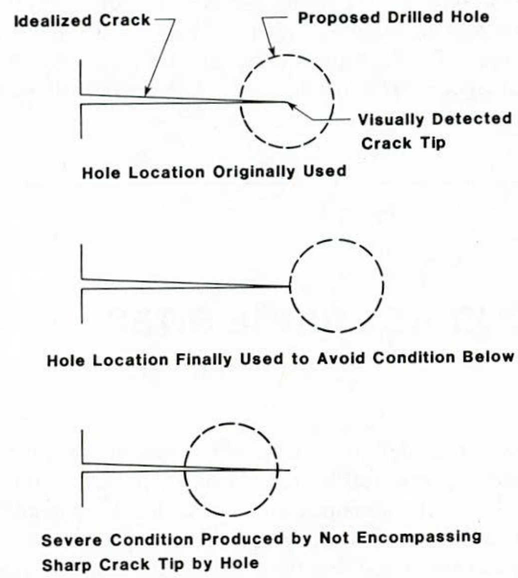

FIGURE 2. Crack Tip and Hole Perimeter Locations. (From Reference 9.)

FATIGUE CONSIDERATIONS IN GENERAL REPAIRS

As mentioned earlier, some of the repair procedures collected during the first phase of the present study contained welded details that showed a disregard for fatigue life. An engineer planning a repair procedure generally faces the same choices as the original designer: the selection of a category of detail having an acceptable fatigue life or the location of a critical detail in a region of low stress range. However, the choice may be limited, as the location of a needed attachment is not often variable in a repair plan. There are cases in which the choice of an alternate detail may provide a higher fatigue strength, as, for example, using a "radiused" transition for a welded flange connection (*11*).

It is obvious that category E details should be avoided. However, this category covers a wide range of welded details, and there are likely to be additional configurations of equally low or lower fatigue strength that are not specifically prohibited in the specification (*8*), Recent research has shown that gusset plates welded to the surface of the flange by transverse welds alone have fatigue resistances less than those of category E (*9*).

Because the fatigue resistance is also directly related to the quality of the weld, it is probably advisable to use bolted connections whenever possible. Questionable welded details were easily modified to bolted connections during the first phase of this study. Changing a nontransitioned welded flange connection to a bolted connection raises it from category E to category B (*11*). The fatigue strength of drilled or reamed holes is generally very high because the surfaces are smooth, with very small initial discontinuities (*7*).

Flame cutting of members in the field should be avoided, as poor quality cutting can result in substantial reductions in strength and very early crack growth (*7*).

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CHAPTER SEVEN

SCOUR AT BRIDGE SITES

Scour can be defined in general terms as the displacement of streambed material by the action of stream or tidal currents. Bridge maintenance and replacement costs are often directly associated with problems resulting from scour, and it has been estimated that the Federal Government averages a $20-million annual expenditure on bridge damage caused by flooding (*1*). This does not include the substantial costs incurred by state and local governments.

The potential for scour damage to occur can be reduced in the bridge design phase. Unfortunately, however, because of economic or practical limitations, scour was not given a great deal of consideration in the design of many bridges in earlier years. Even in many cases where the potential for scour is given consideration during the design phase, changes developing subsequent to construction can lead to problems. Restrictions, for example, in the waterway due to aggradation or the accumulation of debris can increase the capacity of a stream to transport sediment and result in localized scour. Characteristics of the streamflow can also be altered by the activities of man, and this too can lead to unexpected scour problems. Consequently, problems with scour can develop at many bridge sites even though initial precautions may have been taken during design and construction of the structure.

Obviously, the type of streambed material and the geology and foundation conditions at a bridge site are important factors directly related to the potential for scour. In addition, and of major interest to the designer and maintenance forces, is the potential for scour at a site when it is subjected to medium or heavy flooding. Although approaches to bridge design are not within the scope of this report, many of the initial techniques used to counter scour can be applied to the repair of a bridge that has been damaged. Some of these are listed below along with remedial measures that can be used to repair damage or to provide protection.

1. Use dumped rock riprap around substructure units and at fills and embankments to repair scour damage and to provide for future protection.
2. Where applicable, leave in place sheet-pile cofferdams used during construction.
3. Remove any objects from the channel or adjacent areas that will change the flow characteristics of the stream or that will harbor debris.
4. Modify, if necessary, the channel, both upstream and downstream, to reduce scour at the bridge site. A sheet-pile dam downstream, for example, can be used to control degradation of a small stream.
5. Construct a massive toe at the foot of the riprap slope protection to prevent undermining.
6. Where applicable, construct spur dikes or jetties to provide partial diversion of the stream.
7. Repair substructure elements by injecting grout or by using cast-in-place concrete.
8. In some instances it may be desirable to raise or to lengthen a bridge to provide for a larger waterway opening.
9. Where applicable, promote the growth or vegetation at channel/bank locations where its presence may counter potential scour problems.

The foregoing techniques are those that have been used successfully in many repair situations. Each case, however, must be considered with respect to the materials and resources available to the maintenance forces and how these can best be applied to effect a solution.

LITERATURE REVIEW

Considerable information has been developed on the subject of scour and is available in the literature. A great deal of it relates to procedures for analyzing streamflow and estimating the potential for scour. Some procedures for the repair of damage due to scour are available in the literature, but this aspect of the problem has generally not been the main focus of the work that has been reported.

The NCHRP has developed a synthesis of highway practice that is a good basic reference on the subject of scour as well as a bibliography of 58 papers and reports through 1969 (*2*). Formulas and charts that can be used to predict scour depth are presented and discussed. In addition, bridge design considerations and recommendations for construction and maintenance inspection are offered.

A statistical summary of the causes of bridge failures was developed by Chang and was reported in 1973 (*1*). This summary revealed that bridge abutments are damaged most frequently; next are bridge piers and then the superstructure. In 43 percent of the cases reviewed, damage extended to the approach roads. Most of the failures reviewed were attributed to a deficiency in the flow path or to a vigorous change in flow.

In an investigation of scour around bridge piers, Hopkins et al. concluded that three scouring actions were involved: (1) general scour occurring from natural changes in the stream, (2) contraction scour caused by narrowing of the waterway at the bridge site, and (3) local scour caused by flow disturbances introduced by the presence of the bridge pier (*3*).

The most significant contribution to total scour at a bridge pier was attributed to general and contraction scour. It was concluded that local scour, by itself, would not be of sufficient magnitude to explain most failures since it accounted for less than 20 percent of the total scour depth. Therefore, the work by Chang and by Hopkins et al. would seem to substantiate each other since deficiencies in the flow path and vigorous changes in flow would result from natural changes in the stream and narrowing of the channel.

A recent report presents case histories of a large number of bridges and is an excellent documentation of countermeasures for hydraulic problems at bridges (*4*). Each case history includes data on bridge, geomorphic, and flow factors and an evaluation of hydraulic conditions and countermeasures. Of these case studies, problems occurred at piers at 100 sites and at abutments at 80 sites. Performance ratings are given for flow control countermeasures such as spurs, dikes, spur dikes, check dams, and jack fields. Streams are classified into five major types with regard to the lateral stability and behavior that should be taken into account in the design of countermeasures.

A companion report describes the various types of revetment and bed armor, and flow control structures (*5*). Revetment armor is classified as dumped riprap; rock, and wire mattress and gabion; concrete riprap; concrete-grouted riprap; and concrete filled fabric mats and bulkheads. Countermeasures to control flow are also described. Some examples of the former techniques are illustrated later in this report in descriptions of some repairs of scour damage on secondary roads.

Structures for controlling flow can be provided either within or outside a channel, and they act as a countermeasure by controlling the direction, velocity, or depth of flowing water. Figure 1, for example, shows the various types of structures that can be used to help regulate steamflow, control erosion, and protect the bridge crossing. The structures are generally used as shown in Figure 1, but their application must be adapted, of course, to particular situations. Spurs, retards, dikes, and jack fields, for example, may be located either upstream or downstream from the bridge.

A spur is a linear structure, either permeable or impermeable, that is built out into the stream from the bank. Its purpose is to alter the direction of flow, induce deposition, and reduce the flow velocity along the bank. Spurs are sometimes referred to as jetties, groins, dikes, deflections, etc. They may protect the banks of the stream more effectively and at less cost than riprap revetments (*6*). In addition, they help to stabilize a channel, control flow at a bend, and direct flow through the bridge opening. Spurs can be constructed of earth or rock, or can utilize timber, sheet, or steel piles. They can be constructed perpendicular to the stream bank or be inclined upstream or downstream. An upstream-inclined spur is recommended for embankment spurs under the assumption that an area of low water velocity is formed upstream from it, erosion of the upstream face is prevented, and the need for revetment except on the spur nose is eliminated.

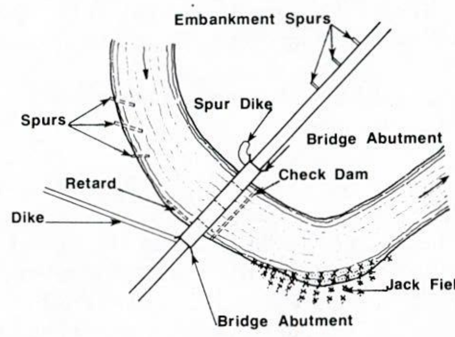

FIGURE 1. Typical Placement of Flow-Control Structures (From Ref. 5)

However, others feel that spurs angled upstream will produce deeper scour holes, and they prefer the 90° inclination as the most economical for bank protection (*6*). The length of spurs should be 50 ft minimum for bank protection on straight reaches, long radius bends, and braided channels. Shorter spurs may be more costly than riprap revetment along the bank. In general, a spacing between spurs of 1.5 to 6.0 times the length of the project upstream spur is recommended.

A retard, as shown in Figure 1, is a linear structure generally built parallel to the stream bank, and it can be either permeable or impermeable. Retards have more of an effect in maintaining an existing stream alignment than do spurs. A particular type of retard is a line of concrete jacks. Each jack consists of three 16-ft concrete beams bolted together at right angles. Several lines of the jacks can be used if necessary. As in the case of spurs, retards tend to reduce flow velocity, induce deposition, or maintain an existing alignment of flow.

Dikes can be used to control over-the-bank flow, and are used mostly on floodplains. They can be used on one or both sides of the bridge opening and their function is similar to that of spur dikes. A spur dike is a straight and outward curving structure that turns upstream or downstream from the approach embankment. Its main purpose is to prevent erosion by eddy action at bridge abutments and piers where flow moving along the upstream side of an approach fill enters the main flow at the bridge. Spur dikes are usually impervious, revetted earth embankments, although rock and concrete rubble have been used. Spur dikes at highway stream crossings are used for three primary purposes:

1. To move the point of high vortex action away from the bridge abutment.
2. To cut off the path of low resistance along a cleared highway right-of-way and force water through vegetation on the floodplain. (If spur dikes are used to cut off the cleared flow path they should be long enough to tie into the vegetation.)
3. To streamline the flow through a bridge opening to reduce velocity concentrations.

A check dam, as shown in Figure 1, is a low dam or weir built across the channel to control the water velocity and degradation. Check dams have been constructed of rock riprap, concrete, sheet pile, rock and wire mattress, gabions, or concrete-filled fabric mat. They have proved effective for reducing degradation in small streams. In alluvial streams, however, an unacceptable amount of scour may occur downstream from a check dam.

References (*4*) and (*5*) are highly recommended for more specific details and numerous case studies of the various types of countermeasures that can be used to control scour. Additional articles concerning scour damage resulting from flooding during 1972 are contained in Ref. (*7*). References (*8*) and (*9*) are further recommended for additional information on the subject of scour.

REPAIR OF SCOUR DAMAGE TO SECONDARY ROAD BRIDGES

The typical repair shown in Figure 2 can be used where pier footings supported on piles have been undermined by scour. In order to protect the exposed piling, a concrete subfooting that jackets the original footing and piling can be used as shown. Ordinarily this repair can be accomplished by constructing a cofferdam around the pier, pumping out the water, and forming for the subfooting. The stone riprap, which is normally placed after the cofferdam has been removed, is used to protect the new foundation by filling in the scoured area around the repaired foundation. Footings that have some scour damage but rest on firm foundation material are often protected from further damage by placing the riprap around the area in a manner similar to that shown in Figure 2. Where the scoured out material is fine-grained, a properly designed gravel filter should be placed prior to installing the riprap. In either case, care should be taken in placing the riprap to avoid damage to the concrete masonry, and checks should be made to assure that the riprap does not overload the existing pile supports. It is advisable, also, to place the riprap in even lifts around the foundation to avoid unbalanced forces against the structure. All debris and construction materials should be removed from the area adjacent to the structure prior to placing the riprap. Whenever possible, riprap should not be placed above the original streambed because this could act as an obstruction to streamflow.

Stone for the riprap can vary in size, depending on the situation. A typical specification for riprap to be used for protective situations such as that previously described would call for stone weighing from 500 to 1,500 lb with at least 50 percent of the stone weighing more than 300 lb. No more than 10 percent of the riprap should weigh less than 150 lb per stone. Rothwell and Boham (*8*) suggest that the following equation can be used for sizing riprap around bridge piers:

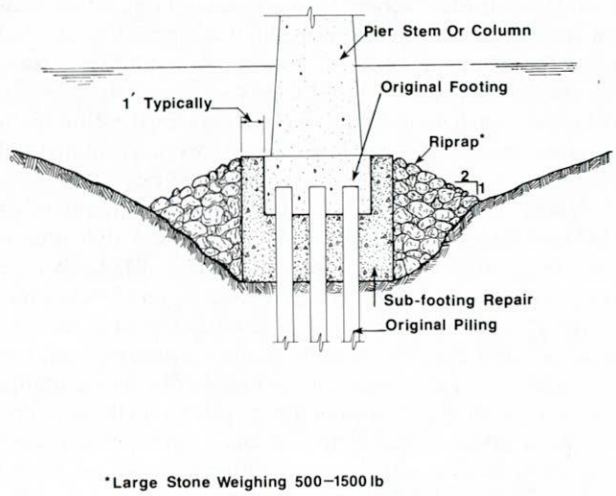

FIGURE 2. Section Showing Repair to Scour Damaged Pile Footing Using Concrete Jacketing and Riprap

$$\frac{d_{50}}{D} = 2.5F^3$$

where

d_{50} = average stone diameter, ft;
D = depth of flow, ft;
F = Froude number of flow = $\frac{V}{\sqrt{gD}}$;
V = average velocity of flow, fps; and
g = acceleration of gravity; 32.2 ft per sec.^{2}

It should be noted that the foregoing equation can be used to determine the minimum size stone required at the location of most severe attack. Smaller size stone can usually be used at the extremities where conditions are less severe.

Pier footings that have been partially undermined by scour can be repaired by another procedure that has been used to successfully repair a number of bridges. This technique involves the use of a flexible nylon form shaped as a tube and filled with structural mortar. The nylon tube configuration is cut to suitable lengths and joined together by using a high tensile strength nylon stitching. The fabric tube is then wrapped around the scoured area of the footing and extended several feet beyond the limits of the problem area.

Prior to placement of the fabric tube, all loose material and debris should be removed from the area of the repair. Once the fabric tube is in position around the foundation area, mortar is pumped into it through suitable openings in the top layer of the material. Pipes to be used for injecting grout into the void under the footing, or foundation, are placed as shown in Figure 3. After the fabric form has been filled, all voids between it and the foundation can be plugged by using a smaller fabric form as suggested in section A-A of Figure 3. Subsequently, the void between the footing and the stream bottom is filled by pumping mortar through the prepositioned injection pipes. Several of the pipes should be provided to allow water to escape from the void during placement of the mortar.

The mortar used in this repair should consist of portland cement, aggregate, and water proportioned and mixed such that the solids in the slurry will remain in suspension without appreciable water gain. Further information on certain aspects of this general technique can be found in papers by Lamberton (*10*) and Davis (*11*).

The intent of this procedure is to perform all of the work underwater, thereby effecting a rapid and low cost repair.

Scour damage similar to that previously discussed can be corrected by other means. Another procedure that could be used is shown in Figure 4. Using this technique, concrete riprap in bags is placed around the damaged area and brought up to a height slightly below the existing footing. This provides a form for tremie concrete that is pumped into the damaged area as indicated in sections A-A and B-B of Figure 4. This repair technique is designed to be completed underwater. A cofferdam could be built, however, and this or another forming procedure used.

The whole streambed at some bridges can, on occasion, be affected by scour. This type of problem can result from changes in the characteristics of the streamflow due to natural or man-made causes. For example, the relief of a downstream constriction in the stream channel can result in increased velocities upstream, particularly during heavy runoff periods. This, in turn, could cause the kind of streambed erosion shown in Figure 5. To prevent further damage to the structure and protect against continued erosion of the steambed, the area can be rebuilt with a crushed stone subbase topped with a heavier stone riprap. The size of the crushed stone riprap would depend on the hydraulic forces to be resisted. However, stone in the 150- to 500-lb range installed in layers totaling approximately 2 ft in depth would be adequate for many situations. (The thickness of a riprap layer is often expressed as two stone diameters.) A combination of this technique and the tube form technique (discussed earlier) for repair of footings could be used where required.

While the undermining of pier foundations lying in the streambed is a major problem, bridge abutments, which are normally positioned at higher elevations, often require repair

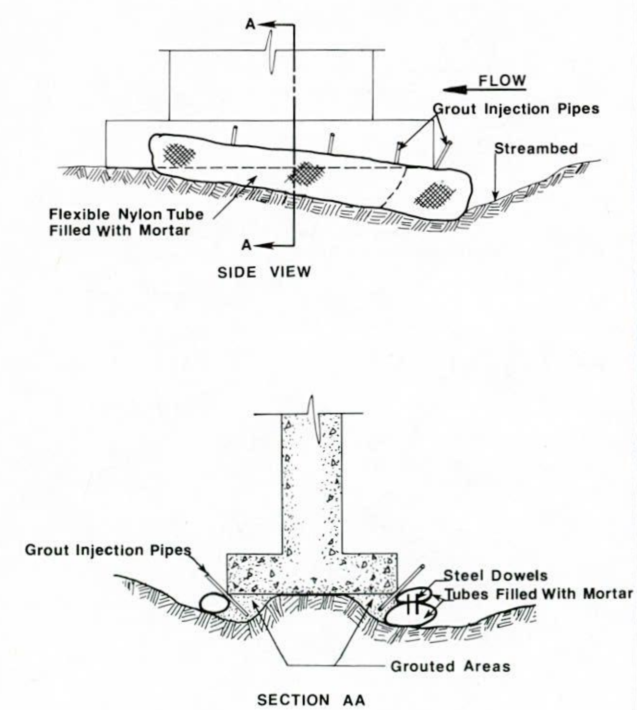

FIGURE 3. Repair of Scour Damage Using A Flexible Nylon Tube Form Filled With Mortar

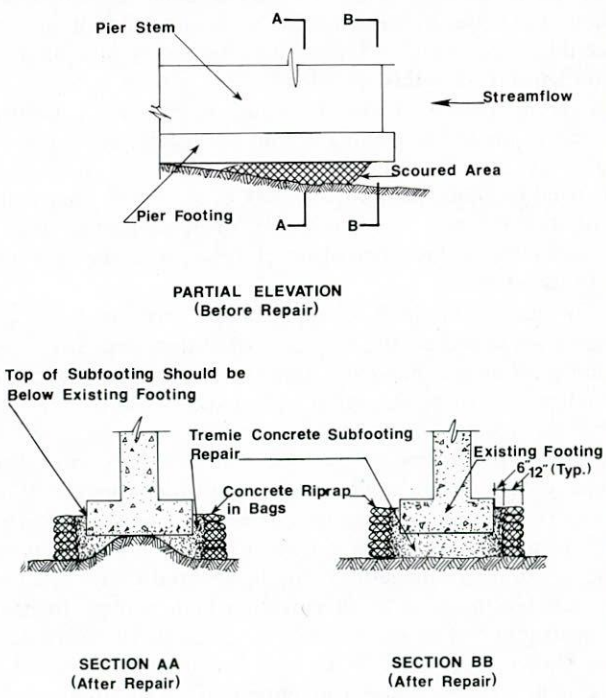

FIGURE 4. Repair of Scour Damage Using Concrete Riprap in Bags and Tremie Concrete Subfooting

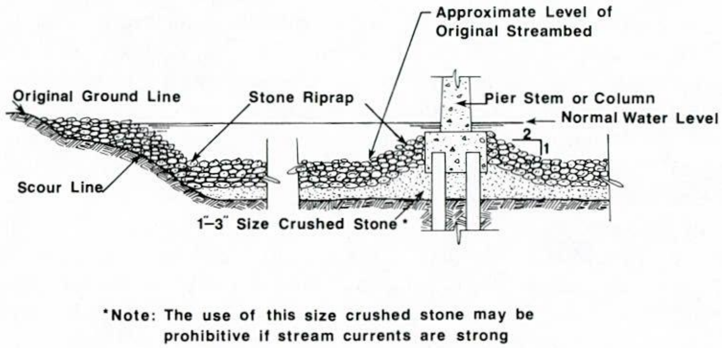

FIGURE 5. Section Showing Rebuilding of Streambed That Has Been Scoured Out Exposing Pier Footings and Piling

to correct damage due to scour. Figure 6 shows a repair that can be used to correct partial undermining of bridge abutments, whether they bear on soil or on piles. A repair technique that has been used successfully for abutments bearing on soil is shown in Figure 6(a). To facilitate tying the original masonry to the new, machine bolts are driven into predrilled holes spaced approximately 2 ft on center. The bolts mechanically tie the new concrete subfooting to the original abutment. If the diameter, length, or spacing of the bolts should need modification to suit a particular situation this, of course, should be determined. A similar subfooting repair could be applied to an abutment supported on piles as shown in Figure 6(b). In this situation the use of bolts to tie the new concrete footing to the old would not normally be used, because the existing piling should adequately serve this purpose. As further protection against future scour, stone riprap is used in either of the two cases as shown in Figure 6(b). Additional future protection, if deemed advisable, could be provided with flow control structures such as the spur dikes discussed earlier.

In many situations, concrete slope protection has been designed to protect the fill slopes of bridges over streams. In some instances, however, this type of slope protection is undermined by scour action and must be repaired to prevent further erosion. In cases where the problem is not severe and there is no undermining of the concrete slope protection, heavy stone riprap can be used as shown in Figure 7. Where undermining of the original concrete slope protection is a problem, the technique shown in Figure 8 has been used successfully. In the latter case, the scoured area is filled with a suitable material to an elevation high enough to enable construction of an extension of the concrete slope protection as shown in section A-A. The remaining void under the original concrete slope protection is filled by dropping concrete through holes punched in the slope as shown. Subsequently, the holes are repaired and the streambed is returned to its original contour. Along the edge of the slope protection, additional correction and protection against scouring can be

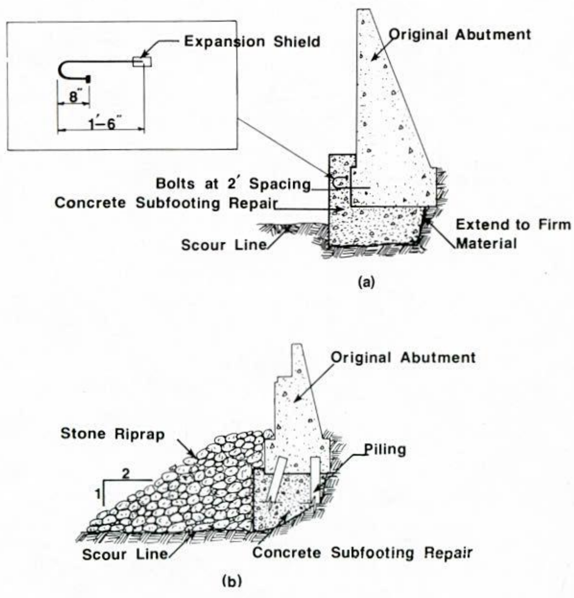

FIGURE 6. Typical Sections of Abutment Repair, (a) For Soil Bearing and (b) For Pile Bearing

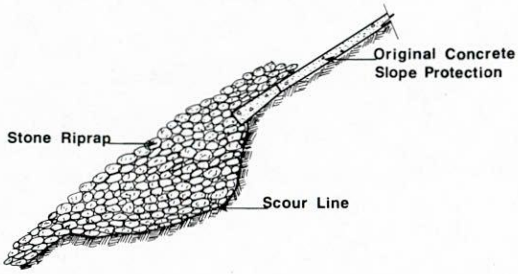

FIGURE 7. Section Showing Repair of Concrete Slope Protection Where Undermining Has Not Occurred

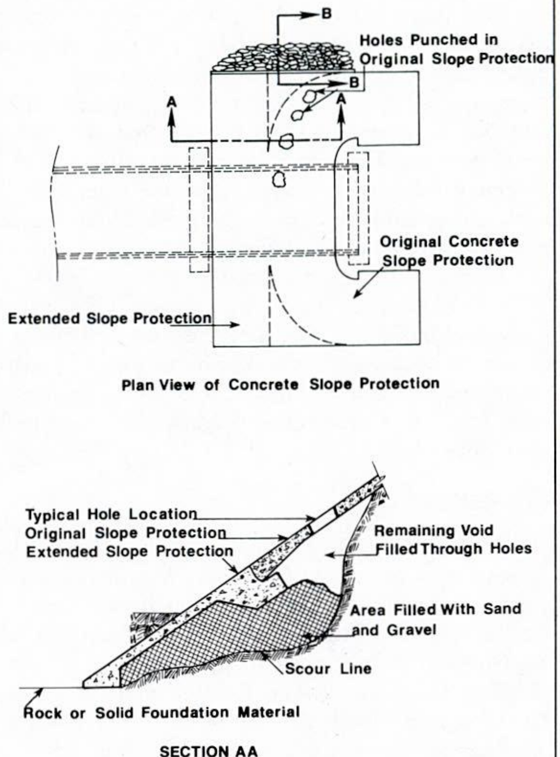

FIGURE 8. Repair of Concrete Slope Protection That Has Been Undermined by Scour

FIGURE 8, SECTION BB. Repair of Concrete Slope Protection Undermined by Scour

provided by constructing the end wall and placing the stone riprap as illustrated in section B-B of Figure 8. Obviously, the extent of the damage and the degree of effort required to repair the site will vary. Therefore, the foregoing examples must be viewed as only example techniques that will likely require some modification in use.

Severe scour can cause movement and, in extreme cases, cause failure of substructure elements. As in the previous examples, the technique used for repair must be adapted to the situation encountered. Also, the question of esthetics must be considered when a repair technique is developed. If appearance is not deemed to be an important factor, some repairs can be undertaken that render a structure serviceable, if not particularly attractive. Figure 9, for example, illustrates a pier that has tilted as a result of scour, causing distress in the superstructure and impending failure. Assuming that the stability of the structure can be assessed, a possible approach to this problem is to drill a number of holes through the deck and into the pier stem. Grout and steel dowels are inserted into the cavity to resist further tipping of the pier until the footing can be repaired (Figure 9(a)). A first-stage pier enlargement is constructed to an elevation above the water line. After this concrete has attained strength, it can be used as a seat to jack the superstructure back approximately to its original grade. Steel shims can then be used to support the superstructure while stage two of the repair is made (Figure 9(b)). To aid in placing the stage-two concrete, additional holes can be drilled through the deck to gain access to the top of the pier. These holes can be closed monolithically with the second-stage concrete placement and the steel shims will be left in place. Prior to the stem repair, the footing must, of course, be repaired. This can be accomplished by one of the techniques discussed earlier and as suggested in Figure 9(a). It should be noted that this repair is intended as an example of repairs that can be, and have been, accomplished on damaged structures. Considerable judgment must be exercised to decide whether to repair or replace in situations such as that illustrated in Figure 9 where stability must be assured and potentially hazardous working conditions must be considered.

As stated earlier, each situation must be treated on an individual basis and only a few examples have been given in the foregoing discussion. There are, of course, many variations to these examples, and many repairs that have been made to structures throughout the country have not been documented.

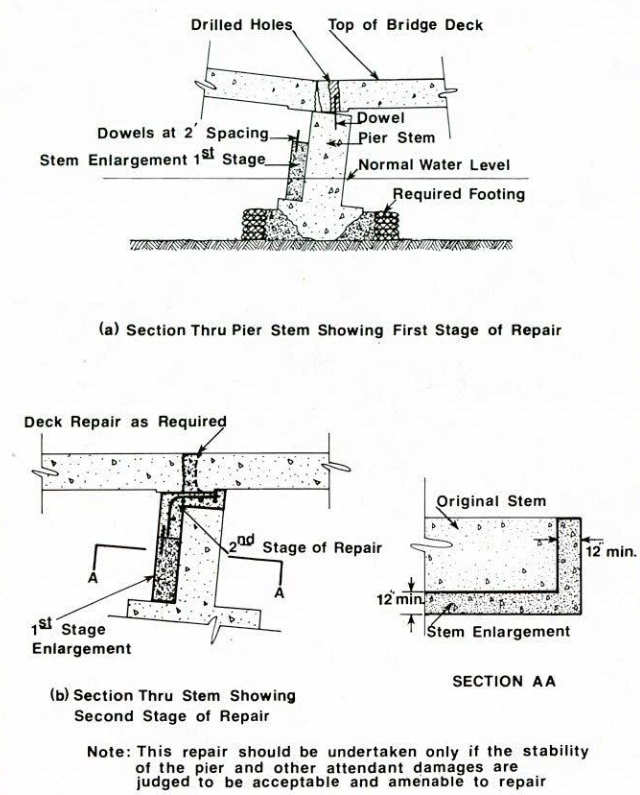

FIGURE 9. Repair of a Pier and Deck That Has Moved or Tilted as a Result of Scour

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CHAPTER EIGHT

SELECTED RETROFIT PROCEDURES DESIGNED TO REDUCE SEISMIC DAMAGE

Although most people consider the West Coast to be the main earthquake region of the United States, records kept by the Coast and Geodetic Survey of the U.S. Department of Commerce give a considerably different picture. Their maps showing locations of major earthquakes indicate that practically every state in the nation has experienced earthquakes and that three of the most severe quakes ever recorded in the United States occurred near New Madrid, Missouri, in 1811 and 1812, and near Charleston, South Carolina, in 1886 (*1*). Recently, on July 27, 1980, an earthquake centered in Kentucky and with an intensity of 5.0 on the Richter scale shook an area of 11 states ranging from Alabama to southern Canada and from Illinois to North Carolina. This earthquake of moderate intensity, which lasted for about 15 sec, knocked ornamentation off Cincinnati's City Hall and shook Detroit's Tiger Stadium while a baseball game was in progress.

Berg (*1*), a structural engineering professor at the University of Michigan, believes no jurisdiction should completely ignore earthquake forces when designing structures where a failure would be catastrophic. This recommendation is given particular credence in view of the wide dispersion of earthquakes in the relatively short history of the United States.

Highway bridges in many parts of the world have been destroyed or seriously damaged by strong motion earthquakes. Bridges in Japan, Mexico, Yugoslavia, Chile, Alaska (1964) and other locations have experienced devastating damage from seismic forces. However, prior to 1971, highway bridges in the contiguous United States had not suffered serious damage from earthquakes. The little damage that had occurred had been limited to minor spalling and cracking of concrete, damaged bearings and grout pads and slight shifting of spans (*2*). The damage had not caused any serious disruptions to traffic, no lives had been threatened, and repairing the damage had been a relatively minor nuisance (*2*).

However, at 6:01 a.m., on February 9, 1971, California's San Fernando Valley experienced an earthquake (Richter scale magnitude of 6.5) which destroyed many structures in just 12 sec. For the first time, bridge engineers in the contiguous United States saw what an earthquake could do to their creations (*3*).

A thorough study by a team of engineers from the California Department of Transportation (Caltrans), which immediately followed the San Fernando Valley earthquake, pointed out a number of deficiencies in the design specifications and practices which had been used up to that time. This investigation pointed out that the force levels required by the existing specifications were not adequate, that the bridge spans were not adequately tied together or secured to the substructure, and that there was an insufficient amount of spiral steel enclosing the vertical reinforcement in pier and abutment concrete columns.

Following this investigation, Caltrans' bridge engineers developed new and more comprehensive earthquake design criteria which have been used in all new bridge designs in California since February 1974. Such factors as proximity of faults, effect of soil conditions on structural response, dynamic behavior of structures, and risk are directly incorporated in the new design specifications. Caltrans points out that very few, if any, other codes consider all of these critical factors. The new code is considered a tremendous step forward and is one of the most comprehensive in existence. This California code was adopted in its entirety by the American Association of State Highway and Transportation Officials (AASHTO) and first appeared in their 1975 *Interim Specifications*. The design criteria, although developed for California, can easily be adapted for other areas of the country. The primary factor that must be evaluated for other areas is the "peak rock acceleration." They suggest that a federal agency such as the U.S. Coast and Geodetic Survey office could develop a map of zones of anticipated rock acceleration values (*4*).

The new Caltrans and AASHTO design practice is to apply an equivalent static force to the conventional structure, and for large, complex, or tall structures a response spectrum technique is used. Reference (*4*) includes a description of the development of the various factors in Caltran's design criteria and a brief discussion on how these criteria could be used in other states with some additional developmental work.

In addition to the development of a new set of design specifications for new bridges, major efforts were directed by Caltrans and the Federal Highway Administration toward the development of retrofit procedures for existing bridges to increase their resistance to damge by earthquake forces. The retrofit designs were directed at strengthening the following major types of structural deficiencies revealed by the San Fernando earthquake:

- Segments of the structure not being adequately connected.
- Columns having too few and improperly detailed ties and spirals.
- Lap splices of main column reinforcement being too short and the surrounding concrete being inadequately confined.
- Footing and bent cap concrete being inadequately reinforced.
- Concrete shear keys being too few or insufficiently reinforced.
- Design force levels being too low considering the seismicity of the location.

Degenkolk (*2*), a design engineer with Caltrans, observes that few existing bridges with these deficiencies can economically be brought up to the same level of seismic resistance as a new bridge. There are, however, several deficiencies that can be greatly improved by retrofitting procedures:

> The lack of adequate connections between segments of a bridge is one deficiency that is quite prevalent and the most readily improved by retrofitting. Fortunately, tying segments of a bridge together is the least expensive of the various deficiencies to correct, and when that is done, it partially alleviates the seriousness of the other deficiencies. Bridges with single column bents are particularly vulnerable when segments are not connected.
>
> Inadequately reinforced columns are the second greatest seismic problem of many older bridges. Many reinforced concrete columns have too few and improperly detailed ties and spirals, and main column reinforcement is frequently spliced with short laps in inadequately confined concrete. This is particularly critical in structures with single column bents.

Degenkolk notes further that "it is not practical to design bridges that will economically serve normal transportation needs but not be damaged to some extent if subjected to severe seismic shaking. The aim is to make structures seismically resistant to the extent that they may sustain damage but not collapse completely." It is also desirable that a bridge should be capable of carrying at least a minimum amount of emergency traffic, even though it is damaged (*2*).

A report prepared for the FHWA includes the following recommendations in making the decision for retrofitting a bridge for increased resistance to earthquake damage (*5*):

> In determining whether a given bridge warrants retrofitting, these three steps (as a minimum) should be considered:
> (a) Will the bridge suffer a critical failure (i.e., so extensive that the bridge could not remain in even emergency use) if subjected to the probable earthquake loading for that bridge site? If the analysis provides a negative answer to the question, one need go no further. If the answer is affirmative, the second step is—
> (b) Determine the level of importance of the bridge to the given locality by considering the type of highway, traffic volume, and accessibility of other crossings. If it is determined that the bridge is unimportant to the locality, it may be decided that retrofitting is not feasible, even though the answer to (a) was affirmative. If, However, it is decided the bridge is important to the area, the third step is—
> (c) Determine the type or types of retrofit measure(s) to employ. This decision is based on the following considerations.
> (1) mode of failure;
> (2) how will the chosen retrofit measure(s) influence the performance of other parts of the bridge under seismic as well as normal service loadings;
> (3) a comparison of expected interference with traffic flow on and under the bridge for different retrofit measures;
> (4) a comparison of expected costs of fabrication and installation of different retrofit measures.

TYPICAL RETROFIT PROCEDURES

Following are ten typical retrofit procedures that have been developed to increase the capability of a highway bridge to successfully withstand earthquake forces. These procedures will improve many of the deficiencies found in existing structures, and the references listed describe and illustrate a number of other successfully used retrofit techniques. Reference (*6*) includes designs for several of the retrofit techniques.

List of Retrofit Procedures and Figures

TITLE:

Steel Girder Bridge Expansion Bearing Modified to Prevent Uplift

DESCRIPTION:

Vertical motion restrainers prevent the superstructure from lifting off the support bearings in the event of seismic loading with a strong vertical component.

RETROFIT PROCEDURE 1 (Fig. 1)

This retrofit measure obviously can be employed in a similar manner for a fixed bearing. An additional technique for fixed bearings is presented in Ref. (5), p. 117. Slotted plates (slots are to allow for normal service motion) are welded to the top of the bottom flange of the girder resting on the bearing, and high strength steel, doubly threaded rods are passed through the slots and connected to the coupling nuts. Two conventional hexagonal nuts are then placed on each bolt leaving 1/16 in. clearance between the bottom nut and the slotted plate.

The replacement of the existing anchor bolt nuts with rod-coupling hexagonal nuts is required.

REFERENCE: (5), pp. 103 and 104.

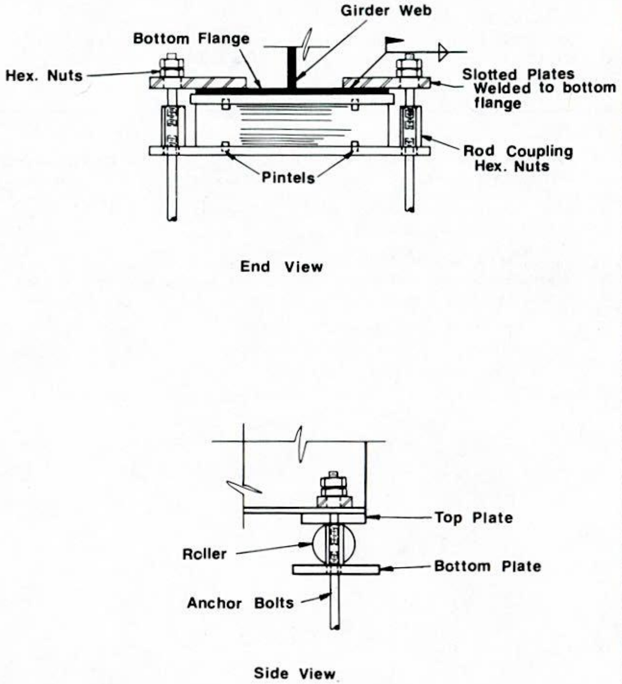

FIGURE 1. Steel Girder Vertical Restraint at an Expansion Bearing

TITLE:

Steel Girder Bridge Vertical Restrainer to Prevent Uplift at a Pier

DESCRIPTION:

Vertical motion restrainers prevent the superstructure from lifting off the support bearings in the event of seismic loading with a strong vertical component.

RETROFIT PROCEDURE 2 (Fig. 2)

This procedure is applicable where there is sufficient open space under the cap beam to accommodate the restrainer details shown or where provision can be made for the beam to pierce a concrete wall that is sometimes used between pier columns.

This vertical restrainer technique is stronger than that described in the preceding Retrofit Procedure 1. This measure will limit the vertical separation that can occur at the support bearings and eliminate bearing instability and thus loss of superstructure support. Relative horizontal motion at the bearings is not restricted with this measure.

REFERENCE: (6), Vol. 2, pp. 19, 20, 21.

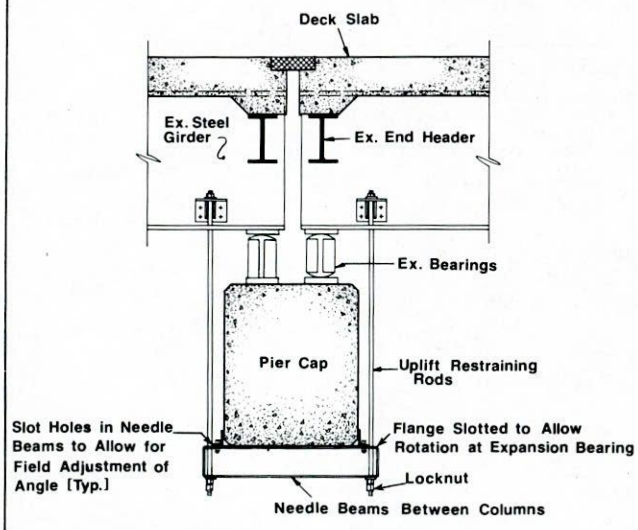

FIGURE 2. Steel Girder Vertical Restraint at a Pier

TITLE:

Longitudinal Motion Restrainer at an Abutment

DESCRIPTION:

Restricts the relative lognitudinal motion between the superstructure and the abutment at the expansion bearing.

RETROFIT PROCEDURE 3 (Fig. 3)

This retrofit measure will eliminate bearing instability and prevent the superstructure from falling off the abutment because of excessive motion. A certain amount of free travel from thermal expansion effects and allowable earthquake motion is permitted to take place before the stopper exerts resistance to motion.

The anchor bolts must be embedded in the abutment in such a manner as to develop the calculated design force. The concrete under the beam seats must be sound and in good condition.

REFERENCE: (6), pp. 11-18.

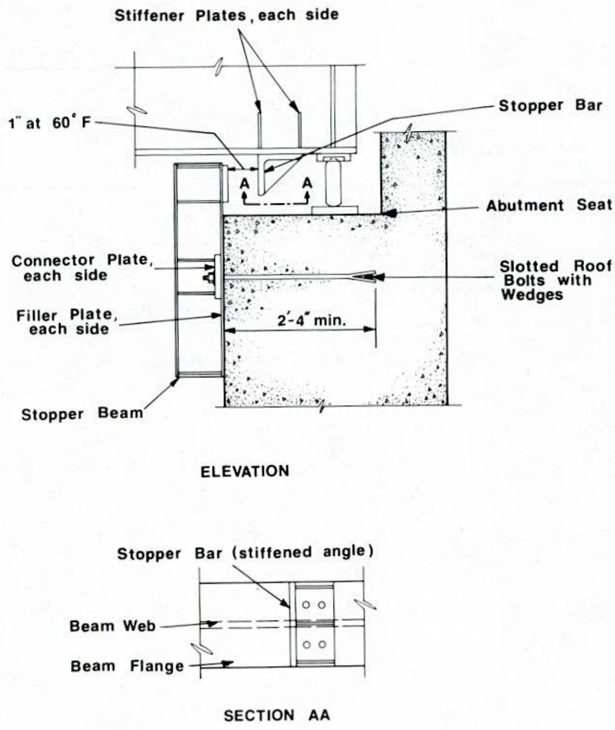

FIGURE 3. Longitudinal Motion Restrainer at an Abutment

TITLE:

Longitudinal Motion Restrainer at an Abutment or Pier Bent

DESCRIPTION:

Restricts the relative longitudinal motion between the superstructure and the abutment at the expansion bearing.

RETROFIT PROCEDURE 4 (Fig. 4)

This is an alternative procedure to Retrofit Procedure 3. The "stopper," a beam or beam grillage, is attached by anchor bolts drilled and grouted into the support. The anchor bolts must be embedded in the abutment in such a manner as to develop the calculated design force. The concrete under the beam seats must be sound and in good condition.

REFERENCE: (5), pp. 108 and 124.

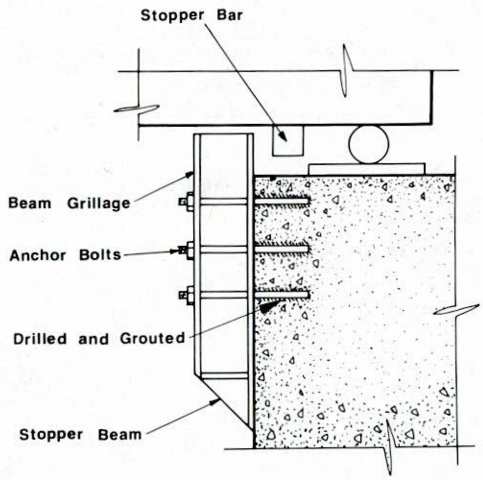

FIGURE 4. Longitudinal Motion Restrainer at an Abutment or Pier Bent

TITLE:

Longitudinal Motion Restrainer at a Pier

DESCRIPTION:

Restricts relative longitudinal separation between superstructure component at hinges and bearings.

RETROFIT PROCEDURE 5 (Fig. 5)

This type of restrainer consists of high strength steel bolts connecting two spans with provision for normal service expansion and rotation at fixed or expansion bearings. The size and number of restrainer units required can be determined once the magnitude of the longitudinal earthquake loading has been determined from analysis. Typically, for steel girder bridges, the number of girders for different spans is the same and the girders lie in straight lines. Installation involves cutting or burning a hole through the end stiffeners of opposing girders, welding or bolting a channel (the same width as the stiffener and with a preformed hole) to the stiffeners (and possibly the web), and, finally, installing the bolts and associated elastic and steel washers and nuts as shown in Figure 5. The size of the gap should be such that normal motion is allowed. Also, the holes in the channels and the stiffeners should be slotted in the vertical direction to allow normal rotation. Although it will not always be necessary for all the girders to be connected by restrainer units, it would be advisable to employ two units per restrained girder, one on each side of the web, and to locate the pairs of restrainer units symmetrically to allow for a symmetric transfer of the load. An alternative to attaching the restrainer units to the end stiffeners of the girders would be to install them between the end diaphragms of opposing girders. This may also require stiffening of the end diaphragms.

NOTE: Installation of this retrofit measure in an existing bridge would cause minimal, if any, traffic interference, but the accessibility of the support(s) must be kept in mind when considering the feasibility of these units. The hinges and/or bearings must be accessible. There must be adequate strength in the end stiffeners or diaphragms to which the restraining rods are attached.

REFERENCE: (5), pp. 98-99, Figure 6.1.

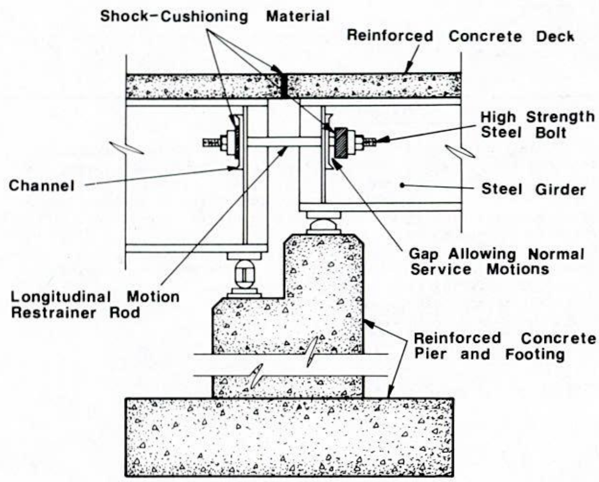

FIGURE 5. Longitudinal Motion Restrainer at a Pier

TITLE:

Steel Girder Hinge Expansion Joint Longitudinal Restrainer

DESCRIPTION:

Restricts the relative longitudinal motion across the expansion joint.

RETROFIT PROCEDURE 6 (Fig. 6)

This retrofit concept eliminates excessive separation displacement across the hinge, and hinge failures created from this effect are eliminated. A certain amount of free thermal expansion and contraction are permitted at the hinge before the restrainer unit resists the earthquake-induced separation.

REFERENCE: (6), Vol. 2, pp. 24-27.

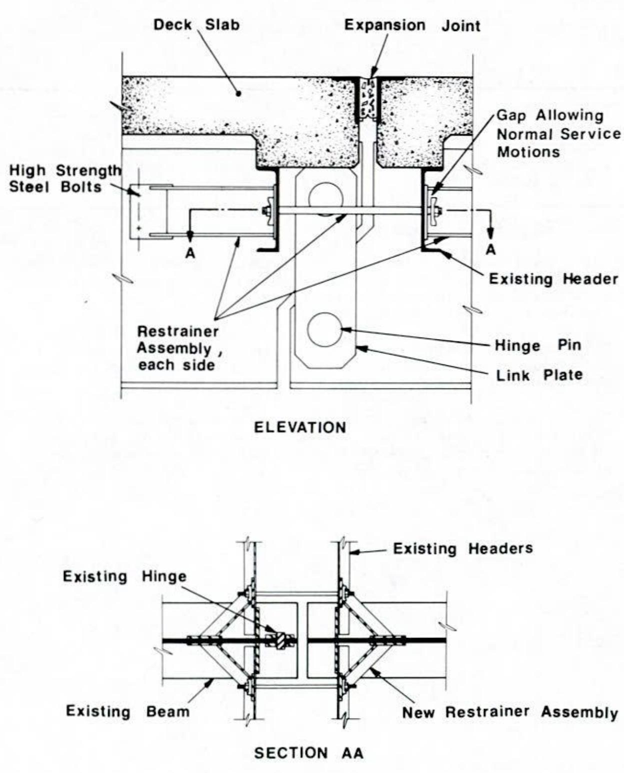

FIGURE 6. Steel Girder Bridge Hinge Expansion Joint Longitudinal Restrainer

TITLE:

Concrete Box Girder Longitudinal Hinge Restrainer

DESCRIPTION:

Restricts relative longitudinal motion between superstructure components at an internal hinge.

RETROFIT PROCEDURE 7 (Fig. 7)

The concrete bolsters are generally required to spread out the concentrated forces of the restrainers so that they do not destroy the hinge diaphragms. One 7-cable (428 kips) unit placed in each exterior cell at each hinge is generally considered to be a minimum requirement for providing adequate resistance to transverse bending of the entire superstructure.

Access to the cells is made through the soffit whenever possible to avoid interfering with traffic on the bridge. If access through the soffit is not possible or desirable because of traffic under the bridge, work is done through the deck openings.

REFERENCES: (2) pp. 8 and 9; (4) p. 24.

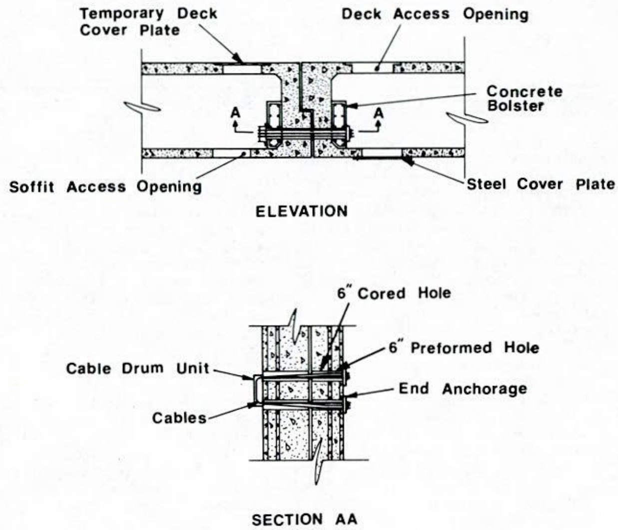

FIGURE 7. Concrete Box Girder Longitudinal Hinge Restrainer

TITLE:

Widening of a Bearing Area at an Abutment or Pier Bent

DESCRIPTION:

Enlarges the bearing area to provide extra bearing width at a support in the event that seismic motion causes the span to fall off the bearing at that support.

RETROFIT PROCEDURE 8 (Fig. 8)

Holes are drilled into the existing support to a depth necessary to develop the strength required in the design. The reinforcement is then inserted and grouted into the holes, forms are put in place, and the new concrete poured. See Figure 8.

The extra bearing width needed for a bearing-widening scheme to be successful is known once the longitudinal displacements at the support have been determined from the analysis.

The abutment or bent must be of sound concrete to receive the grouted reinforcement of the bracket. NOTE: The feasibility of implementing this retrofit measure depends on accessibility of the bearing seats at the abutments and pier caps.

REFERENCE: (5), pp. 108 and 124.

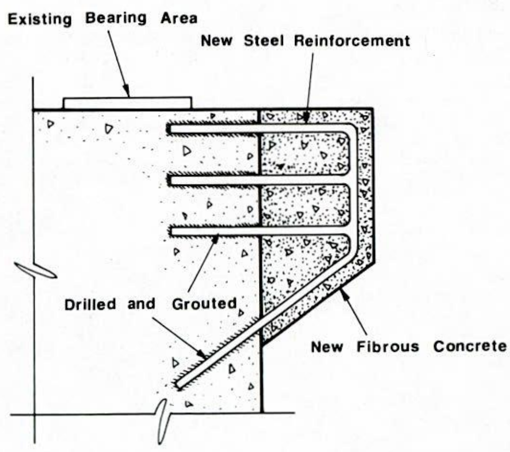

FIGURE 8. Widening the Bearing Area at an Abutment or Pier Bent

TITLE:

Strengthening a Square or Rectangular Concrete Column

DESCRIPTION:

Increases the resistance of an existing concrete column to seismic forces.

RETROFIT PROCEDURE 9 (Fig. 9)

An approach to increasing the seismic resistance of an existing reinforced concrete column is shown in Figure 9. Longitudinal reinforcement is added to the exterior of the column and inserted into holes drilled into the bent cap and footing. The holes are then filled with grout. The depth of the penetration of the added bars into the cap and footing must be great enough for the strength of the connection to be developed. Ties are added along the length of the column. The additional reinforcement is then bonded to the column with gunite. It may also be feasible to add dowels (using a drilling and grouting procedure) from the top of the column into the bent cap and the base of the column into the footing.

NOTE: Implementation of this retrofit measure would cause interference with traffic on the bridge as well as with traffic under the bridge if the bridge spanned a highway. If the bridge spanned a body of water, use of this measure would probably not be feasible.

REFERENCE: (5), pp. 106 and 121.

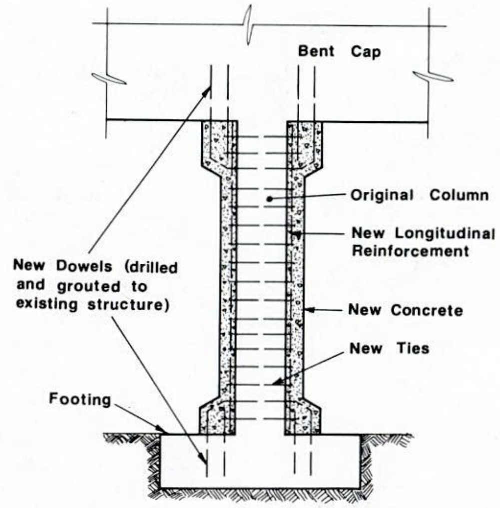

FIGURE 9. Strengthening a Square or Rectangular Reinforced Concrete Column

TITLE:

Strengthening a Circular Concrete Column

DESCRIPTION:

Increases the seismic resistance of an existing concrete column.

RETROFIT PROCEDURE 10 (Figs. 10 & 11)

The second greatest weakness of bridges of pre-1971 (San Fernando earthquake) design was the lack of lateral steel in concrete columns to provide adequate confinement of the core concrete. A lack of adequate connections between segments of a bridge was considered the most serious deficiency.

Figures 10 and 11 show three methods of retrofitting circular columns to improve their capability to resist seismic forces. Figure 10 illustrates two procedures. The first consists of wrapping a column with tensioned prestressing wire and applying a protective coat of shotcrete. A second method consists of pretensioning a series of No. 4 hoops with an especially designed turnbuckle device to pull the two ends of the hoop together. Figure 11 illustrates a method that consists of welding a steel shell around an existing column and filling the space between the shell and the column with grout. "Weathered" steel can be used for appearance, if desired, or ordinary steel can be used and painted.

NOTE: Implementation of this retrofit measure would cause interference with traffic on the bridge as well as with traffic under the bridge if the bridge spanned a highway. If the bridge spanned a body of water, use of this measure would probably not be feasible.

REFERENCE: (2), pp. 14-15.

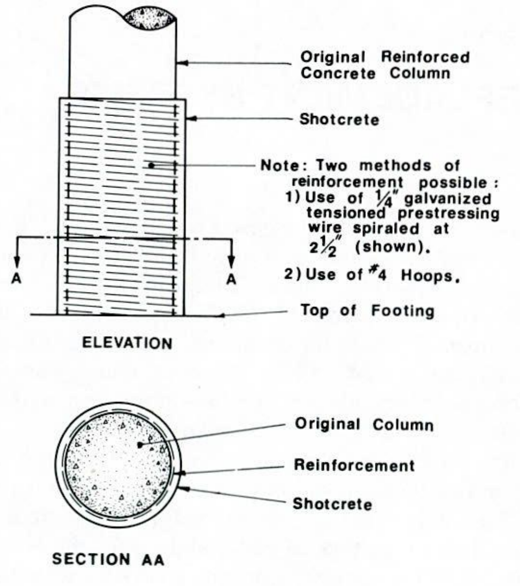

FIGURE 10. Column Retrofit Using Additional Reinforcing Steel

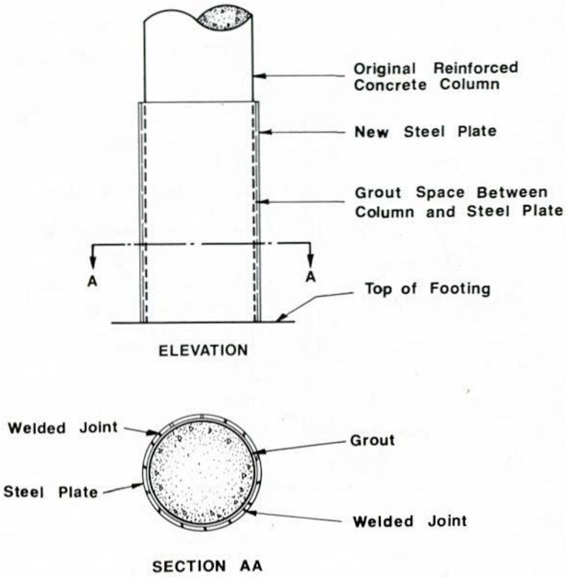

FIGURE 11. Column Retrofit Using Steel Plate Encasement

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CHAPTER NINE

REPLACEMENT SYSTEMS

Chapter Three of the Phase I report (*NCHRP Report 222*) contains information on a majority of the typical bridge replacement systems available for use in the United States. Each of the systems is briefly described and illustrated; prominent features are discussed; and one or more cases in which each system has been used are noted. The names and addresses of manufacturers who supply the materials used in each system are also cited in the Phase I report, except in a few cases where such information was not available. Reference documentation for each system is also provided.

The replacement systems in the Phase I report are grouped according to the type of material used for the primary components of the superstructure or otherwise were placed in a miscellaneous group. The group designations were C, S, T, and M indicating concrete, steel, timber, or miscellaneous, respectively.

In this report, one new replacement system has been added to the concrete group, System C-9 (Short-Span Segmental Construction). Three new replacement systems have been added to the miscellaneous group; namely, System M-7 (Precast Concrete Arch Bridge), System M-8 (Single and Multiple Culverts of Aluminum, Concrete, and Steel), and System M-9 (Field Connected Beams). System M-6 (Long-Span, Corrugated Metal, Buried Conduits) of the Phase I report has been modified to include an illustrative figure omitted from the Phase I report.

The four additional replacement systems presented in this report (in the appendix) are described and illustrated in the same manner as was used in the Phase I report. As in the Phase I report, the designs and dimensions are intended for illustrative purposes only. Agencies or individuals cited in the references should be contacted for specific details. A qualified engineer should always be consulted and be responsible for the design and construction of a specific bridge.

COMMENTARY ON REPLACEMENT SYSTEMS

The four new replacement systems and the illustration of System M-6 are presented here not because they deserve special emphasis, but merely because they were not included in the Phase I report. System M-7 is included in the Phase II report because it represents the concrete alternative to System M-6. System M-8 was omitted from the Phase I manual because, by definition, the report was designed to deal with bridges and with spans of 10 ft or greater length. However, because culverts quite often represent the most economical way of replacing a very short span bridge, it was decided that the manual would be more complete if culverts were included in the Phase II report. System M-9 is included because it illustrates a viable way to use standardized bridge beams to construct spans longer than the standard length of the members. Finally, System C-9 was omitted from the Phase I manual because it was generally felt that segmental construction was best suited for medium- to long-span bridges and would, therefore, seldom be encountered by the typical county engineer. Recent reports, however, suggest that segmental construction may be economical for use on short spans, particularly if many bridges or spans can be constructed using the same form; and, therefore, it is reported here as another viable concrete replacement system.

The replacement systems included in the appendix of this report are considered to be additions, and therefore viable alternatives, to the ones presented in the Phase I manual. As mentioned in that report, many factors must be considered when making a decision as to which system should be used for a particular application. Significant factors include (1) cost and availability of one material relative to those of another, (2) the cost and availability of forms and equipment for fabricating and handling one type of bridge component relative to those for another, (3) the qualifications and experience of the available labor force, and (4) the characteristics desired in the replacement system. In general, a replacement system using a member that in the case of timber or steel is a stock item, or in the case of concrete can be cast in forms that are readily available, will almost always have a lower first cost than a system that requires nonstandard members or the purchase of special forms and equipment. The replacement systems presented here and in the Phase I manual are intended to provide the typical county engineer with descriptive and illustrative information on practical bridge replacement systems. The information should be useful in making preliminary choices between alternatives in a given situation.

SUMMARIES FOR REPLACEMENT SYSTEMS

Innovative Concepts for the Use of Prefabricated Bridges and Bridge Components

The vast majority of the innovative concepts for the use of prefabricated bridges and bridge components was reviewed in Phase I of NCHRP Project 12-20, and the practical systems were subsequently presented in Chapter Three of *NCHRP Report 222*. The concepts include a variety of prefabricated concrete, steel, and timber components, and the reader may refer to that manual for the details of the systems incorporating these components. Additionally, four other replacement systems using prefabricated components are included in the appendix to this report. The principal goals in using these prefabricated components are to achieve economy through the repeated use of forms and to reduce on-site construction time and labor by concentrating the construction effort in the factory rather than in the field.

Other practical innovative concepts will likely surface in the near future, as the literature notes a variety of systems that are being examined on paper or as prototype structures as yet not being marketed. Hanson (*1*) and GangaRao (*2*) have presented concepts for the use of prefabricated substructure components (see Fig. 1), but such components have seen only limited use because typically there are so many differences between bridge sites, such as soil bearing characteristics, the location of bedrock, and depth at which acceptable bearing can be obtained, that it is almost impossible to standardize these units. Successful results have often been obtained by prefabricating a part but not all of the substructure. For example, a prefabricated abutment was used in a bridge in Virginia by first constructing a level surface from which to work (the footing was site-cast concrete) and then placing the prefabricated abutment segments on top of the footing (see Fig. 2). Portland Cement mortar was placed in the keyways between the segments and between the site-cast footing and the bottom of the segments, and two posttensioning strands were used to tie the segments together. Almost all concepts for using prefabricated concrete units in the substructure require the use of either portland cement grout, mortar, concrete, or posttensioning to tie the elements together. Whereas prefabricated components are used routinely in the construction of bridge superstructures, their use in substructures is just beginning.

The use of press-lam is an innovative concept for timber bridges (*3*). In the production of press-lam, thin sheets from rotary peeled logs are dried, lapped, and glued together to produce lumber as large as the production equipment will allow. Not only can timber members of almost any size be produced, but defects in the logs can be scattered so as to allow for greater stress in design and for the use of inferior timbers. The world's first press-lam bridge, which was fabricated by the Forest Products Laboratory and is shown under construction in Fig. 3, was erected in Virginia in 1977 (*4*). The bridge was constructed to demonstrate the suitability of press-lam for heavy construction, and after 4 years the bridge is performing satisfactorily.

With plastic increasingly replacing other materials in almost every area, it is reasonable to expect that plastic components will be used in bridges. A glass-fiber reinforced plastic bridge has been under study in Virginia since 1972 (*5*), and a plastic pedestrian bridge will be installed at a rest area along Interstate 66 in the near future. Plastic components lend themselves to prefabrication because of their high strength-to-weight characteristics. A scale model of a plastic bridge is shown in Figure 4.

There has been considerable innovation in the use of prefabricated bridges and bridge components over the past decade. Notable examples are precast components such as the tee-beam, the permanent bridge deck form, and the parapet; segmental construction; prefabricated steel bridges; and glulam. It is reasonable to expect that innovations will continue as the bridge engineer is forced to deal more with the rehabilitation and replacement of bridges and less with new constructions.

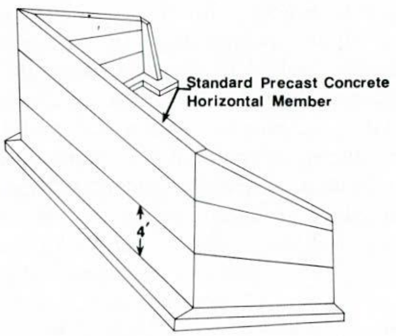

FIGURE 1–A. Abutment With Precast Concrete Horizontal Members (After Hanson Ref.1)

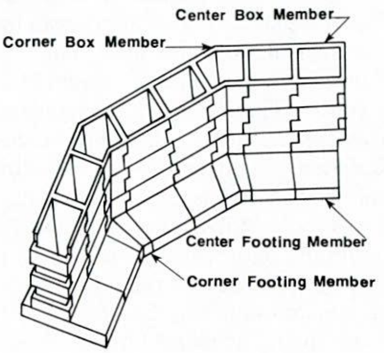

FIGURE 1–B Abutment With Precast Concrete Box Members (After GangaRao Ref. 2)

Short-Span Segmental Construction (See System C-9 in Appendix)

Segmental construction is defined as the construction of a bridge with precast or site-cast segments that are similar or identical in geometry and have cold joints perpendicular to the direction of traffic. Segmental construction has been popular in Europe for two decades, but began to gain popularity in the United States only in the late seventies (*6*). Originally intended for bridges having multiple spans of 150 to 400 ft, segmental construction is now believed to be economical for short-spans (*7*). Economy requires that all the segments be cast in the same forms and, if precast, that match casting generally be required.

For short-span construction the segments could be erected on falsework, and posttensioning requirements could be simplified as compared to those for long-span cantilever construction. It has been suggested that the system can be competitive if the same forms can be used to cast segments totaling a minium of 250 ft in length (*7*). Segmental construction could be a boon in short-span bridge replacement where it can be used on a number of bridges in the same vicinity. Its fundamental disadvantages concern the requirement of engineering expertise and investment in forms and equipment.

Current interest in segmental construction is evidenced by the FHWA's recent award of a contract to T. Y. Lin International to investigate the feasibility of standardizing segmental, prestressed, concrete box girder bridges.

Field Connected Beams (See System M-9 in Appendix)

Quiet often bridge spans must be longer than the longest prefabricated beams that can be transported to the bridge site. Where this is so, wide flange steel beams or plate girders have been the preferred beam because short members can be easily connected end to end by field welding or using bolted splice plates. Recently, however, a prototype, two-span, precast prestressed I-beam bridge was constructed in Illinois in which three I-beam segments were connected end to end using a site-cast concrete connection and posttensioning (*8*). The technique is likely to have some application in the immediate future, because it allows the precast I-beams to compete with structural steel in subject application.

Single and Multiple Culverts of Aluminum, Concrete, and Steel (See System M-8 in Appendix)

Because they are prefabricated, culverts can be installed in a short period of time and quite often provide an economical alternative to bridge replacement. They are made of several materials and are available in a variety of sizes and shapes.

For small drainage areas, pipe sections having the desired cross section can be laid end to end and connected with bolted bands, hugger connectors, sleeve joints, or tongue and groove joints. For large drainage areas, pipes of large diameter can be used or several pipes can be laid side by side. When metal culverts having a diameter greater than about 10

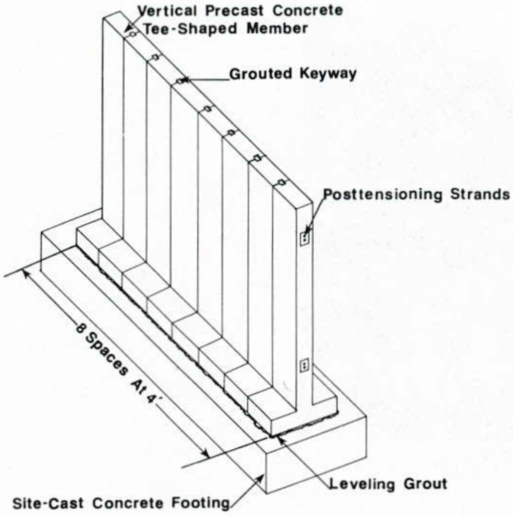

FIGURE 2. Abutment With Precast Concrete Vertical Tee-Shaped Members

ft are required, they usually are assembled at the site using structural plate sections bolted together.

In recent years, precast concrete culverts have become popular (*9*). A standard, inverted, U-shaped, precast concrete section is available that can be erected on a site-cast footing or slab floor. A section can accommodate spans up to 16 ft, and large drainage areas can be served by placing several sections next to each other. Precast box sections placed end to end and connected with a tongue and groove joint are also popular (*10*).

Site plans for culverts require less detail than those for bridges, less time is required for installation, and there is no deck to deteriorate. The fundamental disadvantages of culverts are that quite often they do not provide adequate drainage and they are susceptible to blockage by floating debris. Also, they are not suitable for use in some acidic environments. To prevent corrosion, steel culverts can be galvanized or coated with a bituminous material and concrete culverts can be coated with an approved sealer.

Modular Construction

Modular construction implies the use of one or more standard components of the type employed in many of the systems presented in the Phase I manual. For example, all of the concrete systems can be considered as modular because, within certain limits, the same forms can be used to produce components of a given shape, such as a tee-beam, that can be used for a variety of roadway widths and span lengths. Typically, it is economical to adjust a form to accommodate major changes in span length rather than to use extra concrete and steel. The Standard AASHTO I-beam was a step in the direction of modular construction in that within certain limits of

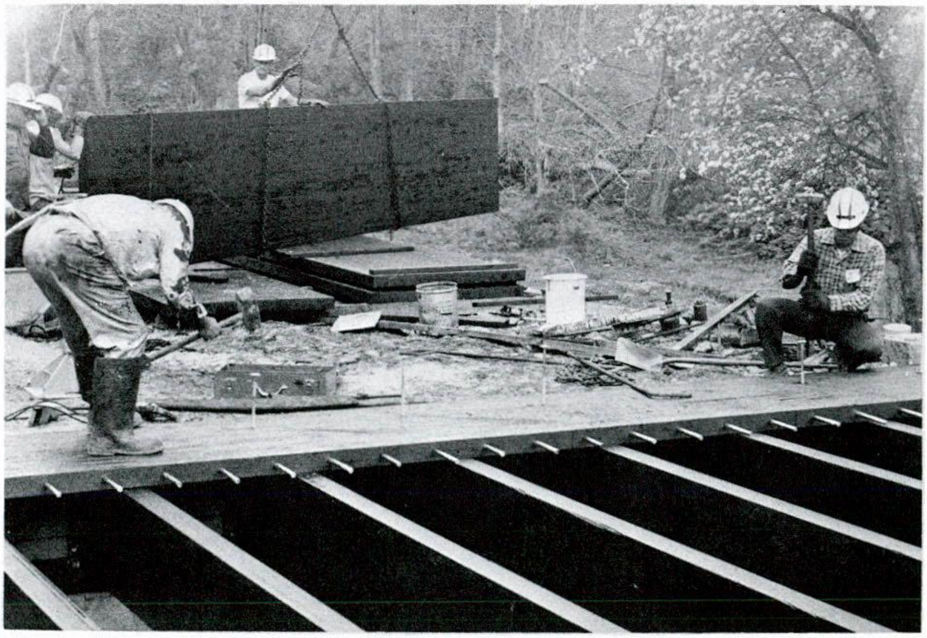

FIGURE 3. Installation of Press-lam Timber Deck Panels

FIGURE 4. Scale Model of Glass-fiber Reinforced Plastic Bridge

span length, the same type beams are used. The length or spacing of the beams can be adjusted to accommodate site conditions.

In the case of steel systems, the standard rolled beams are an attempt at modular construction, but the best example is the temporary bridge (System S-2 in *NCHRP Report 222*). With the temporary bridge, an appropriate number of standard size modules are connected to provide the required load-carrying capacity. To increase or decrease the capacity, modules are added or eliminated.

For reasons of economy, it will not be practical to use many totally modular bridges such as the temporary bridge. Quite often these bridges contain more material than needed and esthetics would be a problem. It is likely, however, that there will be a trend toward greater use of modular components within a given structure as has been the case in the use of precast concrete components over the past decade.

No examples of modular construction other than those reported in the Phase I manual or incorporated in the appendix of this report have been noted by the authors.

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7. "Precast Segmental Box Girder Bridge Manual." Prestressed Concrete Institute, Chicago, Ill., and Post-Tensioning Institute, Phoenix, Ariz. (1978).
8. FADL, A. E., GAMBLE, W. L., MOHRAZ, B., "Tests of a Precast Post-Tensioned Composite Bridge Girder Having Two Spans of 124 Feet." University of Illinois, Urbana, Ill. (Apr. 1, 1977).
9. "Standard Specifications for Precast Reinforced Concrete Box Sections for Culverts, Storm Drains and Sewers." ASTM C789-79 and C850-76, Philadelphia, Pa.

APPENDIX

REPLACEMENT SYSTEMS

LIST OF REPLACEMENT SYSTEMS AND FIGURES

REFERENCES

Note: References *3, 18,* and *41* are contained in *NCHRP Report 222.*

62. Federal Highway Administration, "Review of the Design and Construction of Long Span, Corrugated-Metal Buried Conduits." *Report No. FHWA-RD-77-131* (Aug. 1977).
63. "The Reinforced Concrete Arched Bridge—BEBO System." BEBO-International Heierli and Company, Zurich, Switzerland.
64. "Precast Concrete Arch Bridge Saves Minnesota Town 17%." *Civ. Eng.* (Nov. 1981) pp. 26.
65. "Armco Multi-Plate." Catalog MP-1676, Armco Steel Corporation, Middletown, Ohio (1976).
66. "Corrugated Steel Pipe." National Corrugated Steel Pipe Association, Chicago, Ill.
67. "Design Data." American Concrete Pipe Association, Vienna, Va.
68. "Aluminum Storm Sewers." Kaiser Aluminum Inc., Form No. HP-103, Edition 2 (1977).
69. FADL, A. I., GAMBLE, W. L., and MOHRAZ, B., "Tests of a Precast Posttensioned Composite Bridge Girder Having Two Spans of 124 Feet." Structural Research Series No. 439, University of Illinois, Urbana, Ill. (Apr. 1977).
70. *Prestressed Concrete Inst.* (Jan.-Feb. 1979) pp. 62.
71. "Long Key Bridge." Figg and Muller Engineers, Inc., Tallahassee, Fla.
72. "Bridge Report." Posttensioning Institute.
73. "Precast Segmental Box Girder Bridge Manual." Posttensioning Institute and Prestressed Concrete Institute, Chicago, Ill. (1978).

NAME OF SYSTEM: Long-Span, Corrugated-Metal Buried Conduits	SYSTEM NUMBER M-6
	PAGE 1 OF 3

DESCRIPTION:

Structures are made of corrugated-metal structural plate sections, field assembled in various closed or arch configurations to serve as large culverts or grade separation structures.

PROMINENT FEATURES:

Long-span structural plate culverts or grade separation structures made of steel or aluminum are frequently suitable alternatives for small bridges. Maximum span lengths available from the various manufacturers currently range from just under 40 to just over 50 ft, and multiple lines have been used where great waterway openings have been required. These buried structures are covered in Sections 1.9.10 and 2.23 of the current (1977) AASHTO Standard Specifications for Highway Bridges, and an excellent, comprehensive report on them has been published by the FHWA (referenced below). No design capability is usually required of the purchaser beyond the determination of the waterway opening because manufacturers commonly check standard designs or design-critical structures. Similarly, the presence of a representative of the manufacturer is usually required during construction. Experience on the part of the contractor or agency forces is desirable but not mandatory because only normal earth-moving and compaction procedures and equipment are used.

The AASHTO Specifications require certain minimum geometric and sectional properties and the use of special features such as thrust beams or compaction wings along the edge of the top arch section, soil bins on top of the structure, or transverse ribs. These special features, which are included in the designs of the six major fabricators, aid in compaction during construction and prevent unwanted distortion of the structure.

As is the case with all large, flexible, buried structures, proper construction procedures must be followed. Compaction during backfilling is most important in attaining the desired load-carrying capacity, and the configuration of the barrel must be held within specified limits.

Assuming that a site will accept a culvert configuration with at least the minimum required cover and that acceptable backfill material is available nearby, considerable economy may be realized. Among the advantages cited by manufacturers are relative ease of delivery in rural areas, savings when the bearing capacity of the subgrade is too poor for economical bridge foundations, and the elimination of deck distress from deicing salts.

NAME OF SYSTEM:	SYSTEM NUMBER
Long-Span, Corrugated-Metal Buried Conduits	M-6
	PAGE 2 OF 3

CASE EXAMPLES:

Over 600 long-span, corrugated-metal, buried structures have been built in the United States and Canada since 1960.

MANUFACTURERS:

Armco Steel Corporation, Kaiser Aluminum Company, Republic Steel Corporation, Syro Steel Company, U.S. Steel Corporation, Westeel-Rosco, Ltd. (Canada)

REFERENCES:

Federal Highway Administration Report FHWA-RD-77-131 (Reference 62).

SYSTEM NUMBER M-6 PAGE 3 OF 3

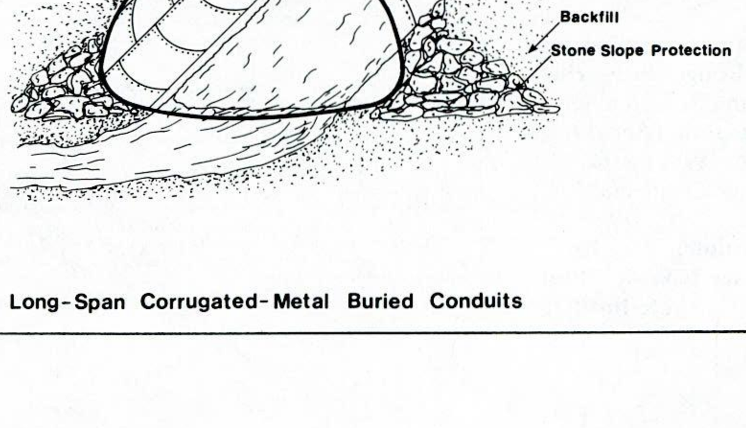

FIGURE 43. Long-Span Corrugated-Metal Buried Conduits

NAME OF SYSTEM: Precast Concrete Arch Bridge	SYSTEM NUMBER M-7
	PAGE 1 OF 2

DESCRIPTION:

Modular precast concrete panels are connected with cast-in-place concrete to form an arched bridge.

PROMINENT FEATURES:

The system is proprietary and marketed under the name "BEBO." Steel arches are temporarily erected at the bridge site to support the standard size precast concrete panels, which are joined together with field welds and cast-in-place concrete. On some occasions, particularly where the arch is wide and therefore many segments are required, it may be economical to cast full length segments so as to eliminate the need for the temporary supports. Conventionaly fill material is compacted over the arch. The system is typically used for spans of less than approximately 50 ft.

CASE EXAMPLES:

The structures have been principally used in West Germany and Switzerland but one bridge has been constructed in Minnesota and others are being considered elsewhere in the United States

MANUFACTURERS:

BEBO - International Heierli & Company, Zurich, Switzerland
Hancock Concrete Products Company, Minneapolis, Minnesota

REFERENCES:

"The Reinforced Concrete Arched Bridge— BEBO System" (Reference 63)
Civil Engineering (Refernece 64)

SYSTEM NUMBER M-7 PAGE 2 OF 2

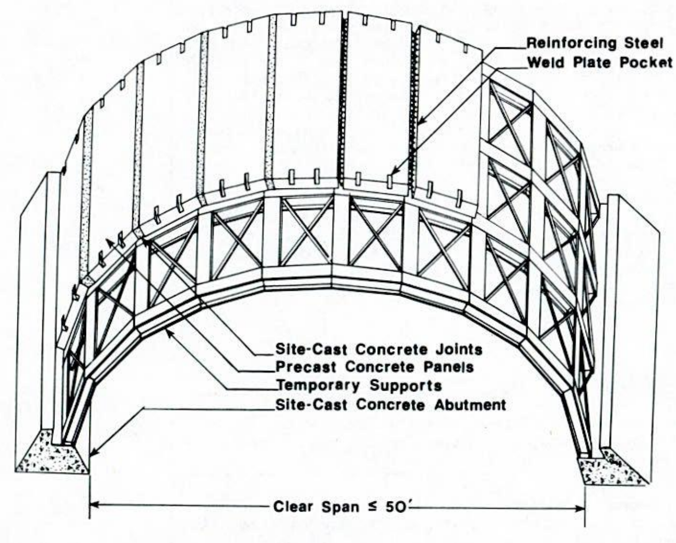

FIGURE 44. Precast Concrete Arch Bridge

NAME OF SYSTEM:

Single and Multiple Culverts of Aluminum, Concrete, and Steel

SYSTEM NUMBER M-8

PAGE 1 OF 2

DESCRIPTION:
One or more pipes of aluminum, concrete, or steel are placed so as to provide adequate drainage beneath a roadway.

PROMINENT FEATURES: Being prefabricated, culverts can be installed in a roadway in a short period of time. Metal culverts having a diameter less than about 5 ft differ from the long-span structural plate culverts (see System M-6) in that little on-site assembly is required and no special bracing is required in backfilling operations. Sections of metal culvert are usually laid end to end and coupled in a variety of ways, usually with bolted bands, hugger connections, or sleeve joints. A tongue and groove joint or sleeve is typically used to connect adjacent sections of concrete pipe or culvert. For diameters greater than about 10 ft, the metal culverts are usually assembled at the site from structural plate sections. Precast concrete U-shaped sections (no bottom) have been fabricated to accommodate spans up to about 16 ft. The precast concrete units are placed end to end on site-cast footings or floors (see Fig. 45). End walls that provide added stability and help prevent erosion are usually constructed from site-cast concrete, metal sheeting, or stone. The culverts are covered with fill material in a manner prescribed by the manufacturer. The roadway is constructed over the fill material.

Culverts have an advantage over bridges in that construction plans are seldom required, there is no deck to deteriorate, and installation is relatively rapid. The principal disadvantages are that they can restrict flow, they cannot be used on navigable streams, and they deteriorate prematurely in some corrosive environments. Steel culverts can be galvanized or coated with a bituminous material to prevent corrosion.

CASE EXAMPLES:

Numerous case studies of the use of culverts can be found throughout the United States.

MANUFACTURERS:

Manufacturers are located throughout the United States.

REFERENCES: "RTP Markets Instant Bridges" (Reference 18)
Armco Multi-Plate (Reference 65)
Corrugated Steel Pipe (Reference 66)
American Concrete Pipe Association (Reference 67)
Aluminum Storm Sewers (Reference 68)

SYSTEM NUMBER M-8

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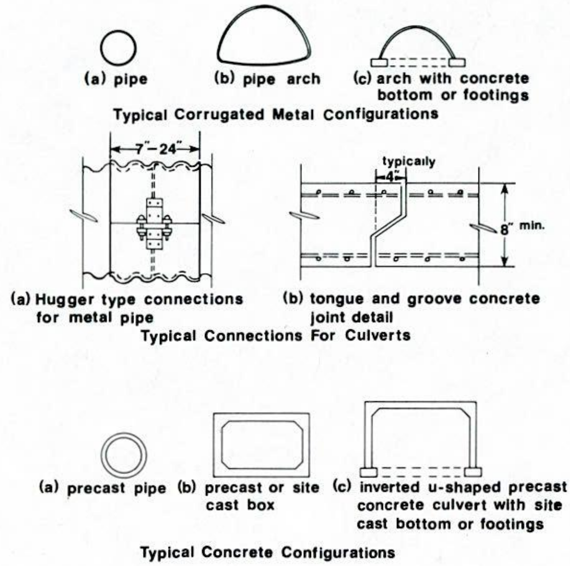

FIGURE 45. Single Culverts

NAME OF SYSTEM:

Field Connected Beams

SYSTEM NUMBER M-9

PAGE 1 OF 2

DESCRIPTION: Standard, precast, prestressed I-beams or steel beams are connected end to end in the field so as to allow the construction of a bridge with a longer span or larger deck joint spacing than is possible without the field-made connections.

PROMINENT FEATURES:

Without field-made connections the maximum bridge span length and, quite often, the maximum deck joint spacing that can be achieved is controlled by the maximum length or weight of the primary supporting beams that can be transported to the bridge site. For steel I-beams or plate girders used in the construction of a bridge superstructure (see System S-9), it has been common practice in most states for many years to use bolted or welded splice plates to connect the beams end to end so as to provide the desired span length or deck joint spacing (see Fig. 46). Long-span concrete beams could be constructed by providing forms and site casting the concrete beams to achieve the desired span length or deck joint spacings. However, the most popular concrete beams to be used in recent years are precast and prestressed ones such as the standard AASHTO I-beam (see System C-8), and these beams have not been routinely connected end to end in the field. A field-made connection which shows promise has been developed and tested at the University of Illinois (see Fig. 46). The field connection of the I-beam is achieved by supporting the I-beam segments on falsework, splicing the reinforcement between the segments, filling the joint with site-cast concrete, and posttensioning the segments.

CASE EXAMPLES: Numerous case examples of steel beams that have been connected end to end can be found throughout the United States. A prototype two-span bridge incorporating three precast, prestressed I-beam segments was constructed in Illinois in 1973.

MANUFACTURERS:
Steel connections—most steel fabricators
Concrete connections—most precast, prestressed concrete producers that can provide on-site posttensioning

REFERENCES:
U.S. Department of Transportation (Reference 3)
U.S. Department of Transportation (Reference 41)
Fadl, A. I., Gamble, W. L., and Mohraz, B. (Reference 69)

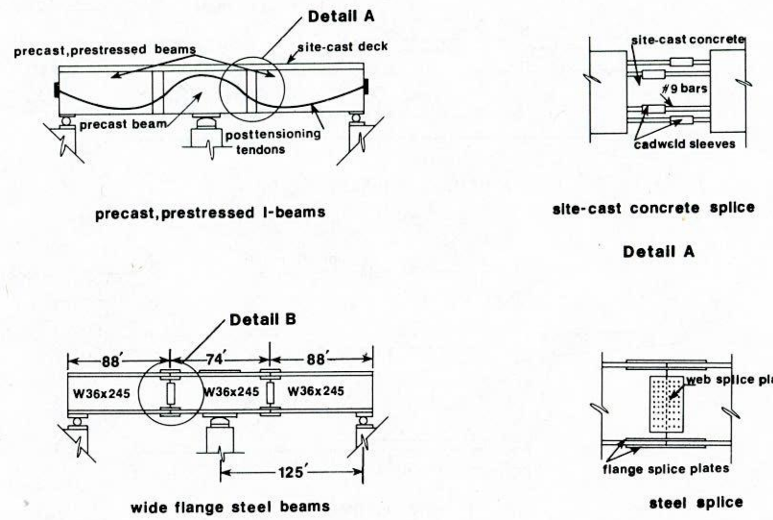

FIGURE 46. Field Connected Beams

NAME OF SYSTEM: Short-Span Segmental Construction	SYSTEM NUMBER C-9
	PAGE 1 OF 3

DESCRIPTION:
Precast or site-cast concrete segments are tied together by post-tensioning.

PROMINENT FEATURES:
Standard concrete boxes incorporating the full roadway width are precast or site-cast in a convenient length and posttensioned in the longitudinal direction to provide a continuous monolithic concrete superstructure. Although segmental construction has been popular in Europe for two decades, it began to gain popularity in the U.S. only in the late 1970's. It has been used primarily for medium to long (150 x 400 = ft spans) multiple-span bridges,but recent studies have indicated that the concept can be economical for use in constructing a typical three-span grade crossing (73). Economy requires that all the segments be cast in the same form and, if precast, that match casting generally be required. For short-span construction, the segments would probably be erected on falsework or constructed span by span on a supporting truss as shown in Figure 47. For longer span bridges, the most popular method of erection is the balanced cantilever method, but the incremental launching method and the progressive placing method have also been used. Posttensioning requirements are much simpler for short spans. A completed bridge constructed by segmental construction is shown in Figure 48.

The advantage of the system is that the shapes of the segments lend themselves to use in a variety of span lengths. Economy favors use of the same form to construct segments for many bridges. The basic disadvantages of the system are the investment in forms, the large equipment required for erection, and the engineering expertise required for a satisfactory job.

CASE EXAMPLES: Examples of medium- to long-span segmental construction can be found in Texas, Indiana, Colorado, Pennsylvania, Washington, Illionis, and Kentucky. The best example of what might be considered short-span construction is the bridge being constructed in Long Key, Florida, which will have 101 spans 118 ft in length.

MANUFACTURERS:
Some specialized contractors and consultants should be able to provide a satisfactory structure.

REFERENCES:
PCI Journal (Reference 70)
Long Key Bridge (Reference 71)
Bridge Report (Reference 72)
Precast Segmental Box Girder Bridge Manual (Reference 73)

SYSTEM NUMBER
C-9

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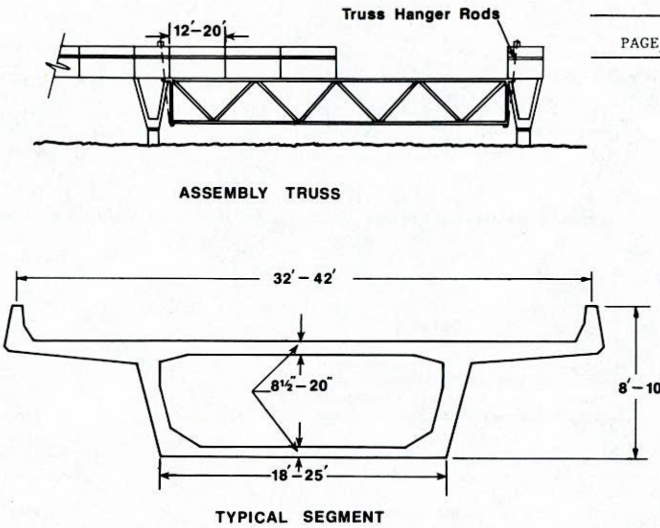

FIGURE 47. Short-Span Segmental Construction

SYSTEM NUMBER
C-9

PAGE 3 OF 3

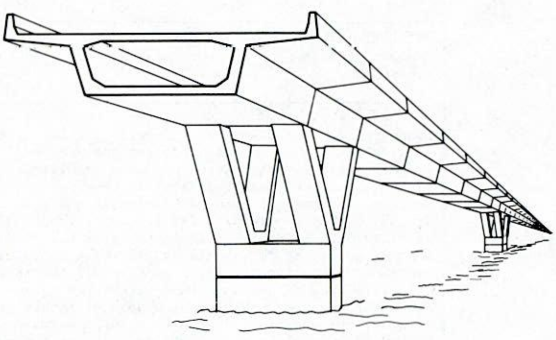

FIGURE 48. Segmental Construction - Long Key Bridge, Florida

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