NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
SYNTHESIS OF HIGHWAY PRACTICE 140

DURABILITY OF PRESTRESSED CONCRETE HIGHWAY STRUCTURES

TRANSPORTATION RESEARCH BOARD
National Research Council

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NATIONAL RESEARCH COUNCIL
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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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PREFACE

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

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

FOREWORD

By Staff
Transportation Research Board

This synthesis will be of interest to bridge designers, materials engineers, maintenance engineers, and others concerned with the durability of prestressed concrete bridges. Information is presented on the factors affecting the durability of prestressed concrete.

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

Prestressed concrete bridges require special attention because the capacity of the structure is strongly dependent on the prestressing force. This report of the Transportation Research Board reviews the factors affecting the durability of prestressed bridge components and describes methods of detecting and assessing deterioration, preventive maintenance, repair, and methods of improving durability in new structures.

To develop this synthesis in a comprehensive manner and to ensure inclusion of significant knowledge, the Board analyzed available information assembled from numerous sources, including a large number of state highway and transportation departments. A topic panel of experts in the subject area was established to guide the researcher in organizing and evaluating the collected data, and to review the final synthesis report.

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

CONTENTS

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

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Adrian G. Clary, Engineer of Maintenance, Transportation Research Board, and Crawford F. Jencks, Senior Program Officer, Transportation Research Board, assisted the NCHRP Project 20-5 Staff and the Topic Panel.

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

DURABILITY OF PRESTRESSED CONCRETE HIGHWAY STRUCTURES

SUMMARY

The first prestressed concrete bridges in the United States were completed in 1950. The use of prestressed concrete components in highway bridges increased rapidly, and by 1985 the number of bridges exceeded 60,000. Although the performance of these bridges has, in general, been good, prestressed concrete bridges require special attention because the capacity of the structure is strongly dependent on the prestressing forces. As the bridges age, the need for methods of inspection and rehabilitation increases. This synthesis reviews the factors affecting the durability of prestressed concrete components in highway structures and describes methods of detecting and assessing deterioration, preventive maintenance and methods of repair, and methods of improving durability in new construction. Suggestions for future research and an extensive list of references are also included.

The durability of prestressed concrete is determined by the condition of both the concrete and the steel. With the exception of structurally induced cracking, the factors affecting the durability of concrete and the mild steel reinforcement are the same for prestressed as they are for reinforced concrete bridges. However, because design and construction requirements usually dictate the use of high-strength concrete in prestressed components, deterioration of the concrete is not a common occurrence. Emphasis has therefore been placed on the condition of the prestressing steel, because of the potentially serious consequences of corrosion of the steel on the performance of the structure. Corrosion can occur through several mechanisms, of which localized pitting corrosion, stress corrosion, and hydrogen embrittlement are of the greatest practical importance.

Detecting deterioration in prestressed concrete structures, and especially determining the condition of embedded steel, is a difficult task. Methods of investigation that are useful to the practicing engineer, together with some recently developed experimental techniques, are described. The first task in any investigation is to define its purpose so that the scope of the work can be determined. The most important requirement of the practicing engineer is for a careful visual survey. Where the need exists, and resources permit, potential measurements and selective exposure of the prestressing steel are the most useful methods of detecting corrosion. Other techniques, such as radiography, are available, but these are expensive and are not to be used in routine investigations. Some experimental methods, such as those based on pulse transmissions and pulse echo, show promise for use as field procedures as improvements in signal processing occur. Guidance is given on interpreting the significance of the measured values for all the procedures described.

The rehabilitation of a deteriorated, prestressed concrete bridge is a complex process of planning, design, and construction. Having determined the cause of the deterioration, the method of rehabilitation and the materials to be used must be selected by following a systematic decision-making process that includes both technical and financial analyses of the alternatives. Much of the deterioration that occurs in highway structures results from the presence of chlorides in the service environment. Because it is often prohibitively expensive to remove all the chloride-contaminated concrete when rehabilitating a corrosion-damaged structure, even where it is safe and feasible to do so, there is the risk that the corrosion activity will continue. Consequently, the performance of repairs using the most common techniques, such as patching, applying concrete sealers, or using permanent formwork, needs to be monitored at regular intervals. Cathodic protection has proved effective in stopping corrosion on mild steel reinforcement, but further research is needed before its use can be considered for the protection of prestressing steel.

The repair of damage that is not corrosion induced, such as construction defects or damage from motor vehicle accidents, is usually less complex and can be undertaken with much greater assurance of long-term satisfactory performance. Relatively simple techniques, such as patching or crack injection, may be sufficient. More complex methods for strengthening prestressed structures and for using prestressing steel for strengthening reinforced concrete structures are described and examples given. Information is also presented on specialized rehabilitation procedures, such as the regrouting of ducts using resin grouts and vacuum-injection techniques.

Because prestressed concrete bridges are difficult to inspect and repair, a high priority must be given to constructing bridges that will not deteriorate. This can be done by improving the quality of the concrete, which also increases the protection provided to the embedded reinforcement; or by protecting the prestressing steel directly against corrosion; or by combining both approaches. However, a durable structure will only result from careful design, materials selection, and construction. Designs that eliminate details that trap moisture and debris, and that are easy to construct, are inherently more durable. During construction, good practices and rigorous quality-control procedures are needed. Techniques for increasing the corrosion protection provided by the concrete include the use of mineral additions such as fly ash or silica fume, corrosion inhibitors, and concrete sealants. Methods for improving the corrosion resistance of the prestressing steel include coated and nonmetallic ducts, coated strand, and dense, high-strength grout in combination with good grouting procedures. A recent development in Europe is nonmetallic tendons made from glass or aramid fibers.

The most pressing need for future research is for an economical and effective nondestructive method of detecting corrosion on prestressing steel. Further work is also needed to determine the influence of deterioration on the capacity of prestressed structures and to develop effective methods of repairing corrosion-induced damage. With respect to new construction, the long-term performance and cost-effectiveness of the nonmetallic tendons need to be determined, together with continuing efforts to improve the quality of materials and products currently in use.

CHAPTER ONE

BACKGROUND

INTRODUCTION

The purpose of this synthesis is to document the durability of prestressed concrete components in highway structures. It includes methods of improving the durability in new construction, methods of detecting and assessing deterioration, preventive maintenance, and methods of repair. Suggestions for future research are also included.

Reinforced concrete is inherently weak in tension. The concept of tensioning the steel reinforcement so as to place the concrete in a state of compression to counteract the tensile stresses resulting from service loads has been known for many years. Early attempts at utilizing the concept were unsuccessful because only mild steel reinforcement was available at the time. The prestress force that could be induced in the mild steel was relatively low and it could not be sustained in the structure because of creep and shrinkage of the concrete and stress relaxation of the reinforcement.

The development of modern prestressed concrete is generally credited to E. Freyssinet of France, who began using high-strength steel wires for prestressing in 1928. The use of prestressed concrete gradually increased in Europe but accelerated considerably after 1945, when the need for rapid postwar reconstruction coincided with shortages of materials.

Prestressed concrete may be pretensioned or post-tensioned, depending on when the prestressing steel is tensioned. Pretensioned concrete is made by stressing the steel (normally wires or strand) between fixed points, usually the ends of a rigid casting bed, and then casting concrete around the steel. After the concrete has gained sufficient strength, the strands are released from their original anchorage, thereby applying compressive stress to the concrete.

In post-tensioned concrete, the tendons are tensioned after the concrete has gained strength. This can be done by using tendons that have been encased in sheathing during manufacture. These covered tendons are cast in the concrete. Alternatively, ducts are cast in the concrete, normally using metal sheathing, through which the tendons are later threaded. When the concrete has achieved a predetermined strength, the tendons are stressed and anchored. Post-tensioned construction can be classified as bonded or unbonded. In unbonded construction, the tendons transfer stress to the structure only at the anchorages. In bonded construction, grout is injected to fill the void between the tendon and the duct. This not only protects the tendon against corrosion but also increases the ultimate strength capacity of the component. Equally important, under overload conditions, crack widths with unbonded tendons are larger than with bonded tendons, which control both crack width and spacing. It is also necessary to distinguish between internal tendons, which are embedded in a member, and those that are external. External tendons are most frequently used in cellular sections and must be unbonded. Except for external tendons, unbonded construction is very rarely used in highway bridges because of the severity of the service environment and the uncontrolled crack width and reduction in capacity at the ultimate limit state.

The first bridges in the United States to utilize prestressed concrete were completed in 1950. The development of an industry making standardized, economical precast units to serve the needs of an expanding highway market in the 1950s and 1960s soon followed. By 1985, there were more than 60,000 prestressed concrete bridges in the United States, and a majority of these are simple, short-span bridges consisting of a cast-in-place concrete slab on precast, pretensioned beams.

Although the first bridges in the United States were post-tensioned, the technique has been used in a small proportion of the total number of prestressed bridges. However, the number of applications can be misleading, and, in terms of total area of bridges constructed, the use of post-tensioning has been much more significant. Its use has increased substantially in the last decade, with the construction of the more spectacular long-span concrete bridges of segmental and, more recently, cable-stayed design.

Prestressed concrete bridges in general have a great deal of strength. Because the strength of a member depends on the prestressing forces, the condition and protection of the prestressing steel is of overriding consideration.

The corrosion of prestressing steel can be more serious than that of normal reinforcing steel for two reasons. First, the prestressing steel has a smaller cross section and is therefore proportionally more adversely affected by local corrosive attack than mild steel reinforcement. Second, the removal by corrosion of a certain proportion of the steel in a prestressing tendon represents a greater proportion of the strength of the structure because the tendon is working at a higher tension than normal reinforcement. On the other hand, the protection provided by the concrete has been enhanced by the use of high-strength concrete, as it is uneconomical to use low-strength concrete because the prestressing losses are too great. High-strength concrete is also used so that the member can achieve sufficient strength for prestressing in the shortest possible time to speed production. The requirement for a specified strength at the time of prestressing often results in a higher concrete strength at later ages than would otherwise be required by the loading conditions. However, except for external tendons, the steel is inaccessible, and nondestructive methods for detecting corrosion have serious limitations. The structural repair of prestressed concrete components is also very difficult from both the technical and practical points of view. Furthermore, a substantial number of the prestressed concrete bridges in the United States are reaching an age when deterioration might be anticipated. These considerations led to the preparation of this synthesis report, which is intended to address the needs of the practicing engineer. Inevitably, in a subject so broad as the durability of prestressed concrete bridges, the detail with which some aspects of the subject must be treated is limited. Consequently, the report includes an extensive list of reference material.

DEFINITIONS

The following definitions are taken from the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges.

Bonded Tendon—Prestressing tendon that is bonded to concrete either directly or through grouting.
Coating—Material used to protect prestressing tendons against corrosion, to reduce friction between tendon and duct, or to debond prestressing tendons.
Couplers (Couplings)—Means by which prestressing force is transmitted from one partial-length prestressing tendon to another.
Creep of Concrete—Time-dependent deformation of concrete under sustained load.
Curvature Friction—Friction resulting from bends or curves in the specified prestressing tendon profile.
Debonding (Blanketing)—Wrapping, sheathing, or coating prestressing strand to prevent bond between strand and surrounding concrete.
Duct—Hole or void formed in prestressed member to accommodate tendon for post-tensioning.
Effective Prestress—Stress remaining in concrete because of prestressing after all calculated losses have been deducted, excluding effects of superimposed loads and weight of member; stress remaining in prestressing tendons after all losses have occurred, excluding effects of dead load and superimposed load.
Elastic Shortening of Concrete—Shortening of member caused by application of forces induced by prestressing.
End Anchorage—Length of reinforcement, or mechanical anchor, or hook, or combination thereof, beyond point of zero stress in reinforcement; mechanical device to transmit prestressing force to concrete in a post-tensioned member.
End Block—Enlarged end section of member designed to reduce anchorage stresses.
Friction (post-tensioning)—Surface resistance between tendon and duct in contact during stressing.
Grout Opening or Vent—Inlet, outlet, vent, or drain in post-tensioning duct for grout, water, or air.
Jacking Force—Temporary force exerted by device that introduces tension into prestressing tendons.
Loss of Prestress—Reduction in prestressing force resulting from combined effects of strains in concrete and steel, including effects of elastic shortening; creep and shrinkage of concrete; relaxation of steel stress; and, for post-tensioned members, friction and anchorage seating.
Post-tensioning—Method of prestressing in which tendons are tensioned after concrete has hardened.
Precompressed Zone—Portion of flexural member cross section compressed by prestressing force.
Prestressed Concrete—Reinforced concrete in which internal stresses have been introduced to reduce potential tensile stresses in concrete resulting from loads.
Pretensioning—Method of prestressing in which tendons are tensioned before concrete is placed.
Relaxation of Tendon Stress—Time-dependent reduction of stress in prestressing tendon at constant strain.
Shrinkage of Concrete—Time-dependent deformation of concrete caused by drying and chemical changes (hydration process).
Tendon—Wire, strand, or bar, or bundle of such elements, used to impart prestress to concrete.
Transfer—Act of transferring stress in prestressing tendons from jacks or pretensioning bed to concrete member.
Transfer Length—Length over which prestressing force is transferred to concrete by bond in pretensioned members.
Wobble Friction—Friction caused by unintended deviation of prestressing sheath or duct from its specified profile or alignment.
Wrapping or Sheathing—Enclosure around a prestressing tendon to avoid temporary or permanent bond between prestressing tendon and surrounding concrete.

USE OF PRESTRESSED CONCRETE IN HIGHWAY STRUCTURES

The first prestressed concrete highway bridge in the United States was built in Madison County, Tennessee, and opened to traffic on October 28, 1950 (*1*). The three-span bridge had a total length of 81 ft (24.7 m) and was constructed by post-tensioning precast concrete blocks with mortar joints. It was followed shortly thereafter by the better-known Walnut Lane Bridge in Philadelphia, which was completed in the fall of 1950 and opened to traffic in February 1951. The construction of this bridge began in April 1949, and it was claimed to be the first prestressed bridge in the United States (*2*). The bridge consisted of post-tensioned, cast-in-place I-girders and had a center span of 160 ft (48.8 m). The first recorded use of pretensioned concrete in a highway structure was a single 24 ft (7.3 m) span box beam bridge erected in Hershey, Pennsylvania, in December 1951 (*1*).

The use of prestressed concrete in highway structures grew rapidly. By the end of 1952, there were prestressed concrete bridges in 8 states, and this grew to 17 states in 1954, 30 in 1956, and 41 in 1958 (*3*). The rapid growth has been attributed to the ability of American engineers to move from the individual, European-dominated practice like the Walnut Lane Bridge to relatively small, economical mass-produced components such as those used on the Illinois Tollway (*4*). By 1985, there were more than 60,000 prestressed concrete bridges in the United States, roughly equally divided between federal-aid and off-system bridges. A histogram showing the date of construction is given in Figure 1, which indicates that there are now approximately 16,000 prestressed bridges that are more than 20 years old.

Table 1 shows the number of prestressed bridges by state. The largest number of federal-aid structures are located in Texas, Pennsylvania, California, and Florida. The largest number of

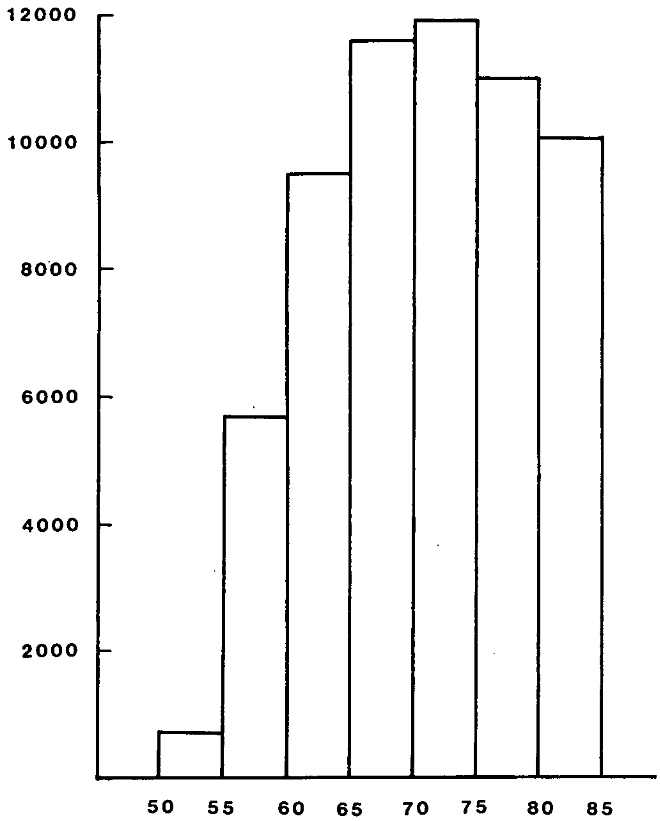


FIGURE 1 Prestressed bridges in the United States by year of construction.

TABLE 1

PRESTRESSED CONCRETE BRIDGES IN THE UNITED STATES

	Number of Bridges			
	Federal-Aid		Off-System	
	Simple	Continuous	Simple	Continuous
Alabama	209	104	263	3
Alaska	152	5	12	0
Arkansas	385	50	106	16
Arizona	15	1	39	0
California	1,682	876	404	276
Colorado	295	99	443	24
Connecticut	237	1	140	0
Delaware	30	2	30	0
Florida	2,495	33	1,525	11
Georgia	259	34	181	14
Hawaii	88	49	58	16
Idaho	517	4	866	3
Illinois	1,671	67	3,168	44
Indiana	940	286	2,398	137
Iowa	1,263	56	1,166	59
Kansas	92	128	320	88
Kentucky	377	247	1,535	70
Louisiana	564	0	345	0
Maine	3	0	11	0
Maryland	50	1	72	4
Massachusetts	239	24	108	0
Michigan	936	21	750	1
Minnesota	1,093	5	792	0
Mississippi	1,305	61	385	0
Missouri	75	210	257	18
Montana	855	18	281	3
Nebraska	246	57	158	25
Nevada	68	26	13	7
New Hampshire	23	0	29	0
New Jersey	567	5	178	4
New Mexico	470	57	67	4
New York	378	47	303	15
North Carolina	536	24	1,057	7
North Dakota	223	113	295	62
Ohio	757	56	1,700	47
Oklahoma	527	2	336	2
Oregon	882	69	911	14
Pennsylvania	2,640	13	2,019	10
Rhode Island	73	0	15	0
South Carolina	470	0	578	0
South Dakota	129	51	417	29
Tennessee	1,119	304	697	191
Texas	3,388	195	845	28
Utah	429	48	150	32
Vermont	8	1	37	0
Virginia	447	1	170	0
Washington	615	383	550	43
West Virginia	146	9	241	1
Wisconsin	552	531	730	74
Wyoming	22	1	105	0
TOTAL	30,542	4,375	27,256	1,382

off-system prestressed bridges are located in Illinois, Indiana, and Pennsylvania. Figure 1 and Table 1 were constructed from data contained in the National Bridge Inventory in July 1986.

The rapid growth in the use of prestressed concrete in the 1950s and 1960s was strongly influenced by the expansion of the freeway network and the need for numerous short-span overpasses and underpasses (*5*). It was soon recognized that considerable economies would accrue in both design and construction from the use of standardized, precast girder sections. This led to the development of the standard AASHTO and Prestressed Concrete Institute girders in the late 1950s and early 1960s. However, some state highway departments have developed their own standard sections in response to the need for greater spans and improved efficiency and economy. More recently, new optimized sections have been proposed (*6*).

The use of post-tensioned concrete also developed rapidly in the 1960s, especially in California, as the ability to construct longer spans and skewed and horizontally curved structures was recognized. The introduction of rigid conduit in 1966 was seen as a major advance in limiting the exposure of prestressing strand to corrosion and in reducing labor costs (*5*).

Another major advance in the use of prestressed concrete in highway structures came with the recognition that prestressed concrete is not just a material but that prestressing is a construction technique. This development led to the balanced cantilever method of construction, in which bridges are built entirely without falsework. The technique was used in Europe in the 1960s, but it was not until the mid to late 1970s, with the construction of both precast and cast-in-place segmental bridges, that it became popular in North America. The first application of precast segmental box-girder construction in the United States was the Corpus Christi Bridge, which was opened to traffic in 1974 (*7*). The first U.S. application of cast-in-place segmental cantilever construction was the Pine Valley Creek Bridge in California, which was also completed in 1974. The construction of these types of bridges, in which segments are attached in turn first to one side of a pier cap and then the other, would not be possible without prestressing. A study completed in 1982 (*8*) concluded that it would be feasible to develop standardized sections for segmental prestressed concrete box-girder bridges. However, because of design complexities, varying construction methods, long-term behavior, and span ranges, doubt was expressed about whether the degree of standardization could ever be as great as for I-girders.

Prestressing also plays an important role in the latest developments in bridge engineering in North America, including cable-stayed bridges, incrementally launched bridges, span-by-span construction, and progressive placement construction.

Other widespread uses of prestressed concrete in highway structures are precast piling (especially for marine applications) and precast bridge deck panels. There have been relatively few applications of prestressed concrete in footings, abutments, and piers (*9*).

Prestressed, precast bridge panels, typically about $2\frac{1}{2}$ to 3 in. (63 to 76 mm) thick, were first used as stay-in-place formwork for cast-in-place, reinforced concrete deck slabs in the mid 1950s (*10, 11*), and were the subject of investigation and application in several states in the 1970s (*12–14*). Both reinforced and prestressed, precast bridge deck panels have been used for full-depth deck construction on steel and prestressed concrete beams (*9, 15, 16*). Transverse panels may be post-tensioned parallel to the direction of traffic to improve shear transfer between panels (*15, 17*). Recently, the use of full-depth precast deck panels has been the subject of renewed interest as a means of replacing bridge decks while maintaining traffic on the structure and minimizing the disruption to the public (*18, 19*). One of the best-known applications of this technique was in the redecking of the Woodrow Wilson Memorial Bridge on I-495 in Washington, D.C. (*17, 20*), where precast, prestressed, lightweight concrete panels were used. After placement, the panels were post-tensioned longitudinally to reduce cracking. All the work was done at night over a period of one year.

Typical examples of the use of prestressed concrete are shown in Figure 2. Additional examples of precast elements and installation details are contained in Synthesis 119 (*9*).

STEEL FOR PRESTRESSING

The AASHTO Standard Specifications for Highway Bridges require that prestressing steel conform to one of the following specifications:

"Uncoated Stress-Relieved Wire for Prestressed Concrete," AASHTO M 204 (ASTM A 421)

"Uncoated Seven-Wire Stress-Relieved Strand for Prestressed Concrete," AASHTO M 203 (ASTM A 416)

"Uncoated High-Strength Steel Bar for Prestressing Concrete," AASHTO M 275 (ASTM A 722)

AASHTO M 204 covers two types of uncoated stress-relieved wire—Type BA (Button Anchorage), in which cold-end deformation is used for anchoring purposes, and Type WA (Wedge Anchorage), in which the ends are anchored by wedges and no cold-end deformation is involved. The wire must have a minimum tensile strength of 235 to 250 ksi (1.62 to 1.72 GPa), depending on the type and the diameter of the wire. AASHTO M 203 covers two grades of seven-wire, stress-relieved strand for use in pretensioned concrete. Grade 250 and Grade 270 have minimum ultimate strengths of 250 ksi (1.72 GPa) and 270 ksi (1.86 GPa) respectively. AASHTO M 275 covers high-strength steel bars having a minimum tensile strength of 150 ksi (1.03 GPa). Bars with an ultimate strength of 160 ksi (1.10 GPa) are available on special order (*21*). Two types of bar are provided: Type I has a plain surface and Type II has surface deformations. The deformations act as threads and make it possible to cut the bar at any point and screw on an anchor or a coupler. Prestressing steel not specifically listed in the above specifications may be used providing that it conforms to the minimum requirements of the appropriate specification.

Although the type and grades of prestressing wires and strand are reasonably well standardized between North America and the European countries, such is not the case for prestressing bars. Similarly, wires and strands can generally be used with any of several anchorage systems, whereas bar can only be used with one anchorage device. This lack of standardization, which makes it very difficult to prepare general recommendations for the quality and the use of high-strength steel bars, is being addressed by an ad hoc committee of the Federation Internationale de la Precontrainte (FIP) (*22*).

High-strength wire is made from high-carbon steel. The steel is rolled into rods that then go through a process called patenting, which is a heat treatment used to obtain a uniform metallurgical structure combining high tensile strength with ductility (*23*). The rod is then drawn through tapered dies of successively smaller size in a process known as drafting or cold drawing. In drawing the wire through the dies to increase its tensile and yield strength, the outer surface is squeezed and receives more cold working than the center portion. The resulting wires have residual surface stresses that are not uniform across the wire cross section. Frequently the wire is subjected to further treatment to modify its physical properties. The most common treatments employed are stress-relieving, oil tempering, and straightening.

Stress-relieving is a carefully controlled time-temperature treatment. The wire passes through a bath of molten lead at about 800° to 825°F (427° to 441°C), or through heated air in an air-tempering furnace. The wire itself reaches a temperature of only about 600°F (316°C), which has the effect of reducing the tensile strength by about 5 percent, but the ductility is greatly increased. The residual stresses are removed without destroying the fibrous grain structure of the steel.

A similar stress-relieving heat treatment carried out under load results in plastic deformation of the steel and is used to produce low-relaxation strand (*24*).

Oil tempering is a heat-treatment process in which the fibrous structure is destroyed by heating the wire to about 1700°F (930°C), quenching it in an oil bath and immersing it in lead at about 800°F (425°C). The elastic limit is increased by this process, but ductility remains low (*25*). Oil quenched and tempered steel, which has a higher tendency to stress corrosion and hydrogen embrittlement than cold drawn wire (*26*), is not permitted by the AASHTO specifications. However, the product is quite widely used in Europe.

Drawn wire retains a high degree of curvature when wound on a reel directly from the wire drawing block. This makes the wire difficult to handle and it is therefore mechanically straightened.

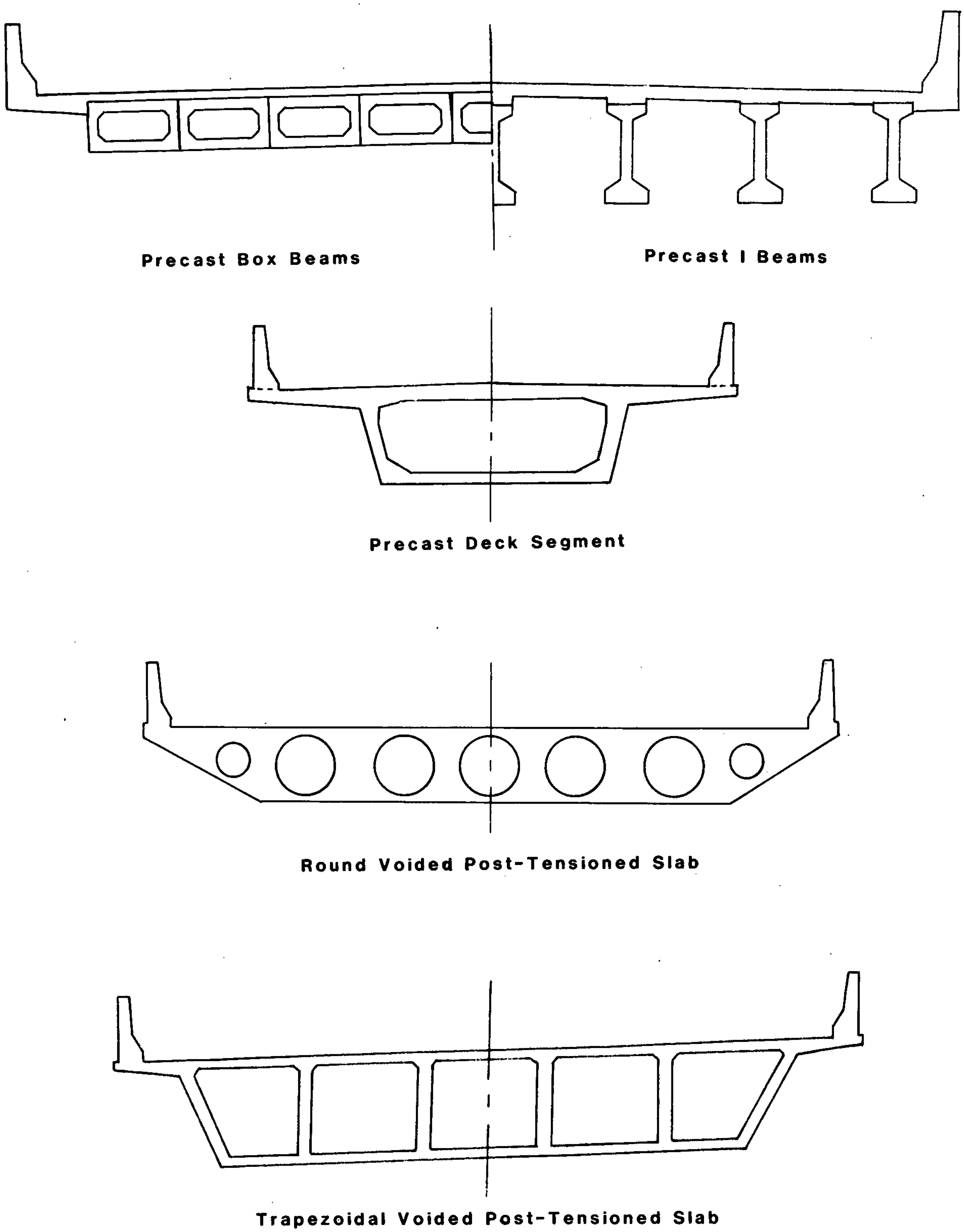


FIGURE 2 Examples of the use of prestressed concrete in highway bridges.

When cold drawn wire sustains a high tensile stress, it creeps. Creep is defined as the continuing elongation of the wire under constant load. The amount of creep for as-drawn wire increases linearly with the logarithm of time. Stress-relieving makes a pronounced change in the creep characteristics of a wire. At stresses below about 50 percent of its ultimate strength, the wire will show no appreciable creep. Beyond 50 percent, creep begins to become noticeable and gradually increases as the stress approaches the ultimate strength.

Relaxation is defined as the loss of stress in a stressed material held at constant length (constant strain). Although there is a similarity between relaxation and creep, there is no satisfactory quantitative relationship between them (*25*). The conditions surrounding a tendon in a structure are more comparable to a relaxation test than a creep test and constitute a more valid approach for the designer. The time period for relaxation tests is usually 1000 hours. For as-drawn wire, relaxation follows a substantially straight logarithmic law. In stress-relieved wire, there is very little relaxation at stresses up to about 55 percent of ultimate strength. At higher stresses the relaxation increases gradually until, at about 70 percent, the relaxation of stress-relieved and as-drawn wires is about equal.

Seven-wire strand for prestressing is made by stranding as-drawn wires. Six wires are stranded to form helices around a straight, central seventh wire. The central wire has a diameter about 4 percent greater than the other wires in order to make sure that the outer wires, when under stress, will grip the center wire securely and prevent it from slipping relative to the strand. After stranding, the wires are stress-relieved by an electrical induction stress-relieving process.

Developments of quenched and tempered steel in Japan have produced a low-carbon, silicone-chrome steel comparable in corrosion behavior to stress-relieved steel in accelerated tests. The steel has been evaluated and found suitable for general prestressed concrete use (*27*) but has not been permitted in highway structures. More recently, metallurgical studies of high-strength, high-ductility, dual-phase steel at the University of California at Berkeley have led to the development of prestressing tendons with an ultimate strength of 300 ksi (2.07 GPa). Dual-phase steel is a term used to describe a low-carbon, low-alloy steel that contains two metallurgical phases, one of which is the ductile phase; the other phase imparts strength and toughness. It is claimed that the steel will be produced at about the same cost as that of current prestressing steel (*28*).

There are numerous anchorage devices and other hardware available for post-tensioned concrete construction, much of which is proprietary. A summary of the hardware is provided in Reference *21* and a complete description in Reference *7*.

CHARACTERIZATION OF DEFECTS AND DETERIORATION OF CONCRETE COMPONENTS

The most common forms of defects and deterioration that occur in the concrete components of prestressed concrete are described below.

Cracking

A crack is defined as an incomplete separation into one or more parts, with or without a space between them (*29*). The significance of cracks in concrete is dependent on their origin and whether the length and width increase with time. There are several possible causes of cracking in concrete structures. The most common causes and characteristics of the cracks resulting from each cause can be summarized as follows (*30*):

- Plastic shrinkage cracks result from rapid drying of the concrete in its plastic state. The cracks are usually wide but shallow and often in a well-defined pattern or spaced at regular intervals.
- Drying shrinkage cracks result from the drying of restrained concrete after it has hardened. They are usually finer and deeper than plastic shrinkage cracks and have a random orientation.
- Settlement cracks may be of any orientation and width, ranging from fine cracks above the reinforcement that result from settlement of the formwork to wide cracks in supporting members caused by foundation settlement.
- Map cracking (a closely spaced network of cracks) usually results from chemical reactions between the mineral aggregates and the cement paste. The number and the width of the cracks usually increase with time. A number of reactions are possible, although the reactions between the alkalis from the cement, or from external sources, and some aggregates (producing alkali-silica or alkali-carbonate reactions) are the most widespread. Both types of reactions result in serious damage to the concrete by causing abnormal expansion, cracking, and loss of strength (*31*).
- Corrosion-induced cracks (resulting from the corrosion of embedded reinforcement) are usually associated with shallow cover and are located directly above the reinforcement. The cracks often terminate at the reinforcement, and rust stains may be associated with them. The width of the cracks increases with time as the corrosion continues.
- Structural cracks may result from any of the following: inadequate flexural and shear capacity, incorrect consideration of thermal stresses, insufficient attention to stresses developed by curvature of tendons, improper or inappropriate construction techniques, failure to meet tolerance requirements, and understrength materials (*32*). It should be noted that cracks are not totally avoidable in prestressed concrete structures because not all portions of the structure are precompressed in three directions and shrinkage is not always controllable. Certain cracks may be structurally serious, although others are not. It is therefore important to determine the structural significance of a crack and its effect on the serviceability of the structure.

Flexural cracking is associated with tensile stresses that exceed the tensile capacity of the concrete. Consequently, in continuous girders, they are usually found at the bottom of the girder in positive moment areas and at the top of the girder in negative moment areas. Flexural cracks in some segmental bridges have resulted from underestimation of the moments from any of a number of causes, including the redistribution of creep effects, lack of consideration of thermal gradients, and stress distribution in the vicinity of anchorages or tendon terminations. In areas near the support, the effect of shear will be superimposed upon the flexural stresses, as shown in Figure 3(a).

Shear cracks occur in the webs and are inclined at approximately a 45-degree angle. They are most common in a zone between the support and an inflection point, as shown in Figure 3(b).

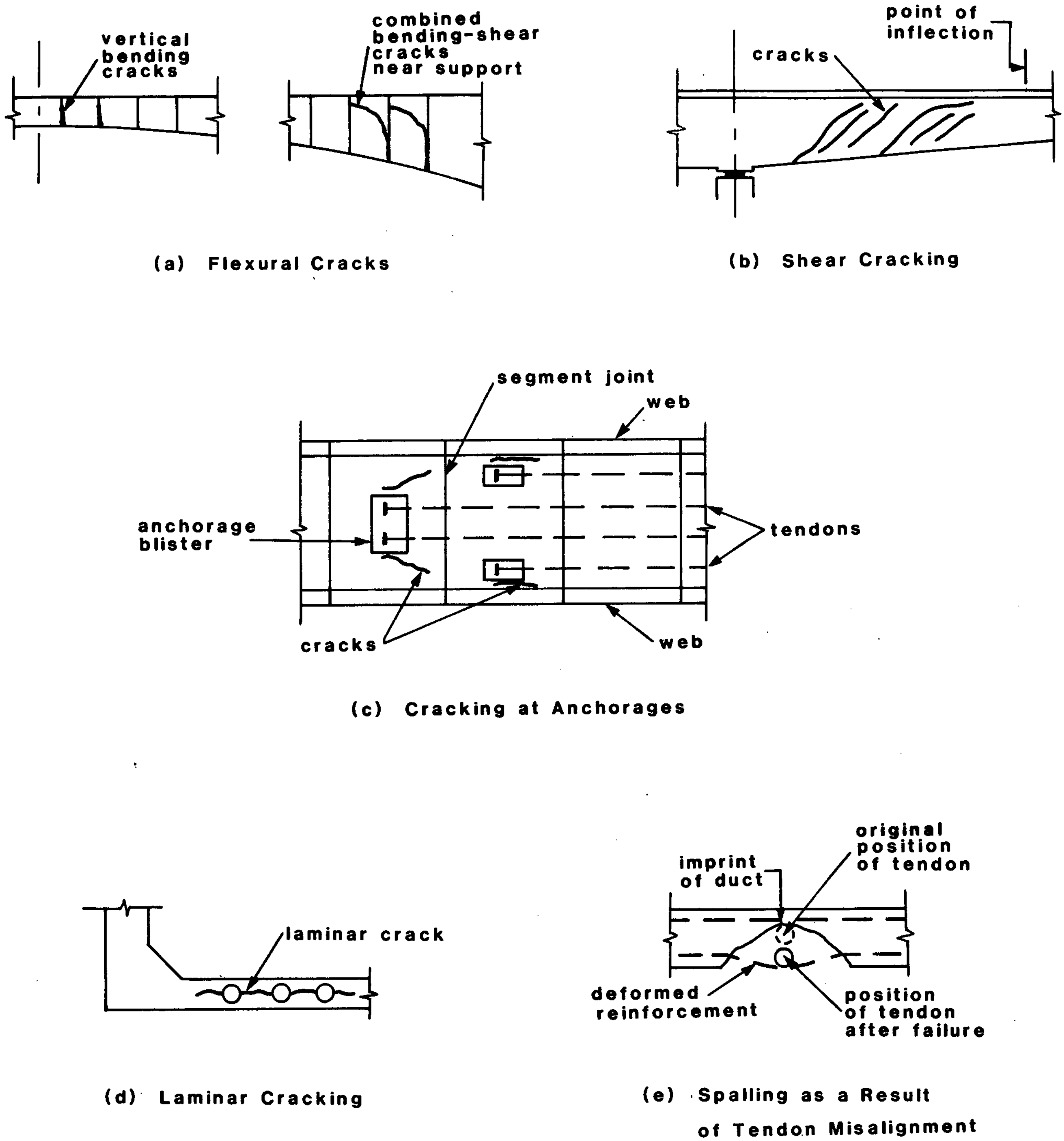


FIGURE 3 Examples of cracking in prestressed concrete bridges.

Thermal stress cracking can result from improper consideration of thermal expansion or thermal gradients, or both. Thermal gradients can result in stresses larger than live-load stresses, especially in long-span, continuous structures. Thermally induced cracks most often occur as vertical cracks close to intermediate supports.

Cracking may occur at or near anchorage positions because of inadequate design or poor details. Figure 3(c) shows a case of anchorage blisters attached to the bottom flange of a box girder. This cracking originates in the bottom flange and propagates toward the webs. If the rear face of the blister is located near a segment joint, cracking may develop in the joint.

Cracking may be associated with either vertical or horizontal curvature of tendons. In both cases, the cracks are oriented roughly parallel to the tendons. It may also result from misalignment of the tendons. Depending on the geometry of the component, a laminar crack may develop between adjacent tendons, as shown in Figure 3(d), or, in an extreme situation, a spalling failure could occur, as shown in Figure 3(e).

Scaling

Scaling is the flaking away of the surface mortar of the concrete. As the scaling progresses, coarse aggregate is exposed and eventually loosened. Scaling results from the repeated freezing of concrete that is critically saturated with water and is aggravated by the presence of deicing salts.

Scaling may occur in a weak surface layer resulting from poor finishing or curing practices (including construction in the rain), in which case it progresses no deeper. More commonly, it is indicative of inadequate air entrainment and may ultimately progress to complete disintegration of the concrete. Scaling is more common in older structures, in asphalt-covered decks that are not waterproofed because the bituminous surfacing prevents the concrete deck slab from drying, and in concrete located beneath leaking expansion joints.

Honeycombing and Air Pockets

These defects may be present in formed surfaces and originate at the time of construction. Air pockets result from inadequate consolidation. Honeycombing occurs when the mortar does not fill the spaces between the coarse aggregate particles. It may be caused by either incomplete consolidation or leakage of the mortar between sections of the formwork. The webs of voided structures are vulnerable to honeycombing where it is difficult to detect.

Popouts

Popouts are shallow depressions that result from the breaking away of the concrete surface because of internal pressure. They are most frequently associated with frost-susceptible aggregates. Generally, a shattered aggregate particle will be found at the bottom of the hole with a part of the particle still adhering to the popout cone.

Surface Deposits

The most common types of surface deposit are efflorescence and exudation. Efflorescence is a deposit of salts, usually white, that results from the flow of a solution (mainly dissolved calcium hydroxide) from within the concrete to the surface, where the water evaporates. Exudation is the solid or gel-like component of material discharged at an opening in the concrete surface. Both efflorescence and exudation may be associated with cracks, although this is not always the case.

Chemical Attack

The most widespread form of chemical attack on concrete is by sulfates. In the case of highway structures, sulfate attack of prestressed concrete elements is most likely in a marine environment because seawater is moderately aggressive to concrete. Sulfate attack results in the formation of calcium sulfate or calcium sulfoaluminate in the concrete. Both reactions result in an increase in solid volume, although most of the expansion and disruption is generally attributed to the formation of calcium sulfoaluminate (ettringite). Prestressed concrete is rarely in contact with sulfate-containing soil or groundwater.

The deterioration of concrete by acids is primarily the result of a reaction between these chemicals and calcium hydroxide. In most cases the reaction results in the formation of water-soluble calcium compounds, which are then leached away. Although acid attack of concrete in highway structures is not common, it can occur as a result of bird or animal droppings. In such cases deterioration is not simply the effect of organic acids on the concrete alone, but, because droppings and nesting material trap moisture (often containing chlorides), acid attack is found in combination with scaling and even corrosion of embedded reinforcement. The ends of the superstructure where ventilation is poor and the expansion joint is leaking are the most vulnerable areas of a structure.

Ammonium salts, often found in farm fertilizer, are very aggressive to concrete, even in low concentrations (*31, 33*). Ammonium nitrate is of particular concern because nitrates can initiate stress corrosion of prestressing steel (*34*).

Wear

Excessive wear may be present on concrete surfaces exposed to traffic abrasion and is usually concentrated in the wheel tracks. Serious rutting may be present where studded tires are permitted (*35*).

Erosion

Ice movement or, in the case of rivers with high bedload, abrasion by solid particles causes erosion of the surface of piers or abutments exposed to river or lakes. Similar deterioration occurs in marine structures, where erosion may be combined with cavitation damage from wave action.

CHARACTERIZATION OF DEFECTS AND DETERIORATION OF STEEL COMPONENTS

Corrosion of Embedded Reinforcement

Corrosion of reinforcement is the electrochemical degradation of steel in concrete. It occurs when the passivity of the steel is destroyed by either carbonation of the concrete or by the presence of more than the threshold concentration of chloride ions at the steel surface and an electrochemical cell develops.

Carbonation of concrete is the result of the reaction of carbon dioxide and other acidic gases in the air (which form weak acids in solution) and the alkaline constituents of cement paste. As a result, the alkalinity of the concrete is reduced and the steel is no longer protected against corrosion. The depth of carbonation increases with age but is a very slow reaction, typically no more than about 0.04 in. (1 mm) a year. The rate is highest in structures with a high water-cement ratio and in a dry environment. Carbonation has not been identified as a serious problem in highway structures, and corrosion is normally initiated by the ingress of chloride ions and not carbonation.

The quantity of chlorides necessary to destroy the passivity of steel in concrete is still a matter of some dispute, but the generally accepted value is 0.20 percent acid-soluble chloride ions by mass of cement. This quantity is termed the threshold value for corrosion. For a concrete mixture containing 660 lb per cu yd (390 kg/m^3) of cement, the threshold value corresponds to 0.034 percent chloride ion by mass of concrete or 1.32 lb chloride ion per cu yd (0.78 kg/m^3). It should be noted that a much lower limit is applied to the mixture ingredients in new, prestressed concrete construction. This is discussed in Chapter Four.

For an electrochemical cell to function, four basic elements are necessary: an anode, where corrosion takes place; a cathode, which does not corrode but maintains the ionic balance of the corrosion reactions; an electrolyte, which is a solution capable of conducting electric current by ionic flow; and a conductor, which connects the anode and cathode. In the case of steel in concrete, the anodes and cathodes occur on the reinforcing steel, which also acts as the conductor; moist concrete or grout acts as the electrolyte. This is illustrated schematically in Figure 4.

The rate of corrosion is largely controlled by the size of the anodic and cathodic areas, the distance between them, the availability of oxygen and moisture at the cathode, the polarization of the cell, and the resistivity of the electrolyte.

As the steel corrodes, it expands and often causes a delamination (a separation along a plane parallel to the surface of the concrete), usually located at or near the level of the reinforcement. As the corrosion processes continue, delamination eventually becomes detached from the concrete body, resulting in a spall. Cracks are not necessary for the occurrence of corrosion because chloride ions will penetrate uncracked, high-quality concrete. Depending on the amount of cover and degree of corrosion, cracks may or may not be present within or at the extremities of delaminated areas.

Corrosion of Prestressing Steel

The corrosion of prestressing steel is fundamentally the same as that of reinforcing bars. The high-strength steel used is marginally more reactive than mild steel and highly stressed steel is somewhat more readily corroded than lightly stressed steel, but these differences do not have any practical significance (*36*). Corrosion damage, when it does occur, is likely to have more serious consequences in prestressed structures, and stress corrosion- and hydrogen embrittlement-type failures are more likely in the highly stressed steel used for prestressing. It is possible to distinguish several types of corrosion that may occur on prestressing steel, and the characteristics of each type are discussed in the following sections.

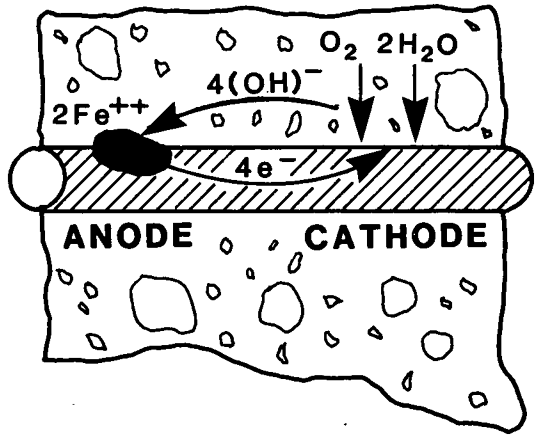


FIGURE 4 Simplified model of the corrosion of steel in concrete.

General Corrosion

General corrosion is the uniform diminution of a metal by chemical attack (*37*). Such attack ideally requires that every atom in the exposed surface be equally sensitive to the corrosive medium and that the medium have uniform access to all surface atoms. This abstract case probably never occurs in practice but is approached in the action of acids on some metals—for example, when iron is immersed in hydrochloric acid.

Localized Corrosion

Corrosion will not usually be uniform but will be localized because of the formation of local electrolytic cells that arise because of differentials within the system. These differentials may be in the chemical composition of the metal or in the concrete, variations in stress or metallurgical structure, or differences in the environment surrounding the steel.

Differentials in metallurgical structure in prestressing tendons are unlikely to be important except where parts of wires have been subjected to sharp bends or kinks. Work hardened parts are anodic to less deformed metal. This would not affect cold drawn wire, which is already fully work hardened in manufacture, but could be significant in quenched and tempered steel. Stress differences between one piece of wire and another would produce only small potentials and are not thought to be significant. The potential arising between high carbon prestressing steel and mild steel reinforcement is greater but does not seem to have led to corrosion damage that has been reported. Perhaps the most common chemical concentration cell is that caused by differential oxygen potentials. These arise in "crevice" situations; where steel emerges from concrete into air; where the porosity of the concrete varies; in imperfectly grouted ducts; or where the moisture content of the concrete varies, because moisture restricts the availability of oxygen at the steel surface. Crevices occur at many locations; for example, wire against wire, wire against duct, wire against aggregate particle. In such circumstances, the steel exposed to the higher oxygen concentration may not be passivated. Because many of the situations described are unavoidable, the danger must be minimized by limiting the supply of oxygen so that the driving potentials disappear and the reactions are stifled.

Another concentration cell of major importance that can result in a dangerous potential is that caused by differential concentrations of chloride, or any other soluble ion. Once the chloride-ion concentration exceeds the threshold value, the passivity of the steel is destroyed and corrosion can occur.

If the localized attack is independent of the crystal structure, the corrosion is termed pitting corrosion. If the attack is concentrated at grain boundaries, it is termed intercrystalline corrosion.

Stress Corrosion

Stress-corrosion cracking is another type of highly localized corrosion that is defined as the cracking of an alloy that results from the combined action of corrosion and static tensile stress, though the amount of corrosion that occurs before cracking failure is often not measurable (*3*). The stress may be either residual or externally applied. The effect on the metal may be an intercrystalline crack, the same as occurs in intercrystalline corrosion. Stress-corrosion cracking, however, does not occur in the absence of stress, whereas intercrystalline corrosion does. Unlike intercrystalline corrosion, the cracking may be transcrystalline, that is, the cracks pass through individual crystals. Generally there is a threshold stress below which stress-corrosion cracking does not occur.

Although the mechanism of stress-corrosion cracking is imperfectly understood, the fact that this type of attack occurs only in alloys indicates that the internal metallurgical structure, which is influenced by composition, heat treatment, and mechanical processing, must be a major factor. Failure is in the form of a sudden fracture without deformation.

Because prestressing steel is used in the tensioned state, all such steel is, in theory at least, susceptible to stress-corrosion cracking. In practice, only a very small number of instances of stress corrosion are known (*38*), and it is thought that this type of corrosion is caused by nitrates (*39*). Furthermore, some considerable length of exposure to nitrate solutions is required at normal temperatures (*26*). Stress-corrosion cracking of steel in chloride-contaminated concrete is most unlikely (*3*). Although stress-corrosion cracking is rare, failures because of hydrogen embrittlement are much more common.

Hydrogen Embrittlement

The ductility of steel is reduced as the hydrogen content is increased, and this can result in embrittlement and sudden failure. Reports of failure in the literature often fail to distinguish between stress corrosion and hydrogen embrittlement. In both cases the failure is a sudden, brittle fracture, but the processes leading to failure are fundamentally different. Stress corrosion is the result of anodic processes; hydrogen embrittlement is a cathodic process (*38*).

Hydrogen embrittlement is enhanced by slow strain rates and elevated temperatures. Steel that has been embrittled by hydrogen shows the following characteristics (*40*):

i) The notch tensile strength may be less than normal and directly reflects the loss of ductility caused by hydrogen.

ii) Delayed failure may occur over a wide range of applied stress.

iii) There is only a slight dependence of the time to failure on the applied stress.

iv) There is a minimum critical value of stress below which failure does not occur.

There are a number of possible sources for the hydrogen, but that which results from corrosion reactions is probably of the greatest practical importance. In the "normal" case, the atomic hydrogen that is formed in every corrosion process quickly recombines to molecular hydrogen, which, at normal temperatures, is harmless to prestressing steel. In the presence of contaminants, of which hydrogen sulfide and sulfur dioxide are of the greatest practical importance, the hydrogen is able to penetrate into the lattice of the steel, where it preferentially accumulates at the grain boundaries. Very small amounts of hydrogen sulfide (as little as a few milligrams of gas per litre of aqueous solution) can cause fracture. Although the mechanism is imperfectly understood, the hydrogen atoms combine into molecules and the steel becomes brittle (*26*). It is generally believed that hydrogen embrittlement will occur only in a medium of less than pH 9 or pH 10 (*38*). This is consistent with the development of electrolytic cells, which are the primary cause of failure. The reaction also occurs in zones of oxygen impoverishment, such as at the contact points between wires, or between wires and the duct (*41*).

Other examples of brittle fracture have been reported in which hydrogen was absorbed into the steel during corrosion before it was stressed (*36*). Failure occurred within a few hours of stressing.

In general, cold drawn wire has a much lower tendency to stress corrosion and hydrogen embrittlement than oil quenched and tempered wire of equal tensile strength (*38, 39, 42, 43*).

Galvanic Corrosion

Galvanic corrosion occurs when two dissimilar metals are present in an electrolyte. Because a potential difference exists between the two metals, the anode will be attacked and consumed.

If the two metals are electrically connected outside the electrolyte, hydrogen will be evolved at the cathode. This phenomenon has been found to occur in practice when galvanized ducts were used and grouting was inadequate (*38*), and electrical contact occurred where the prestressing steel touched the duct. If, at such points in the incompletely grouted duct, pockets of water and air are present, a galvanic cell will exist. This means that atomic hydrogen may penetrate the steel and hydrogen embrittlement could result.

The possibility of galvanic corrosion is also important when considering the selection of metallic coatings for the prestressing strand. This is discussed in Chapter Four.

Stray-Current Corrosion

Stray currents are defined as electrical currents that flow along unintended paths. Stray-current corrosion occurs when a direct current flows at least partly electrolytically. Any metallic conductors in the electrolyte may then become a path for the current. The current entering the conductor causes cathodic effects. The point where it leaves the conductor is subject to anodic effects (corrosion), usually concentrated at the point of discharge from the structure. Because stray-current effects tend to be concentrated at the ends of the tendons, a post-tensioned structure is more sensitive to damage than a comparable pretensioned structure. This situation arises because in post-tensioned structures the full length of the tendon is under tension, whereas in pretensioned structures there is a reduction in tension at the ends because of the transfer length of the strand.

The cathodic reactions cannot be ignored. Because hydrogen ions are produced, embrittlement of the steel is possible, although the risk is diminished by the fact that cathodic effects take place over a larger metal surface than do the anodic effects. However, in concrete containing chloride, there is a possibility of hydrogen evolving under even very small currents.

There are very few documented cases of stray-current damage to prestressed structures, but where it occurs, the consequences can be serious. Direct-current welding activities and d.c. transmission in connection with transit systems are the most common source of stray currents. No effect has been found of stray alternating currents of normal frequency (50 to 60 Hz) on prestressing steel in concrete.

Stray-current corrosion is a specialized subject, and a more complete discussion of its effects on prestressed concrete structures is contained in Reference *42*.

PERFORMANCE OF PRESTRESSED CONCRETE STRUCTURES

There are only a relatively few reports on the condition of prestressed concrete structures, and particularly on the condition of prestressed concrete components in highway structures, in the public literature.

The most complete documentation of the condition of highway structures was prepared as part of NCHRP Project 12-5 "Protection of Steel in Prestressed Concrete Bridges." The results, sometimes supplemented by data from other sources, appeared in a number of publications (*3, 34, 44*). Very few of the approximately 12,000 bridges in use in 1966 were reported to show any visual evidence of corrosion. (The survey estimated 12,000 bridges in use in service, though Figure 1 indicates the actual number was closer to 16,000 bridges.) It was concluded that a good, dense, and impervious concrete cover, in the case of pretensioning, and high-quality grouting, in the case of post-tensioning, can offer satisfactory protection in most environments. Only one serious case of corrosion, in post-tensioning cables passing through the pontoons of the Hood Canal Bridge in Washington, was reported. However, it was recognized that even the oldest bridges were less than 16 years old at the time of the survey.

A number of failures of prestressing steel caused by corrosion have been reported outside the United States (*34*). These included external prestressing steel in a bridge in Denmark located above a railroad yard, serious embrittlement in structures in Germany built with high alumina cement, and failures of strand in Japan as a result of being exposed to sea spray during storage. In Canada, failures in a sewage-disposal plant and in prestressed concrete pipe resulted from the use of calcium chloride as an admixture in the concrete. In the United States, failures in post-tensioned cables at the Richmond Reservoir in California occurred because of a long delay between prestressing and grouting, combined with a corrosive environment (*45*). Several examples of failures of oil quenched and tempered wire in Europe were also cited (*34, 44*).

A survey of the durability performance of post-tensioning tendons published in 1978 (*46*) cited 28 structures, with approximately 200 tendons exhibiting corrosion. It was estimated that 30 million tendons of stress-relieved steel had been installed at the time of the survey. The number of corrosion incidents was dismissed as negligible on the basis that there had been no catastrophic failures and the reason for the corrosion incidents was understood and could have been avoided. However, 10 of the 28 failures reported were in bridges, and 6 of these were in a marine environment.

Unbonded tendons, often in paper-wrapped ducts and protected by grease, have been quite widely used in parking structures. Severe corrosion, including severed cables, has been reported (*47*), though the experience cannot be related directly to bridges because the quality of concrete and amount of cover used in parking structures are less than in most highway structures.

The durability of post-tensioned concrete beams in severe environmental conditions has been investigated at the Corps of Engineers facility at Treat Island, Maine (*48*). Particular attention was paid to the type of end caps used to protect the anchorage systems from deterioration. Twenty beams were exposed to twice-daily tides plus an average of 130 freezing and thawing cycles per winter beginning in 1961 for a period of between 12 and 13 years. The study found that, of all the methods tested, epoxy-concrete end caps provided the best protection to the anchorages. The post-tensioning wires were not damaged wherever they were encased in a metal duct and protected with a portland cement grout. In the single beam with unbonded wires in paper conduits filled with grease, there was heavy corrosion on the wires.

During 1982, visual inspections were made of about 20 bridges on the Illinois Tollway (*11*). More than 200 bridges incorporating prestressed concrete elements were built in 1957 and 1958 for the Tollway. Several innovations were incorporated in the construction of the Tollway bridges, including standardized, pretensioned I-girders, pretensioned sub-deck panels, and hollow, cylindrical, prestressed concrete piles for bridge piers. Most of the girders were found to be in excellent condition, with only minimal freeze-thaw deterioration or rust staining. Delaminations were found in the girders of one structure, and at least one strand had corroded and broken. Several stirrups were also severely corroded. However, the most serious problem affecting the Tollway bridges was widespread cracking and spalling at the girder ends as a result of "frozen" bearings.

The sub-deck panels were used in six bridges. No evidence of distress associated with the panels was observed.

Piers on about 90 of the bridges incorporated hollow, prestressed piles that were 36 in. (914 mm) in diameter with $4^1/_2$ in. (114 mm) thick walls. Cast-in-place concrete was used for the pier caps. The Tollway is one of the few examples of prestressed concrete being used in substructure components. Most of the piers were in good condition, though some exhibited freeze-thaw damage within the "splash zone," where solutions of deicing chemicals are splashed on the piers by passing vehicles.

There have been two well-documented failures of prestressing strands in highway bridges in the United Kingdom (*49, 50*). One structure completed in 1970 had an externally post-tensioned multicellular deck. The cables were protected by an aluminum resin within a polyvinyl chloride duct. Broken wires were discovered during a routine inspection within a year of the bridge being opened to traffic. In 1977, one complete strand failed, followed by another a short time later. This led to a decision to replace all 120 cables. The failure was attributed to moisture penetrating to the strands.

The second example was the sudden collapse of the Ynyswgwas Bridge near Port Talbot in Wales in December 1985. The structure was a post-tensioned segmental construction, 32 years old at the time of failure. The bridge had a single span of 60 ft (18.3 m) and consisted of nine I-beams, each containing eight precast sections post-tensioned together and grouted. The bridge collapsed under its own weight as a result of corrosion of the cables. The deck suffered total failure of all the internal beams. The line of failure, at approximately mid span, is shown in Figure 5. The edge beams and the blockwork parapets remained standing. Observations indicated that corrosion of the

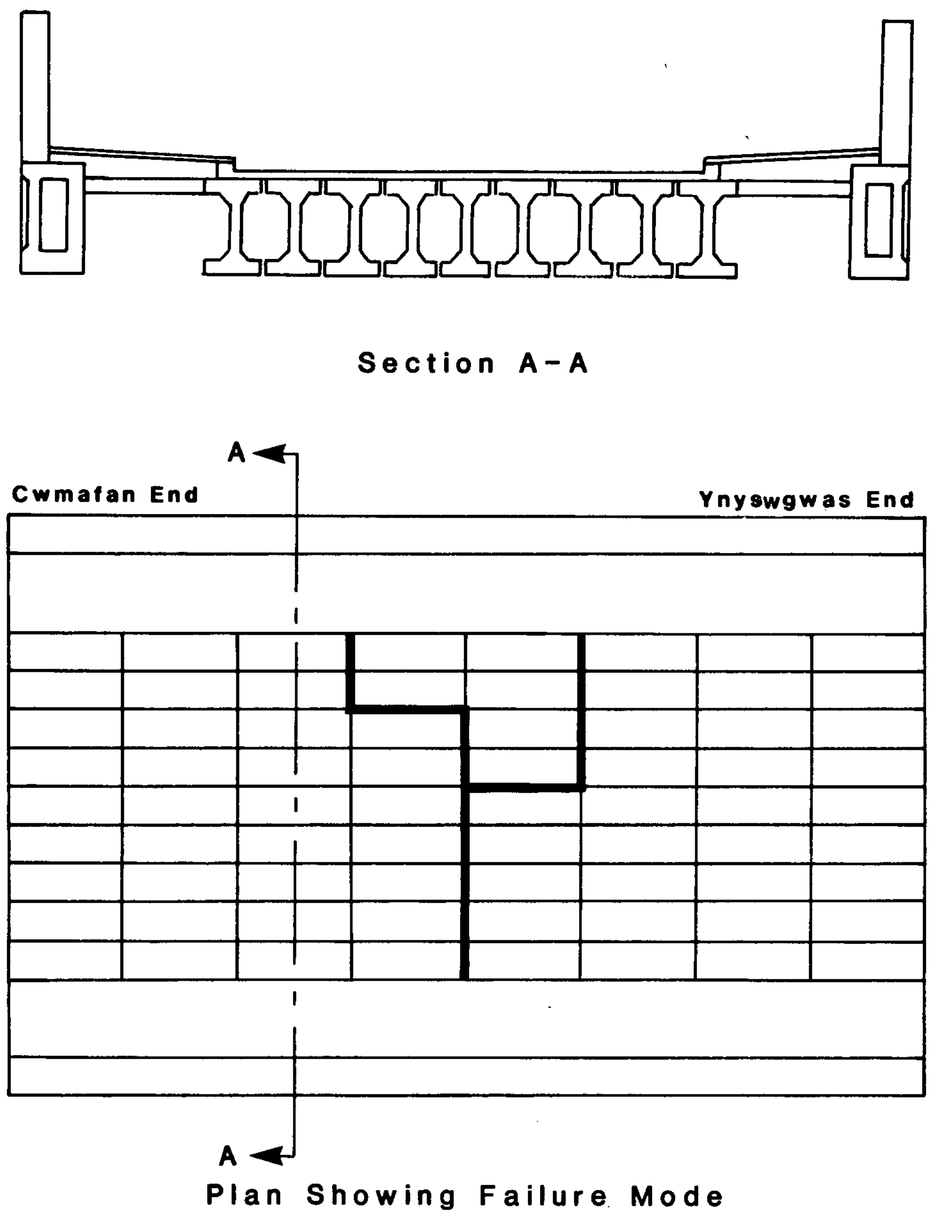


FIGURE 5 Details of the Ynyswgwas Bridge failure.

prestressing steel was confined to the location of joints. The quality of grouting in the longitudinal prestressing tendons was generally very good. The collapse is significant in that it had previously been anticipated that in bridges of this type there would be signs of damage well before there was any risk of structural collapse. Furthermore, local failure would have been expected to precede complete collapse. The cause of the collapse is under investigation.

The majority of the investigations that have examined the durability of prestressed concrete have led to favorable conclusions. Failures have been few, and a number of these have been the result of obvious defects in design, materials, or construction. However, there is a danger that such positive conclusions, combined with the dismissal of the incidence of failure as negligible, can lead to complacency. This is not to suggest that prestressed concrete is an inferior construction material for highway structures. In fact, there is evidence to suggest that it has performed better overall than either steel or reinforced concrete. The major concern is that the highway environment constitutes a particularly severe exposure condition for any structural material and this has not always been adequately recognized. In addition, prestressed concrete presents unusual difficulties for inspection and repair. Many of the prestressed concrete bridges in the United States are reaching an age when durability problems can be anticipated and vigorous inspection and rehabilitation activities, combined with research to develop new techniques, will be required to maintain these structures in service.

CHAPTER TWO

METHODS OF DETECTING DETERIORATION

INTRODUCTION

This chapter describes methods of detecting defects and deterioration in pretensioned and post-tensioned concrete components. The content has been selected to satisfy the routine needs of practicing engineers but also includes a description of specialized techniques that would only be considered for unusual situations. Some of the newer techniques are still experimental and are the subject of continuing research. The material is arranged to discuss each method of detecting deterioration in turn. The relative importance of the methods to the practicing engineer involved in the inspection and evaluation of existing prestressed concrete bridges is summarized at the end of the chapter.

There are few standard methods of investigating the condition of prestressed concrete bridges, and it is often necessary to adapt existing techniques according to the nature of the deterioration and the type of structure under investigation. It is therefore very important to understand exactly what properties of the structure are being measured in order to avoid erroneous interpretations of the data. Although both laboratory and field methods are discussed, prominence is given to techniques suitable for use in the field.

The inspection of components below the waterline presents additional difficulties, especially in deep, cold water and poor visibility. The individual performing underwater bridge inspections must not only be a fully qualified diver but also have a comprehensive knowledge of bridge substructures in order to interpret and report observations properly. Underwater inspection and repair is the subject of Synthesis 88 (*51*). The inspection and evaluation of components below grade is also a specialized field (*52*).

PLANNING AN INVESTIGATION

The first item that must be defined clearly is the purpose of the investigation and the end product. This determines the information that must be collected and the necessary level of detail. For example, a visual survey may suffice for the purpose of assessing overall integrity and structural safety of a structure. Testing may be required if the existence and approximate amount of deterioration must be known for the purposes of assessing priorities or planning for more detailed investigation and future rehabilitation. Yet a third level of investigation must be employed when it is necessary to define the extent and precise location of deterioration for the purpose of evaluating the capacity of a structure that has been subject to vehicle impact or is in poor condition and when contract documents for rehabilitation must be prepared. In some cases, the information that must be collected is governed by legal requirements.

Careful planning involves identifying the required information and the human and technical resources necessary to collect such information. It includes selecting the test procedures to be used and defining the sequence and the approximate number of readings to be taken. This is not to imply that the investigation can be defined by the engineer in the office and carried out by technicians following a predetermined set of instructions. In most cases, the opposite is true. Investigations of any structure should be carried out under the supervision of an experienced professional who can modify the procedures as the investigation progresses. This is particularly true when dealing with prestressed concrete bridges, because of the lack of well-established procedures and the need for qualified personnel who can interpret the significance of test results. Modifications may involve adjusting the number of readings taken in view of the condition of the structure or introducing additional procedures to resolve anomalies. At the planning stage, it is necessary to establish the strategy and the scope of the investigation in sufficient detail to identify the resources and budget.

In cases in which access is difficult, such as on high-level structures, many marine bridges, or where traffic volumes are heavy, the traffic-control plan and the means of providing access to the structure must be determined. This may require a visit to the site. It is desirable that close-range observations be made even though this may require special equipment for access. Such equipment is essential if physical testing is involved.

Further information on the safety, planning, organization, and manpower required for inspection and testing is given in References *30, 53–55,* and in future NCHRP Synthesis, Topic 16-01, "Bridge Inspection Practices—Equipment, Staffing, and Safety."

Although there are numerous test procedures available for use on concrete components (*56*), there are very few that can be used to determine the condition of prestressing steel. In fact, the methods for detecting corrosion on internal prestressing steel must still be considered unsatisfactory, and the assessment of corrosion damage is even more difficult. Consequently, it is usually necessary to use indirect methods of assessment, such as examining the condition of the mild steel reinforcement or the concrete surrounding the prestressing steel. Rarely will a single test procedure provide the necessary information. The challenge for the engineer is to select the right combination of procedures from which the condition of the structure can be assessed. Considerable experience is necessary to make such a selection. In fact, the process can be likened to the completion

TABLE 2

ASTM TEST METHODS FOR USE IN THE FIELD

Designation	Title
C 42	Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
C 597	Test Method for Pulse Velocity Through Concrete
C 803	Test Method for Penetration Resistance of Hardened Concrete
C 823	Practice for Examination and Sampling of Hardened Concrete in Constructions
C 876	Test Method for Half Cell Potentials of Reinforcing Steel in Concrete
D 3633	Test Method for Electrical Resistivity of Membrane-Pavement Systems
E 94	Recommended Practice for Radiographic Testing
E 494	Recommended Practice for Measuring Ultrasonic Velocity in Materials

of a jigsaw puzzle in which the results of each test method constitute a single piece of the puzzle.

Where equipment and procedures have been standardized, the applicable test methods are listed in Table 2.

VISUAL INSPECTION

Any investigation of a structure or an individual component should indicate a careful visual inspection. The first step is to look for evidence of serious, hidden deterioration and particularly the corrosion of the prestressing steel. Such evidence may be in the form of abnormal deflections, rust stains, or a separation between precast segments. Until fairly recently, it had been generally accepted that there would be sufficient warning of the impending collapse of a prestressed bridge because the probability of sufficient prestressing steel failing by corrosion at any time was considered extremely small (*57*). A design study of typical post-tensioned bridges up to 138 ft (42 m) span was undertaken to determine the effects of loss of prestress when wires were broken (*58*). It was indicated that although changes in deflection would be small and not a good indicator of loss of prestress, cracking would occur well in advance of local failure under live load. It was concluded that collapse would be unlikely to take place suddenly and without warning. The sudden collapse of the Ynyswgwas Bridge in Wales as a result of corrosion of the prestressing cables means that it cannot be assumed that such warnings will always occur. Also significant is the fact that the 32-year-old structure was reported to have been given a thorough visual inspection within a year of the collapse and no evidence of distress was recorded (*50*).

Key indicators of incipient problems that can be detected by visual inspection are (*56, 59*):

- cracking
- wet spots
- crushing or spalling of the concrete, especially in the anchorage zones or other areas of high stress
- unexpected deflections or deformations
- efflorescence or rust staining
- defects such as porous areas and environmentally induced deterioration, such as scaling, of the concrete
- general structural settlement or uplift

Special attention should be given to ensuring that structures containing voids were built with drain holes and that the drains are functioning. There have been reports of several box girders containing significant amounts of water (*1, 60, 61*). Water freezing in the void has caused cracking, and in one case was responsible for the replacement of the suspended span of a prestressed concrete overpass (*60*).

The results of a visual inspection are normally reported on forms developed for the purpose, supplemented by photographs of significant items. In most cases, access to the surface of the concrete is possible for components that are above-grade and above the waterline. The most significant exceptions are asphalt-covered bridge decks. Of particular concern are box-girder bridges where a bituminous surfacing has been applied directly to the top of the beams. Not only can the bituminous concrete trap salt-laden moisture on the deck but deterioration may go undetected. No matter what type of structure is involved, the condition of the bituminous surfacing is a poor indicator of the condition of the underlying concrete (*56*). It is possible to observe extensive deterioration of the surfacing even though the deck slab is in good condition, especially if an effective waterproofing membrane has been provided. Conversely, a bituminous surfacing in good condition may be hiding extensive corrosion damage in the deck. Because most prestressed decks consist of thick slabs or box girders, examination of the soffit rarely gives a good indication of the concrete near the top surface. Consequently, selective removal of the surfacing is necessary for a thorough visual inspection.

In carrying out a visual inspection, it is usual to describe cracks with respect to their location, orientation, and width. Depth is also important but can only be determined by coring, except where the crack is full depth and visible on opposite surfaces of the member. If the continuity of cracks between surfaces is uncertain, it can be checked using dyes (*62*). The following terms are used to describe the orientation of a crack with respect to the major axis of a member: longitudinal, transverse, diagonal, and random. On vertical surfaces, the terms vertical and horizontal are often substituted for longitudinal and transverse.

In general, it is rare that the structural cracks can be traced to a single source (*32*). Each of the sources listed in Chapter One, when taken individually, usually produces stresses or overstresses that are relatively minor. However, superposition of effects from several sources may result in cracking. Because it is difficult to diagnose the triggering mechanism, an extensive investigative effort is required, usually by a process of elimination, to determine the most probable cause or group of causative factors.

Precise measurement of crack width is not usually warranted. If necessary, crack widths can be measured to an accuracy of 0.001 in. (0.25 mm) using a crack comparator, which is a small, hand-held microscope with a scale marked directly on one lens (*63*).

Care is required to ensure that crack widths are estimated with reasonable accuracy. Raveled edges or moisture associated with the crack may make the crack more visible and appear wider than it is. Conversely, narrow cracks are difficult to identify on surfaces that are rough or uniformly wet or dry.

The location of areas of poor consolidation or honeycombing should be reported. The depth of honeycombing can sometimes be measured by feeler gauges.

Scaling should also be reported with respect to its location and severity. Areas that are wetted frequently and subject to freezing are particularly prone to scaling. These include the ends of beams and deck slabs located beneath leaking joints, bearing seats, and decks that have a bituminous surfacing but are not waterproofed.

Corrosion of reinforcement can sometimes be detected by rust stains on the concrete surface. However, care must be taken to avoid confusion from ferrous sulfide inclusions in the aggregate or the rusting of tie wires (*64*). Delaminations are not usually visible except where they are very shallow and there is a discoloration associated with them. Spalls are, of course, easily recognized, and the reinforcing steel is often exposed within the spalled area.

It should not be assumed that severe corrosion will always result in spalling. There is at least one documented case in which all 11 strands in the soffit of a prestressed box beam were rusted through as a result of water being trapped in the void without there being any spalling (*61*). The only visible defects were two longitudinal cracks and leakage stains, but the beam appeared to be in no worse condition than the adjacent beams, which were subsequently found to be in relatively good condition. The box in question was sounded and found to be extensively delaminated, and, when the delaminations were removed, the severed strands were discovered.

MEASUREMENT OF CRACK MOVEMENT

Crack movement can be measured in a number of ways (*53*, *63*), although it is often difficult to separate temperature and load effects from the permanent growth of cracks (*65*, *66*). Consequently, single crack-width measurements without correction for temperature effects can give misleading results. When it is known that movement is perpendicular to the crack, several types of transducer (most notably linear variable differential transformers or LVDTs) or extensometer can be used. Alternatively, crack movement indicators, which give a direct reading of crack translation and rotation, can be attached to the surface of the concrete. These devices are relatively inexpensive and suitable for long-term measurements. A technique has been devised using glass fibers, etched chemically to control the level of strain to fracture, to detect the onset of cracking (*67*). The device is bonded to the concrete surface, and a light source and detector at either end of a group of fibers reveals continuity or breaks in the fibers as the result of crack growth. The technique could become very useful for monitoring critical areas of structures.

INSPECTION OF CLOSED CELLS AND INACCESSIBLE AREAS

It is sometimes necessary to supplement visual inspection by using instruments to inspect areas inaccessible to the naked eye. This situation may arise when it is necessary to inspect the interior of prestressed box beams or voided post-tensioned thick slab decks or the ends of beams. A small hole (no more than 1 in. or 25 mm in diameter) permits examination of the interior of closed voids using a tubular instrument known as a borescope in North America and an endoscope in Europe. Light is transmitted through a bundle of optical fibers to illuminate the areas of interest, and the image is transmitted back to the eye through a lens system. Camera attachments enable photographs to be taken. The major disadvantage of the borescope is that the angle and the field of view are shallow and hence visibility is limited.

Larger holes, made with a core drill, enable visual observations to be made with the assistance of an electric light source and a mirror or periscope. This technique has been used to examine inaccessible areas between the ends of beams and the ballast wall of the abutment (*61*). Video cameras can also be used, although it is difficult to find equipment ideally suited to the task. Cameras used in sewer inspections are too large for this work and smaller units tend not to be sufficiently rugged for field use.

Where permanent formwork has been used, the ability to inspect closed cells is extremely limited. In such cases the deterioration would have to be very extensive to show through the forms.

EXPOSURE OF PRESTRESSING STEEL

When corrosion of the prestressing steel or deficient grouting is suspected, selective exposure of the steel may be undertaken. However, the decision is not to be taken lightly, because it is expensive and there is always the danger that holes will not be restored properly and hence corrosion could be accelerated. The new concrete in the restored areas would also not be prestressed.

In prestressed concrete structures, the primary concern is usually with locating the most serious corrosion in the structure and with the condition of the prestressing steel at the points of maximum stress. In order to determine the worst conditions, the decision of where drill holes are made should be based on visual indications, such as rust stains or wet spots, and test results, such as chloride-ion content and half-cell potentials. Care in exposing the steel is essential. In pretensioned beams, the location of the strands can often be determined with a pachometer. Because the cover to the external strands is small, the most practical means of removing the concrete is usually an electrically powered chipping hammer. In post-tensioned structures, the cables may be difficult to locate even when plans of the structure are available. Core drills can be used where access is easy (for example, on bridge decks) and the cables are deep in the member. Coring must be terminated before contact with the duct is made, and the remaining concrete over the duct should be removed with a chipping hammer. Care must also be taken not to puncture the duct, because the integrity and condition of the grout is an important piece of information. Where grout is found, a sample is usually retained for analysis of its chloride content. Where a void is found in the duct, a borescope can be used to determine the extent of the void (*57*).

In Germany, the borescope is used in conjunction with an innovative drilling procedure to examine the condition of prestressing steel. The drill is equipped with magnetic sensors that shut down the drill as soon as it is on the point of contacting steel. The procedure is used routinely and is relatively inexpensive.

Where the steel is exposed, its condition is recorded. The record should distinguish between general and pitting corrosion and note the depth of pits and the loss of cross section. An absence of corrosion should not be interpreted as indicating that the entire strand is free from corrosion, because a cathodic area of the strand may have been exposed. It is therefore important that several strands be exposed and that the interpretation be made by knowledgeable, experienced professionals.

On badly corroded structures, or when a detailed assessment of load-carrying capacity is to be made, it may be necessary to remove sections of strand or wire for laboratory analysis. A saw equipped with a carborundum blade is normally used for this purpose. Such removals must, of course, be undertaken on fully bonded cables and only under the direction of a structural engineer. Where strands are exposed or removed, the hole should be backfilled with a dense concrete or a polymer mortar. However, there is a risk that a concentration cell may be established because of the creation of dissimilar conditions that could promote further corrosion. This is discussed in the section on patching in Chapter Three.

REBOUND AND PENETRATION METHODS

Rebound and penetration methods are used to measure the hardness of concrete. Although hardness is not itself an important property of concrete, these methods can be used to detect areas of poor consolidation or to predict the strength of the concrete.

Most surface hardness methods involve indenting the surface of the concrete in a standardized manner and measuring the size of the indentation. Several devices are available (*68*), but the Schmidt Rebound Hammer and the Windsor Probe are the best known and most widely used.

The rebound hammer measures the rebound of a spring-loaded plunger as a percentage (called the rebound number) of the initial extension of the spring. The instrument is inexpensive, rugged, and easy to use but has serious limitations. The rebound number is affected by many factors, including the angle of test, surface smoothness, mixture proportions, type of coarse aggregate, the moisture content of the concrete, and carbonation of the surface (*69, 70*). The rebound hammer should be calibrated for each possible position, vertically down, vertically up, or horizontally. Several readings should be taken in each area of interest, being careful to avoid readings directly on coarse aggregate particles (*71*). The accuracy of a properly calibrated hammer in predicting the strength of concrete in an existing structure is probably no better than ± 25 percent (*68*). Consequently, the instrument is most useful in checking the uniformity of concrete in a structure with a view to identifying areas requiring further investigation. It can also be used to identify areas where the concrete is delaminated.

The Windsor Probe is a more powerful device that penetrates the concrete. It consists of a drive unit, or gun, and hardened alloy probes that are fired into the concrete. The depth of penetration is related empirically to the compressive strength of the concrete. The probes penetrate up to 2 in. (50 mm), so that the results are less influenced by surface moisture, texture, and carbonation than the results of the Schmidt hammer. However, the results are affected by the size and distribution of the coarse aggregate (*72*), with the net effect that the results of the two methods are comparable (*56*). Consequently, the Schmidt hammer is generally preferred for use on highway structures because it is cheaper, quicker to use, and less destructive.

SOUNDING METHODS

Several tools and instruments have been devised for detecting delaminations by striking the concrete surface and listening to the response. Delaminations are associated with a characteristic dull response. The first striking devices were hammers and iron rods, and later chains were introduced. More recently a device that combines a mechanical tapping device with an electronic sensor has been developed.

The use of a hammer is tedious because only a small area of the member is investigated at a time. On horizontal surfaces, a chain increases both operator comfort and speed of the survey, but on vertical surfaces there is currently no alternative to the hammer that is practical and accurate. Operator fatigue affects the accuracy of the survey, as the constant sounding tends to reduce the operator's sensitivity to changes in tone. This is especially true where echoes occur, such as when sounding the interior beams of slab-on-beam structures. Areas of delamination are normally marked directly on the structure and later recorded on a plan. Where there are numerous, small delaminations, this can be very time consuming. However, despite these limitations, sounding techniques are not only inexpensive but reliable (*56*). A portable electronic instrument known as the Delamtect was developed specifically for bridge decks to eliminate the subjective judgment of the operator and provide for direct recording of the areas of delamination (*73*). The instrument is convenient to use but is not as accurate as the manual methods.

ULTRASONIC METHODS

Striking the concrete surface with a hammer is just one example of the use of stress as the interrogating medium. In the case of the hammer, the stress waves generated are in the audible range, but there is a much larger ultrasonic range that is useful in nondestructive testing.

Stress waves can propagate through solid materials and exhibit the four principal properties of wave motion, which are reflection, refraction, defraction, and interference. The ability to use stress waves to detect flaws or discontinuities in a material is based on the fact that the boundary between the flaws and sound material offers differing resistance to the passage of the waves. The boundary acts as a site for partial reflection (or refraction, defraction, or interference) of wave energy. Thus, waves passing through a material strike either a flaw or, eventually, an external surface. The vibrations are reflected and the nature of the return signal indicates the location and type of reflecting surface.

Being a heterogeneous material, concrete has a large number of internal boundaries, and high-frequency pulses are subject to considerable attenuation. Consequently, an ultrasonic technique known as a pulse transmission or pulse velocity must be used. This involves measuring the time taken to transmit a pulse of energy through a member. The velocity of the pulse depends only on the elastic properties of the material and can be determined from the following relationship:

$$V_c = \sqrt{\frac{E}{d}}$$

where

V_c=velocity of the compression wave
E=dynamic elastic modulus
d=density

A more complete mathematical treatment of the derivation of the equation is given in Synthesis 118 (*56*). However, what is important in the practical sense is that the wave velocity is almost independent of the geometry of the material. Additional information is also provided because the elastic properties can be correlated with other mechanical properties.

Pulse-Velocity Techniques

Investigation of equipment for making ultrasonic pulse-velocity tests began in the 1940s (*74, 75*) and led to the development of the soniscope (*76*).

Further development of the equipment in the 1960s succeeded in producing ultrasonic testers that were small, battery operated, and fully portable, and that had a digital readout. They were much better suited to field use than were the heavy soniscopes, which were more suited to research studies and required trained personnel to operate. Of the commercial units available, the PUNDIT (portable ultrasonic nondestructive digital indicating tester) from Britain and the V-Meter, made in the United States, are probably the best known in North America.

The method consists of measuring the time of travel of an ultrasonic pulse passing through the concrete being tested. The pulses are generated using electronic circuitry and transformed to mechanical energy by a transducer containing piezoelectric crystals. This results in vibration frequencies that are generally in the range of 20 to 50 Hz, depending on the specific model of equipment. The pulses are repeated at the rate of 50 to 150 per second. Although the higher frequencies are more sensitive to detecting voids and can be used with much thinner specimens, they are also subject to greater attenuation. Equipment operating at a frequency of 50 Hz is not recommended for use on sections less than 6 in. (150 mm) thick, and 20 Hz equipment should be limited to use on sections more than 12 in. (300 mm) thick (*77*).

Contact with the concrete is made with a suitable acoustic coupling medium, such as petroleum jelly or a kaolin-glycerol paste. If the concrete surface is very rough it may be ground, or a smooth surface may be made by using a thin layer of plaster of paris, a quick-setting epoxy mortar, or other suitable material. A similar transducer is coupled to the concrete at a measured distance from the transmitting transducer to act as the receiver so that the time of travel between the two is measured electronically.

There are three ways of measuring pulse velocity through concrete. In the direct transmission method, the transducers are attached to opposite faces of the member. This method is preferred wherever access to opposite sides of the component is possible because it provides a well-defined path length and results in maximum sensitivity. Sometimes the geometry of the component requires that the semidirect method (in which the transducers are attached to adjacent surfaces) be used. Surface transmission (so called because the transducers are placed on the same surface) is the least satisfactory of the three methods and should be used when access to only one surface is possible. Not only is the maximum energy of the pulse being directed into the concrete, but the method only indicates the quality of the concrete near the surface and is influenced by the presence of reinforcement parallel to the surface.

Pulse-velocity measurements have been found to be a reliable indication of the overall quality of concrete. This is because the wave velocity is reduced by the presence of porosity (which affects strength) and internal cracking (which is often associated with deterioration). The ability to detect porosity also means ultrasonic measurements can be used to define areas of incomplete consolidation.

There appear to be reasonably good correlations between pulse velocity and compressive strength, which enable the concrete strength to be predicted within ± 20 percent, provided a calibration curve is established for the concrete being investigated. Pulse-velocity measurements can be used to detect voids (*78*) and cracks, providing that the crack is approximately perpendicular to the propagation of the pulse. It has been suggested that the transmission of ultrasonic waves is negligible across an air-filled crack more than 0.001 in. (0.025 mm) wide and transmission across a water-filled crack is only about 4 percent of that through uncracked concrete (*74*). A more recent study found that air-filled surface cracks 0.002 in. (0.05 mm) wide and 1.5 in. (40 mm) deep could be detected in plain concrete but similar cracks $^3/_4$ in. (19 mm) deep could not be clearly detected under laboratory conditions. Crack detection capability under field conditions is expected to be much less (*79*). When the instrument is equipped with an analog display, amplitude assessment techniques can sometimes be used successfully to detect cracks.

Although pulse-velocity techniques are useful for the assessment of defects in concrete, they provide little information about the condition of embedded reinforcement. In fact, because the velocity of the waves in steel is 1.2 to 1.9 times the velocity in plain concrete, the presence of steel can make the interpretation of the data extremely difficult. Ideally, path lengths should be chosen to avoid the influence of reinforcement, though this is rarely practical on real structures. Consequently, it is necessary to apply correction factors to the measured values. When the axis of the reinforcing bars is perpendicular to the direction of wave propagation and the quantity of reinforcement is small, the influence of the reinforcement is also small. When the axis of the reinforcing bars is parallel to the propagation of the pulse, the influence of the reinforcing bars can be substantial. Methods of calculating correction factors are usually contained in equipment manuals, and simple procedures are given in References *68* and *77*. Some theoretical considerations and more complete approaches are discussed in References *80* and *81*. In the case of two-way reinforcement parallel to the pulse, it is almost impossible to make reliable corrections. Furthermore, the correction factors assume a knowledge of the size and location of the reinforcement, which is not always available for structures in the field.

Pulse-Echo Techniques

In the pulse-echo method, both the pulse source and the receiving transducer are mounted on the same surface. The receiver monitors reflection of the pulse from internal defects and external boundaries, as shown in Figure 6. If the velocity of the pulse is known, the distance of a defect or interface can be calculated. Conventional pulse-echo techniques used on steel and other homogeneous materials cannot be extended directly to concrete because of its heterogeneous composition and the severe attenuation of the high-frequency pulses (*82*). A continuing study reported progress in detecting defects under laboratory conditions, but much work remains to be completed in the study of echo patterns and in developing equipment suitable for field use. (*83*). A pulse-echo device that utilizes a Schmidt hammer to transmit a mechanical, low-frequency stress wave into the concrete has been used in the field for many years, particularly for the assessment of fire damage and for evaluating repairs that involve bonding new concrete to old (*84*).

An attempt has been made in England (*82*) to develop a pulse-echo technique for detecting fractures in prestressing strands. The method depends upon having access to one end of the strand, and a high-frequency pulse (1.5 MHz) is applied. Wire fractures are detected by analyzing the signal reflected by the strand. There is a considerable loss of energy to the surrounding concrete, and the technique is only suitable for detecting fractures within 16 ft (5 m) of the end of the beam (*58*). Work is in progress to extend the range of the method up to 65 ft (20 m) and to improve signal-processing techniques.

Although ultrasonic techniques have the advantages of being both rapid and truly nondestructive, there are practical limitations to their use. It is often not possible to use the direct transmission mode because one surface is inaccessible. Equipment suitable for use from one surface is very desirable because, even when transducers can be attached to opposite surfaces, it is difficult to align them properly and measure the path length accurately. The interpretation of results is complicated by the heavy reinforcement found in highway structures. Despite these

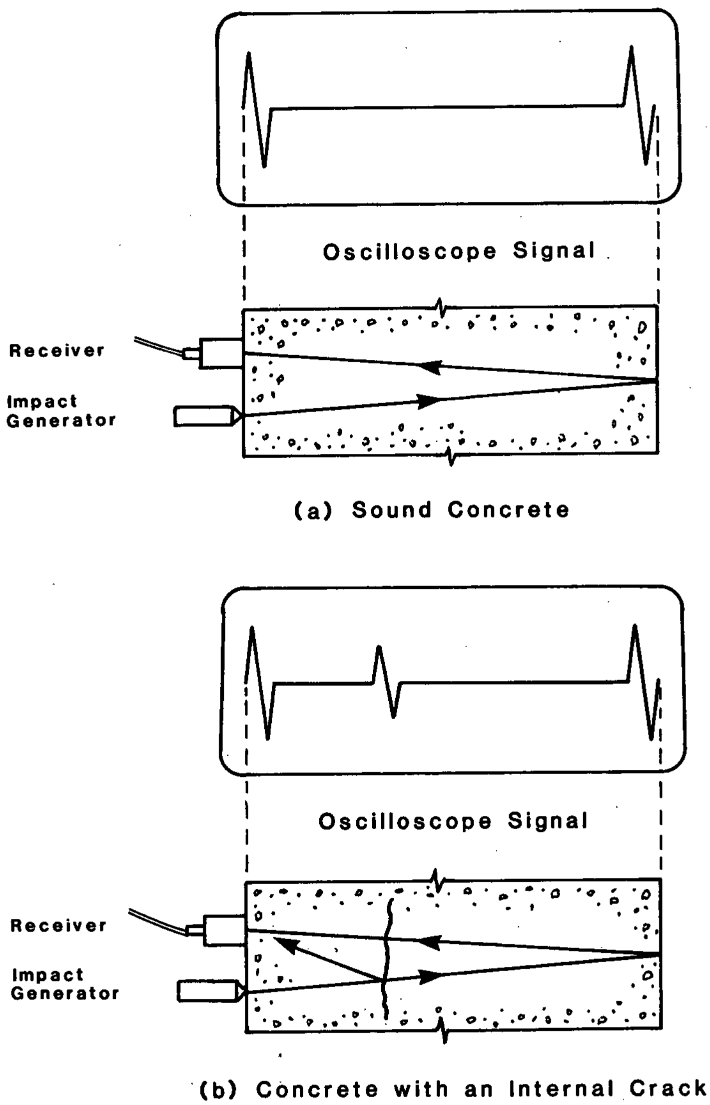


FIGURE 6 Examples of pulse-echo reflections.

limitations, ultrasonic methods can be used to advantage to identify and diagnose defects in prestressed concrete structures. Furthermore, their usefulness can be expected to increase because advances in signal-processing techniques can be anticipated.

ELECTRICAL METHODS

Electrical methods used in the field are currently limited to resistance and potential methods. However, polarization studies for measuring corrosion rates have been made in the laboratory (*85, 86*) and are being developed for field applications (*87, 88*). High-frequency capacity measurements have also been used in the laboratory to measure the moisture content of concrete (*89*).

Resistance Tests

One of the first applications of electrical resistance testing was the development in California of a method measuring the permeability of bridge deck seal coats (*90*). The method assumes that when a dielectric material is used to seal the surface of concrete, its electrical resistance is a measure of its watertightness. The procedure involves measuring the resistance between the reinforcing steel and a sponge on the concrete surface and has been published as a standard test method (ASTM D 3633). The method can be applied to any element with a nonconductive seal coat, provided that it does not contain epoxy-coated reinforcement.

More recently, interest in determining the resistivity of concrete has increased, because resistivity is one of the factors that controls the rate of corrosion of steel in concrete. As the resistivity increases, the corrosion currents decrease so that corrosion is not of practical significance in dry concrete. It is most important to note that all measurements of the resistance of concrete must be made with an a.c. meter in order to eliminate polarization effects. (To be precise, such measurements should be termed impedance rather than resistance, because of the capacitance effects of the concrete.)

Resistivity is normally measured by the four-electrode method common in geophysical testing. A major disadvantage of the test method is that surface effects dominate, whereas it is the resistivity at the level of the steel that is of primary interest. The test procedures and the means of calculating resistivity values are described in Synthesis 118 (*56*).

Moist concrete usually has a resistivity of the order 10^4 ohm·cm, whereas oven-dried concrete has a resistivity of about 10^{11} ohm·cm (*85*). The resistivity associated with corrosion activity is somewhat uncertain. Observations on marine structures in California indicated that when the resistivity exceeded 60,000 ohm·cm, no corrosion occurred, but corrosion was detected below 60,000 ohm·cm (*91*). Work in the United Kingdom has indicated that corrosion only occurs at much lower values of resistivity, and it was suggested that (*92*):

- if the resistivity exceeds 12,000 ohm·cm, corrosion is unlikely;
- if the resistivity is in the range 5,000 to 12,000 ohm·cm, corrosion is probable;
- if the resistivity is less than 5,000 ohm·cm, corrosion is almost certain.

The figures assume that the concrete contains sufficient chloride ion to initiate the corrosion reactions. Other work in the United Kingdom (*93*) has suggested that corrosion is unlikely to occur in concrete with a resistivity in excess of 20,000 ohm·cm and that resistivities in the range 5,000 to 10,000 ohm·cm are needed to support corrosion activity. Further work is needed before resistivity measurements can be used as a reliable indicator of whether corrosive conditions exist in a bridge, but the method does provide useful information to supplement potential measurements.

Consideration has been given to using the electrical resistance of steel as a method of determining the corrosion or fracture of reinforcement and prestressing steel. Although it was once thought that the method showed promise for detecting a reduction in the cross-sectional area of bars, the spread of the data was so large as to make the method impractical. Using a simplified model, it was shown (*94*) that the presence of a 3 in. (75 mm) long section with only 6 percent of the cross section remaining increases the total resistance of a 40 ft (12.2 m) bar by only 10 percent. Furthermore, a 10 percent variation could result from dimensional tolerances on reinforcement alone. This illustrates clearly that resistance measurements on steel bars are insensitive to reduction in cross section when access is available only to the ends of the beam. Furthermore, because most steel in a structure is electrically continuous, multiple circuit paths are likely to affect the results.

Potential Tests

When steel corrodes in concrete, a potential difference exists between the anodic half-cell areas and the cathodic half-cell areas on the steel. The potential of the corrosion half cells can be measured by comparison with a standard reference cell, which has a known, constant value. A copper-copper sulfate (CSE) cell is normally used in fieldwork because it is rugged, inexpensive, and reliable. The potential difference between the steel reinforcement and the reference cell is compared by connecting the two through a high-impedance voltmeter. This is done by connecting one lead of the voltmeter to the reinforcing steel. The other lead is connected to the reference cell, enabling electrode potentials to be measured at any desired location by moving the half cell over the concrete surface in an orderly manner. The cell can be used vertically downward, horizontally, or vertically upward, provided that the copper sulfate solution is in contact with the porous plug and the copper rod in the cell at all times. A full description of the equipment and test procedures has been published by ASTM (ASTM C 876). In order to detect all anodic areas, a very close grid spacing (as small as 6 in. or 150 mm) is required (*95*). This is particularly important when working on pretensioned beams where local corrosion cells are not uncommon. Some of the practical considerations in applying the test procedure are discussed in Reference *30* and in Synthesis 57 (*96*).

Convention dictates that the potentials of steel relative to the copper-copper sulfate electrode be reported as negative values. The reason for this is that iron is more negative than copper in the electrochemical series.

The interpretation of the potential measurements has been as follows:

less negative than −0.20 V (CSE)—greater than 90 percent probability of no corrosion;
between −0.20 and −0.35 V (CSE)—corrosion activity is uncertain;
more negative than −0.35 V (CSE)—greater than 90 percent probability that corrosion is occurring.

However, measurements in both the field and the laboratory have indicated that corrosion can occur at a potential of about −0.20 V (CSE). An alternative approach to interpreting the data is to examine the potential gradients on a structure. Although criteria have not been established, it is generally agreed that differences in potential of more than 50 mV are significant and differences of 100 mV are indicative of active corrosion. If positive readings are obtained, it generally indicates that there is insufficient moisture in the concrete and the readings should not be considered valid. It is important to recognize that the half-cell measurements indicate the potential for corrosion at the time of measurement but give no information about the rate of corrosion. Corrosion rates of steel in highway structures are primarily controlled by the resistivity of the concrete and the availability of oxygen at the steel surface. Consequently, it is possible to have high potential measurements but low corrosion rates. It should also be recognized, especially when working on prestressed structures, that the potential measured is that of the steel nearest to the cell. If surface measurements are made on most prestressed structures, it is the potential of the mild reinforcement that is measured. This can, in fact, be useful information, particularly if the potentials indicate no corrosion activity. Under such circumstances it is likely that the more deeply embedded prestressing steel is also not corroding. If the potential of pretensioning strand is required, wells must be drilled so that the half cell can be placed in close proximity to the strand. Unless electrical continuity can be shown to exist between the strand and other steel, the voltmeter must be connected directly to the strand. Potential measurements cannot be made through post-tensioning ducts. If the potential of post-tensioning steel is to be measured, the duct must be opened and the cell placed on the grout adjacent to the post-tensioning steel. Care must be taken that it is actually the potential of the strand that is being measured and not that of the duct, which may be corroding while the strand is not.

The disadvantages of potential measurements are counterbalanced by the fact that the test is the only nondestructive test available that is a direct measure of corrosion activity. The test is rapid, inexpensive, and relatively easy to carry out. When interpreted in conjunction with data from other tests, such as resistivity and chloride-ion content, it can be extremely useful in assessing the corrosion performance of a structure.

MAGNETIC METHODS

The main application of magnetic methods in the testing of concrete structures is in determining the position of reinforcement. Although not strictly a technique for detecting defects or deterioration, the fact that inadequate cover is often associated with corrosion-induced deterioration (*96*) means that locating the reinforcement can be an important component of a condition survey.

Pachometer Surveys

There are several portable, battery-operated devices available that have been designed to detect the position of reinforcement and measure the depth of cover. These devices are known as cover meters or pachometers. The operating principle of the meters is that a battery generates a magnetic field between the two poles of a probe. The intensity of the magnetic field is proportional to the cube of the distance from the pole faces. When a magnetic material is present (a reinforcing bar or metallic prestressing duct, for example), the magnetic field is distorted. The degree of distortion is a function of the mass of the steel and its distance from the probe. The distortion is recorded on the meter, which is calibrated to indicate the distance between the probe and steel directly.

For accurate results, the axis of the probe must be parallel to the steel being measured. Conversely, when the orientation of the reinforcement is uncertain, it can be determined by rotating the probe on the concrete surface until a maximum disturbance (minimum cover reading) is obtained. When this occurs, the axis of the probe coincides with the direction of the reinforcement.

It is generally agreed that cover meters can measure cover to within 0.25 in. (6 mm) in the range of 0 to 3 in. (0 to 75 mm), although manufacturers have claimed greater accuracy. The instruments give satisfactory results in lightly reinforced members, such as bridge decks, but in heavily reinforced members, such as prestressed beams, the effect of deeper steel cannot be eliminated, and it is virtually impossible to obtain reliable results. Parallel bars and strand also influence the meter reading if their spacing is less than two or three times the depth of cover (*68*). A further complication arises when some of the constituents of the concrete are magnetic because the measured cover is less than the actual cover. Some pozzolans, especially fly ashes, contain magnetic particles and many concrete sands contain particles of magnetite. In such cases, calibration curves must be established by exposing the reinforcement in some locations and comparing the recorded and actual values of concrete cover.

Electro-magnetic Scanning

Magnetic methods have also been applied to solving the difficult problem of developing nondestructive methods of detecting loss of section or fracture of prestressing steel, primarily in beams. An extensive study of this problem was undertaken at the Southwest Research Institute under the sponsorship of the Federal Highway Administration (FHWA) (*94*). Several methods were investigated, and preliminary studies indicated that magnetic field methods showed promise for detecting deterioration on strand in concrete and strand in a steel duct such that the method had potential application to both pretensioned and post-tensioned concrete structures.

The equipment that was developed involved applying a steady-state magnetic field to the beam by means of a large, d.c.-excited electromagnet and scanning along the beam with a magnetic field sensor. As the sensor was moved parallel to the strands, perturbations could be detected and related to defects in the beam. The equipment had a good signal-to-noise ratio and could detect a fracture when the separation between the ends of the bar was as small as 0.01 in. (0.25 mm).

Following laboratory studies, a prototype unit was constructed for evaluation in the field. The electromagnet and sensor were mounted on a cart, which ran on a track hung from the bottom of I-section prestressed beams. The power supply, drive controls, and readout were all positioned at ground level. A field trial was undertaken in Utah. This involved substantial logistical problems because the cart, electromagnet, and sensor weighed 260 lb (118 kg) and had to be suspended beneath the beams. The results were encouraging in that perturbations were apparent, but it was difficult to relate the signal responses to physical defects. Signals from steel close to the surface, such as stirrups, tended to mask responses from steel deeper in the member. It was concluded that the full potential capability of magnetic methods could only be realized by developing better techniques for discriminating between signatures from deterioration and signatures from normal reinforcement details. A second research study was undertaken (*97*) to upgrade the prototype unit by using multiple sensors and improved signal acquisition and processing capabilities. The equipment has been found capable of detecting breaks in strand, but has not been developed to the stage at which it is suitable for routine operational use. There are still some difficulties with signal recognition, such as perturbations from stirrups, chairs, and tie wires. There is also the logistical problem of access, especially near piers. However, the method is promising, and a more extensive evaluation of the capabilities of the equipment under field conditions is needed.

CHEMICAL METHODS

The principal chemical test methods applicable to concrete structures are those used to determine depth of carbonation and chloride-ion content. Both methods are used to establish if the passivity of the reinforcement might have been destroyed.

Carbonation Testing

The depth of carbonation can be measured by exposing a fresh concrete surface to a 2 percent solution of phenolphthalein in ethanol (*85*). Phenolphthalein is a pH indicator with a color change about pH 10. The magenta areas represent uncarbonated concrete, and the colorless areas, carbonated concrete. Because of the presence of porous aggregates, voids, and cracks, the carbonation only approximates a straight line parallel to the concrete surface. A fresh concrete surface can be exposed by breaking off a piece of concrete with a hammer and chisel or by taking a core and breaking it in the laboratory. The measurement must be made as soon as possible after exposing the fresh surface.

Chloride-Ion Testing

The chloride-ion content of concrete is usually measured in the laboratory using a wet chemical method of analysis. However, the method of sampling affects field operations, and there have also been attempts to measure the chloride content in situ (*98*).

The simplest method of sampling for making chloride-ion measurements is to take cores. The cores are sectioned in the laboratory and the sections of interest are pulverized and analyzed. To eliminate sawing and pulverizing in the laboratory, a percussion drill (sometimes called a rotary hammer) can be used to obtain a pulverized sample in the field. If it is desired to sample from several horizons, a hole is drilled until the required depth is reached, and the pulverized material is collected and placed in a sealed container. The hole is then thoroughly cleaned out with a vacuum cleaner before the next sample is drilled for. Sometimes it is only of interest to measure the chloride content at the level of the reinforcing or prestressing steel. When this is the case, a hole is drilled to the level of the steel, cleaned out, and the sample collected as already described.

Although pulverizing in the field offers the advantages of speed and economy, considerable care is required to prevent contamination of the samples. A common source of contamination is abrasion by the drill bit on the sides of the hole, especially near the surface, where chloride levels are highest. This problem can be overcome by using a progressively smaller drill diameter for each successive sample taken. In any event, all samples must be checked to be sure that they completely pass a 300 μm (No. 50) screen before testing. In some cases, a short period of pulverizing in the laboratory may be necessary.

RADIOGRAPHY

There have been numerous attempts over the past 40 years to apply X rays and gamma radiography for the nondestructive examination of concrete and concrete structures, especially in Europe (*99–101*). Although X rays and gamma rays differ only in their origin, the difference has considerable practical significance. X-ray equipment has a high initial cost, produces very high voltages, and is not portable, with the result that X rays offer little scope for use in the field (*68*).

The principle involved in radiographic methods is that as the radiation passes through a material of variable density, more radiation is absorbed by the denser parts of the material than the less dense parts. Most applications of radiographic techniques involve the transmission of wave energy rather than the reflection and refraction methods. Back-scatter techniques can be used when only one face of a member is accessible, although these are much less satisfactory than direct transmission.

Two experimental techniques, radiography and radiometry, are employed (*89*). In radiography, the emerging radiation is detected by a photographic emulsion, and variations in the density of the exposed film reflect the internal structure of the material under examination. In radiometry, variations in the gamma intensity are detected by radiation detectors, such as Geiger or scintillation counters.

Radiography with gamma rays from a cobalt 60 source has been investigated for detecting variations in consolidation in members up to about 18 in. (450 mm) thick, locating reinforcement, measuring the extent of corrosion, and assessing the quality of grouting in prestressing ducts (*89, 99, 100, 102, 103*). At thicknesses greater than 18 in. (450 mm), the long exposure needed makes the process uneconomical (*104*).

The major application for gamma radiography is for detecting voids in grouted ducts. Laboratory work in England (*100*) showed that radiography could detect voids in grout as small as $^3/_{16}$ in. (5 mm) in concrete beams 5 in. (125 mm) thick. It has also been used in the field on sections up to 16 in. (400 mm) thick (*57*). It has been suggested that radiography is more useful for determining the extent of a cavity once it has been found than it is for locating voids (*101*). The procedure is expensive and time consuming because each radiograph typically requires an exposure time of about one hour and covers an area approximately 12 in. × 16 in. (300 mm × 400 mm). Moreover, the use of radiography can generate strong opposition from the general public.

Most of the experience with radiography in the field has been in France, where it has been used since 1968 to locate the position of prestressing cables, detect defects in cables, and examine the quality of grout (*105*). The equipment has been developed into a system that makes possible the detailed inspection of cables in box, I-section, and slab bridges (*106*). The system is known as SCORPION (from radioSCOpie par Rayonnement Pour l'Inspection des Ouvrages en betoN, which can be translated as radiation radiography for the inspection of concrete structures). It consists of a radioactive source, a detector, and a remote command module, as illustrated in Figure 7. The detector includes a filter, a converter (which forms an optical replica of the incident radiation), a mirror, and a low-light-level camera. The source and detector are mounted on movable platforms, which can be operated by remote control from the command module. The command module also includes a television monitor, storage unit, and videotape recorder. A prototype unit was constructed in 1979 and has been found capable of examining concrete thickness up to 18 in. (450 mm) in field trials. In 1984, another unit was constructed. Known as SCORPION 2, the unit utilizes a linear accelerator as the source of radiation, giving the capability to examine concrete up to 4 ft (1.2 m) thick. A portable linear accelerator has also been used to examine a bridge in Hampshire in England (*82*).

Radiographic techniques are the most feasible method of detecting voids in grout and strands or wires that are broken or out of position. However, small amounts of corrosion, particularly if located perpendicular to the radiation, will not be detected.

OTHER POTENTIALLY USEFUL TECHNIQUES

Air Permeability

Air permeability measurements have been used in the United Kingdom as a method of assessing the quality of grout in post-tensioned cable ducts (*57*). The method involves drilling holes into the top of the duct. The duct is then evacuated at one hole and the amount of vacuum measured at the other holes in the same duct to determine whether voids are continuous along the duct. The volume of any voids present was measured by connecting the evacuated voids to a water gauge consisting of a narrow plastic tube dipping in water. The height to which the water was drawn up the tube was measured and the volume of voids calculated. Although the method is subject to errors because of leakage into the duct and possible presence of undetected voids, it does give an indication of the quality of grouting in ducts accessible by drilling. The method cannot be used to detect voids that are not continuous with the hole that is evacuated.

A refinement of the method was to measure the rate at which air could leak out of the ducts by pressurizing them with nitrogen gas and measuring the input flow rate required to maintain a pressure of 2.5 psi (17 kPa) above atmospheric pressure. When high flow rates were measured, a soap solution was applied to the exterior of the concrete so that points of leakage could be detected.

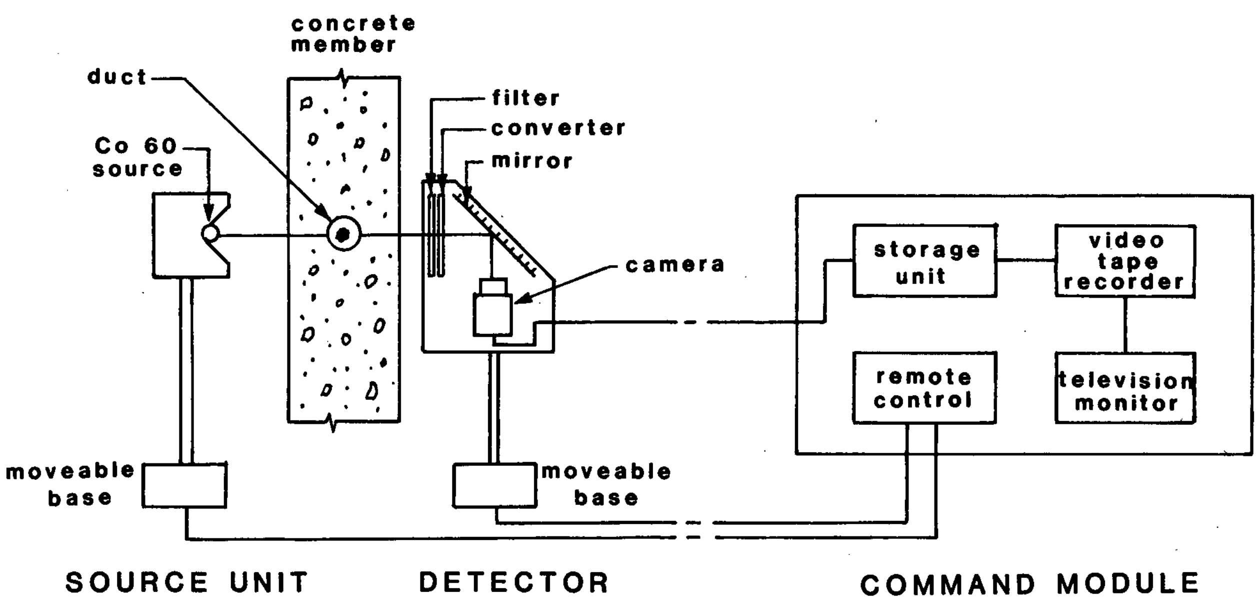


FIGURE 7 Elements of the SCORPION system.

Thermography

Infrared thermography has been found to be capable of detecting delaminations in bridge decks (*107–109*). It can also be used on other concrete elements, such as columns, provided that they are exposed directly to the sun. The method works on the principle that a discontinuity within the concrete and parallel to the surface, such as a delamination, interrupts the heat transfer through the concrete. This means that, in periods of heating, the surface temperature of delaminations is higher than that of the surrounding concrete. At night, when there is usually a loss of heat from the concrete to the surrounding air, the surface of the delamination is cooler than the average temperature of the solid concrete.

The main application to prestressed concrete components would appear to be for detecting delamination on components for which access for sounding methods is difficult; for example, on marine structures. However, preliminary studies would be needed to define the components that constituted valid application of thermography.

Thermography has also been used successfully to survey the soffits of a large number of prestressed concrete box girders in an elevated expressway to identify those girders that contained water (*110*).

In addition to constraints imposed by weather, the main disadvantage is that, although a positive result is valid, a negative result could indicate that the component is free from delaminations or that it contains delamination that could not be detected under the conditions prevailing at the time of test. Nevertheless, the method has the advantages that it is both quick and remote and that the equipment is readily available on a rental basis. Consequently, it could be an extremely useful technique at certain sites.

Radar

Investigations into the use of ground-penetrating radar for detecting deterioration in pavements and concrete bridge decks began in the mid 1970s (*111, 112*). These investigations were prompted by the development in the 1960s of low-power, high-frequency pulsed radar, which offered the resolution necessary to detect small flaws in concrete. A number of studies have been carried out on both bare decks and asphalt-covered bridge decks (*109, 113, 114*). In all cases, the radar was found to be capable of identifying anomalous areas in the deck, though there was a practical problem of analyzing large amounts of data and relating the different radar waveforms to physical distress in the deck.

Radar could be used on prestressed concrete components to locate delamination, voids in the concrete, and the position of embedded steel. It could not be used to detect voids inside metallic ducts, because the duct acts as a shield. There are a number of problems to be overcome, including access to vertical surfaces and soffits and the development of software for automated signal processing. Despite these difficulties, the problem of obtaining reliable information on the condition of prestressed concrete components is sufficiently great that pilot studies into the potential applications of radar would be worthwhile.

Acoustic Emission

Acoustic emission is the term applied to the low-frequency sounds emitted when a material is deformed (*115*). In its simplest form, the emission can be of such a high level that it is audible to the unaided ear. A familiar example is the cracking of timber when loaded to near failure. Most materials emit sounds or stress waves as they are deformed, and these sounds can provide information on the deformation characteristics of the material and warn of impending failure. However, the sounds are usually of such a low level that sophisticated instrumentation and signal-processing techniques are required to detect and analyze the minute perturbations.

Piezoelectric sensors attached to the concrete surface and the variations in the time of arrival of the stress waves at each sensor are used to locate the source of the deformation (*116*). Although the method has potential for such applications as in situ strength prediction and detection of crack growth and corrosion, further development is needed. It has also been used experimentally to detect stress corrosion on strands in a prestressed concrete beam (*117*). Laboratory tests were encouraging, but field testing proved inconclusive. An attempt to monitor cracks in the anchor block of a prestressed concrete bridge was also inconclusive (*67*).

The technique suffers from two serious limitations. First, it is not possible to distinguish between sources of emission that are detrimental to the structure and those that are not. Secondly, the emissions depend on the previous loading history of the structure, and with bridges this is not usually known. However, some of these difficulties are expected to be overcome as more experience is gained and data-processing methods are refined.

FULL-SCALE TESTING OF BRIDGES

Problems arise when the condition of the bridge as a whole needs to be known. Most test methods give only qualitative information about the condition of a component in the locality of the defect or deterioration. It is often very difficult to analyze the effect of the defect on the overall performance of the bridge or even on the capacity of individual components. In these circumstances, techniques that examine the overall condition of a bridge directly can be useful in either diagnosing defects or determining their significance. These techniques can be divided into two categories—load testing and surveillance techniques. Surveillance techniques include measurement of vibrational response and acoustic emission and optical methods for detecting changes in the geometry of a structure with time.

A chapter on the full-scale testing of bridges is contained in Synthesis 118 (*56*), and additional information is given in the references cited in the chapter.

LABORATORY PROCEDURES

Because it is difficult to assess the condition of prestressing steel in the structure, field observations often need to be supplemented by laboratory testing. This is particularly true when corrosion of the steel has occurred and it is necessary to determine the extent and type of corrosion and the actual properties of the steel.

Although it is relatively easy to remove cores from most concrete structures, prestressed structures are usually heavily reinforced. Not only must care be taken not to drill through prestressing steel, but cores must be secured that do not contain mild reinforcement, which is difficult. As noted in the field procedures section, samples of prestressing steel should only be removed under the supervision of a structural engineer.

The procedures used for testing samples of concrete in the laboratory are discussed in Synthesis 118 (*56*). Consequently, this section is limited to a discussion of the laboratory testing of the prestressing steel. Standard test methods applicable to prestressing steel are given in Table 3.

Metallographic Examination

A metallographic examination consists of a detailed study of the steel, including the identification of any defects present and a description of such properties as chemical composition, mechanical strain, and grain size. Numerous procedures can be involved, such as the preparation of polished or etched surfaces and the use of optical or scanning electron microscopes. These techniques have been described by ASTM (ASTM E 807), and the examination will normally be carried out by a specialized professional. The skilled practitioner can determine the degree and the type of corrosion, how long it was ongoing, whether the failure was normal, and the possible cause and can alert the engineer to any unusual features. Furthermore, the cost is modest, typically $500 to $1000 per strand. Unfortunately, there are very few metallurgists experienced in the examination of prestressing steel removed from concrete structures.

More specifically, failure surfaces, either on wires already broken when removed in the field, or on specimens tested in the laboratory, should be examined very carefully to determine the cause and nature of the failure. Of particular significance are the degree of ductility at failure and an examination of the grain boundaries to distinguish between intercrystalline and transcrystalline failure surfaces. Analysis of the metal for contaminants, particularly chlorides, sulfides, and nitrates, may permit a positive diagnosis of the cause of failure. Although brittle fractures can be identified, the circumstances that caused the electrolytic cell to develop in the structure may be difficult to establish. Pitting corrosion cannot be assumed to be the result of contamination. Although it is often found in steel that has been exposed to chlorides (either admixtures or penetrating the concrete from external sources), it may also occur because of atmospheric corrosion caused by improper storage, when tendons remained ungrouted for a long time or when tendons were grouted incompletely (*118*). In practice, it has been found that most failures in wires from highway bridges have been ductile failures.

Physical Testing

The two most significant mechanical properties of prestressing steel are the ultimate strength and the ductility. These can readily be measured in the laboratory and compared with the values measured on the steel at the time of construction (or with the specified values, if test certificates are not available).

TABLE 3

ASTM TEST METHODS FOR USE IN THE LABORATORY

Designation	Title
A 370	Methods and Definitions for Mechanical Testing of Steel Products
E 3	Methods of Preparation of Metallographic Specimens
E 30	Method for Chemical Analysis of Steel, Cast Iron, Open-Hearth Iron, and Wrought Iron
E 328	Recommended Practice for Stress-Relaxation Tests for Materials and Structures
E 340	Method for Macroetching Metals and Alloys
E 407	Method for Microetching Metals and Alloys
E 807	Practice for Metallographic Laboratory Evaluation
E 883	Practice for Metallographic Photomicrography

Any degradation in either of these properties has important ramifications in evaluating the structural capacity of the component from which the steel was removed. Pitting corrosion is, by its very nature, irregular. It is often very difficult to measure the cross-sectional area of sound steel remaining. Because information on the loss of section is needed primarily to determine the remaining strength of the steel, it is often better to measure the strength directly.

Ductility is important because the greater the ductility of the steel, the greater the resistance to brittle fracture. However, ductility is impaired by notch effects resulting from pitting corrosion. The degree of impairment on pitted steel is difficult to determine because of the need to use a very small gauge length. The standard gauge lengths of 10 in. (250 mm) and 24 in. (610 mm) for ASTM A 421 wire and A 416 strand respectively will give results that indicate an apparent brittle failure, but the test can be useful in assessing the effects of the corrosion on the performance of the structure.

When it is not practical to run relaxation tests for 1000 h, a short-term test for 30 min will often permit the qualitative identification of low-relaxation strand. A maximum relaxation of 1.2 percent has been suggested for low-relaxation strand stressed to 70 percent of the minimum guaranteed breaking strength and tested at 70°F (21°C) (*7*). This compares with a maximum of 2.5 percent in the 1000 h test. A difference between measured and specified values of relaxation would have some effect under service loads but very little effect at ultimate load.

Corrosion of the steel affects the different strength properties in different ways depending on the type and the extent of the corrosion. For example, pitting corrosion has a marked influence on ductility and on fatigue strength but less influence on tensile strength. The reduction in tensile strength is approximately proportional to the loss of area, whereas the reduction in fatigue strength and ductility is proportionally much greater. The influence of corrosion notches is relatively more important with steels of higher strength. When these effects are of particular concern to the structural engineer, the test program must be formulated accordingly. Similarly, creep and relaxation tests may be included when an accurate measurement of these properties is required.

It is difficult to generalize as to the type of physical testing and the number of specimens, because structural requirements are a function of the type of structure and the degree of deterioration. There may also be other constraints, such as practical limits on the number of samples available or budget limitations. In any event, the testing program should be under the direct supervision of a qualified structural engineer who can interpret the test results and, if necessary, modify the test program in the light of the initial results.

INTERPRETATION AND ASSESSMENT OF CONDITION SURVEYS

The inspection and evaluation of prestressed concrete structures is not a simple matter. Detecting corrosion on embedded prestressing steel and determining the effect of the corrosion on the affected component and on the structure as a whole is particularly difficult. Considerable engineering judgment is required to interpret the results of a condition and to assess the reliability and significance of the data. Consequently, the whole process of inspection, collection, and assessment of data has to be related to the effect of the deterioration on the overall structure in order to identify alternatives for future action.

Although it is important to emphasize the need for treating each structure uniquely, there are a few general principles that can be enunciated to assist in the interpretation of condition survey data. The first step is to diagnose the cause of any defects and deterioration present. In this respect it is useful to compare the results of two or more test procedures in order to increase the confidence in the interpretation or to identify anomalous results. For example, in structures exposed to chlorides in service, one would expect to find reasonable agreement between areas of delamination and spalling, low cover, active corrosion, and high chloride contents at the level of the steel. Similarly, areas of scaling would be expected to be related to the air-void system of the concrete and the degree of saturation. When there are clearly contradictions between the results of different tests, further fieldwork may be necessary.

Data on the strength or other properties of the steel and the concrete are used to assess the capacity of the structure. In doing so, it is necessary to consult the contract drawings for the structure and preferably the "as-built" drawings. The specifications and codes current at the time of construction will assist in giving nominal properties of the materials. The design calculations indicate the assumptions and concepts on which the design was based. It may be possible to use more sophisticated forms of analysis to calculate a capacity of the structure that is different from that indicated by the original design calculations. Site records and inspection reports may yield information on any unusual events that occurred during construction.

There will usually be some uncertainty surrounding the extent of the deterioration, and in such cases the formulation of several scenarios, including best case, worst case, and most probable, is often a valid approach. In developing the scenarios, an assessment needs to be made as to the likelihood of the deterioration continuing. It is also important to examine whether local deterioration of the prestressing steel could lead to local failure or even to collapse of the structure. Consequently, it is very important to determine whether the structure has a secondary load distribution system, in case of a local failure in the primary system.

In the best of circumstances, when the deterioration is found to be localized and the structure has considerable redundancy, it might be concluded that local repairs would be sufficient together with regular inspections to determine if there is any further deterioration. In the worst case, when there is significant deterioration in a critical component, then structural strengthening or modification would be required with some degree of urgency. In most cases, intermediate conditions are likely to be identified and the assessment of the data not as straightforward.

When cracking, rust staining, or spalling has occurred, the cause must be established. The results of ultrasonic tests can be very useful in locating internal cracks. For external cracks, measurement of crack movements is particularly useful in diagnosing their cause and significance. When corrosion is present, the areas of the structure affected can be assessed from half-cell potential data and, when chlorides are the cause, from chloride-ion profiles. The loss of section on the steel can only be determined by exposure of the steel, by radiography, or by the removal of the steel to the laboratory. Chloride profiles, potential measurements, and resistivity measurements can also be used to predict where future deterioration might occur in a structure and the approximate time frame. For example, if the chloride content is above the threshold value for corrosion, the concrete resistivity is low, and potential measurements are in the range -0.20 to -0.35 V (CSE) over much of the structure, then extensive corrosion can be expected within a short time. Conversely, if chlorides have not penetrated to the steel, the concrete resistivity is high, and the potentials are uniformly low, then several years of satisfactory performance can be anticipated.

The use of condition survey data to identify maintenance strategies and select a method of repair is discussed in more detail in Chapter Three.

SUMMATION

The need to define clearly the purpose of an investigation and the manner in which the results will be used is of the utmost importance, because these factors determine the scope of the work that must be accomplished. In practice, most investigations are severely limited with respect to both the financial and human resources available. A careful visual inspection by a trained observer is the most cost-effective activity. Given the realities under which most highway agencies must operate, supplementary physical testing is the exception rather than the norm.

When a structural evaluation is being made, the strength of the concrete needs to be determined, and this can be done by impact hammer readings, selective coring, or a combination of both methods. When corrosion is suspected, potential measurements on a small grid are usually the most useful of the several test methods available, because they are relatively quick, reliable, and inexpensive. However, measuring the potential of prestressing steel can be difficult because of the presence of other steel (for example, ducts, anchorages, and mild steel reinforcement). Exposure of the prestressing steel at carefully chosen locations is most desirable for both direct observation and for interpretation of the potential survey. If fractured or extensively corroded wires are located, a metallurgical examination is often a good investment, provided that a suitably qualified practitioner is available.

The other test procedures described in this chapter are more specialized. They are discussed in order to assist the engineer faced with a nonroutine investigation of a prestressed concrete bridge to choose procedures suitable for the structure under consideration, and not waste time with those that are not. Because none of the nondestructive methods are well suited to identifying corrosion, techniques that must still be considered experimental have been described. Even though cost and logistics preclude the use of some of these sophisticated techniques, it is important for both the practitioner and the researcher to be aware of the progress that has been made toward the nondestructive detection of corrosion on pretensioned and post-tensioned steel. In this respect, procedures based on pulse velocity and pulse echo appear to show the most promise for eventual development into portable, modestly priced equipment suitable for use in the field. The section on the interpretation and assessment of the results of condition surveys is intended to provide guidance in identifying anomalous results and in identifying alternative courses of future action.

CHAPTER THREE

MAINTENANCE AND REPAIR TECHNIQUES

INTRODUCTION

The rehabilitation of a deteriorated prestressed concrete bridge is, in common with most structural rehabilitation, a complex process of planning, design, and construction. There are three basic reasons why a structure might deteriorate:

i) Failure to appreciate the severity of the service environment
ii) Inadequate design
iii) Faulty construction

In every case it is necessary to establish the causes of the deterioration before a decision is made on the method of repair. If the structure has deteriorated because of a defect in construction—for example, failure to grout a tendon—then correction of the defect may be all that is required. If damage has occurred because of inadequate design, then strengthening of the structure will be required. If the structure has deteriorated because of environmental actions, either because of an incorrect assessment of the service conditions or because of poor-quality materials (for example, concrete with inadequate air entrainment), repair of the damage alone is generally not sufficient to prevent the deterioration from occurring again. In some cases it may be possible to eliminate or mitigate the causes—for example, by improving drainage or sealing joints.

The rehabilitation must result in a structure with improved durability, yet there are fewer options for selecting the protective treatments than there are for new construction. Add to this the operational constraints of traffic, weather, and budget, and it is easy to see why the rehabilitation of prestressed concrete bridges presents a significant engineering challenge.

This chapter discusses the factors that influence the selection of the rehabilitation scheme and the materials, procedures, and performance of the techniques available. It includes not only the rehabilitation of prestressed bridges but also the use of post-tensioning in the rehabilitation of reinforced concrete structures.

REPAIR STRATEGIES

The first decision to be made in planning and designing a rehabilitation scheme is whether individual components, or the entire structure, should be repaired or replaced. If the condition survey indicates deterioration that cannot be repaired (for example, cracking caused by reactive aggregates or serious corrosion-induced distress) or if replacement is more economical, then the best solution is likely to be to spend the least money to maintain the structure in a safe condition (or close the bridge to traffic) until it can be replaced. The cost of demolition must be included when calculating replacement costs. Demolishing prestressed bridges can be both technically difficult and expensive (*119*).

Most structures will not require early replacement, and the decision to repair immediately, undertake continued maintenance and repair at a later date, or undertake continued maintenance with a view to future replacement becomes much more complex. Once different planning horizons and different figures for the time-value of money are taken into account there are an almost infinite combination of maintenance, rehabilitation, and replacement strategies. No matter what course of action is taken, the decision must be technically sound for the particular structure under consideration. General, network-wide policies for methods of rehabilitation are rarely appropriate because they do not reflect adequately the complexities of individual structures and local conditions. However, condition and maintenance data in a network management system may be valuable in identifying problems common to structures of similar type and age. The most satisfactory means of developing the best solution for an individual structure is to follow a systematic decision-making process. This includes a condition survey of the structure to determine the causes and extent of the deterioration and a technical and financial analysis of the alternative strategies. Other factors, such as the functional adequacy of the structure and future construction activity in the geographical area, must also be taken into account.

Decision matrices for the rehabilitation of prestressed concrete components have not been developed in the same way as they have for reinforced concrete bridge decks (*120–122*), and criteria that lead to the selection of the most promising rehabilitation scheme are needed. Such decision models should include life-cycle cost data, and, inevitably, would be very crude until reliable cost and service-life data on each of the rehabilitative treatments are developed. The analogy with bridge deck repairs should not be surprising, because it is the systematic aspect of the decision-making process that is being advocated, and the principles apply to any type of rehabilitation. There are other analogies with the repair of decks. One that unfortunately is true is that it is common practice to repair a prestressed structure only when deterioration becomes apparent. In many cases this is already too late, and the implementation of preventive-maintenance practices and minor repairs may be significantly more cost-effective. It is for this reason that this chapter discusses both preventive-maintenance measures and rehabilitation techniques.

Much of the deterioration that occurs in highway structures results from the presence of chlorides in the service environment.

One of the most difficult problems in developing a repair strategy is to determine what to do about the chloride-contaminated concrete in the structure. At the present time, there is no economical method of removing chloride ions from concrete. Consequently, unless cathodic protection is applied, the only way to be sure of making a permanent repair is to remove all the concrete containing chlorides in excess of the corrosion threshold value, clean the steel to remove all contaminants, and prevent further ingress of chlorides. In many cases the chloride-contaminated concrete is physically sound, so that even when it is technically feasible to remove the concrete, it is prohibitively expensive. However, when chloride-contaminated concrete is left in place, there is always a risk that corrosion activity may continue or even accelerate. This risk is not always appreciated, with the result that repairs are often carried out in a piecemeal fashion without formulation of a plan for the future management of the structure. Such actions cannot be condoned.

There is one final caveat from the bridge deck field (*96*). Rarely will there be an ideal repair strategy, and the method selected will, almost invariably, be a compromise solution that is technically acceptable and economically feasible. In many cases, even after rehabilitation, the structure will be imperfect and will require regular inspections and preventive-maintenance treatments in order to perform satisfactorily.

INSPECTION AND PREVENTIVE MAINTENANCE

Because repairs are so difficult, this leaves maintenance as the most effective means of protecting the investment in prestressed concrete bridges. A program of regular inspection and preventive maintenance can be extremely cost-effective. Unfortunately, because of limited funding and the inevitable pressures to use the money that is available for activities that are perceived to be more urgent, such programs are rarely implemented. A strong argument can be advanced that not implementing such a program results in the wasting of money on subsequent repairs, many of which will be ineffective. However, the content of a routine inspection and preventive-maintenance policy needs to be carefully thought out in the light of each agency's needs and practices. General directives applying to all structures are of limited usefulness: The program should be tailored to several categories of type and age of bridges in order to maximize the benefits (*1*).

Inspection should focus on those parts of a structure most vulnerable to deterioration. Anchorage zones and beam ends located at joints in the deck slab need particular attention. Often these two categories coincide, in which case there is the possibility of leakage through the joint penetrating the anchorage and eventually along the tendons by capillary action, even where the ducts are properly grouted (*123*). Leakage through mortar plugs of anchors in the Walnut Lane Bridge in Philadelphia is suspected of causing corrosion of the prestressing steel, though this could not be checked (*42*). Deficient grout is not uncommon at the ends of draped tendons, which is also the location at which they are most susceptible to moisture and chlorides. The high stresses in anchorage zones also make these areas vulnerable to cracking (*67*). Unfortunately, these areas are also the most difficult to inspect, which is all the more reason for diligence and the accurate documentation of any defects observed so that changes in condition can be determined.

Preventive maintenance essentially consists of good housekeeping practices and is based on the philosophy that it is better to prevent a problem from developing, or to correct it in its early stages, than it is to wait until a serious problem has developed for which there may be no solution. In keeping with this philosophy, actions that eliminate or reduce the causes of damage, particularly the exposure to moisture and chlorides, should be undertaken. These include promptly repairing leaking joints; modifying drainage pipes that discharge onto beams, piers, or abutments; removing accumulations of sand and salt from winter maintenance operations; and improving ventilation and access in anchorage areas. Defective bearings, which could adversely affect stress distribution in the structure, should be repaired. The application of concrete sealers in critical locations is a good investment, providing that it is done before significant amounts of chloride have penetrated the member.

The importance of ensuring free drainage from box girders and the consequences of not doing so are discussed in Chapter Two.

REPAIR OF ACCIDENTAL DAMAGE

Prestressed concrete bridge members are often subject to accidental damage. The most frequent cause is overheight vehicles, though mishandling beams during manufacture and construction is not an infrequent occurrence. This type of damage was studied in NCHRP Project 12-21, "Evaluation of Damage and Methods of Repair for Prestressed Concrete Bridge Members." The project dealt with the repair of pretensioned beams and was carried out in two phases. Phase I synthesized available information and recommended procedures for assessing and selecting repair techniques. The findings were published in *NCHRP Report 226: Damage Evaluation and Repair Methods for Prestressed Concrete Bridge Members.* In Phase II, laboratory testing of the more promising techniques was undertaken and a manual of recommended practice prepared. The results were published in *NCHRP Report 280: Guidelines for Evaluation and Repair of Prestressed Concrete Bridge Members (124)*. The methods of repair that are discussed include epoxy injection, concrete patching, internal strand splicing, adding external post-tensioning, metal sleeve splicing, and girder replacement. In view of the recent publication of these comprehensive reports, the repair of accidental damage is not discussed further in this report.

Although at first it might appear that there are close similarities between the repair of accidental damage and the repair of environmentally induced deterioration, there are also some important differences. In some situations the need for emergency repairs may dictate the selection of the repair method. Furthermore, accidental damage is usually clearly defined and localized. Therefore, if the strength of the damaged member can be restored, there is every likelihood of a satisfactory repair and no further deterioration. Techniques that involve splicing fractured strand are not well suited to the repair of corrosion damage, because strand rarely corrodes at only one location. In most cases, several areas of several strands would be corroded, and numerous splices would be impractical. However, some of the techniques that are described can be used or modified for the repair of environmentally induced deterioration, and these are described elsewhere in this chapter. Design procedures for these various methods are given in Reference *124*.

PATCH REPAIRS

Patching is probably the most common method of repair, because it is perceived to be a quick, simple, and inexpensive solution. In some cases this is true. In others, particularly for the repair of corrosion damage, it may aggravate the problem, as will be discussed later.

Patching involves five basic steps:

1. The area to be patched is defined and a sharp edge, at least $^1/_2$ in. (12 mm) deep, is formed around the perimeter to prevent featheredging.
2. The concrete within the patch area is removed to a pre-established criterion.
3. The concrete surface and the exposed steel is cleaned.
4. A bonding agent is applied.
5. The repair material is placed and cured.

Extensive removal of concrete should not be undertaken from prestressed concrete structures without a thorough evaluation first being made of the structural implications of such actions.

Estimating the quantity of concrete to be removed in advance of a repair contract is not an easy task. Removing concrete from prestressed concrete structures, especially where access is difficult, such as when working over traffic or water, is very expensive. Substantial overruns in quantities are not uncommon. Overruns can make contract administration difficult and result in claims from the contractor. Payment is complicated by the fact that removal quantities are small and difficult to measure and unit costs are very high. Many payment schemes have been devised, including some form of "cost-plus" agreement and bid prices based on area, volume, volume of concrete replaced, or even equipment rental, in which the contractor bids an hourly rate for the provision of designated labor and equipment. In some situations, there is a separate pay item for providing access rather than including this cost in other bid items. This practice is common when the access cost, such as for the rental and erection of scaffolding, can be reasonably well established but the quantity of concrete to be removed is uncertain. In defining the pay items it is best to rely on local conditions and experience, while ensuring that the terms are clearly understood by all parties.

On bridge decks, large mechanical equipment such as scarifiers or "hydrodemolishers" can be used. Scarifiers are normally used to remove the top $^1/_4$ in. (6 mm) of deck surface and are followed by jackhammers above the level of the reinforcement and chipping hammers below the level of the reinforcement (*96*). Further restrictions are required when work is done around post-tensioning ducts, and hand tools may have to be used if the duct is not to be damaged. Hydrodemolishers are relatively new and use high-speed, high-pressure (up to 15,000 psi or 100 MPa) water jets. The "demolition" effect is obtained by a combination of three processes—direct impact, rapidly increasing pressure within the macro and micro cracks in the concrete, and cavitation. The amount of concrete removed is not easily controlled, especially in poor-quality concrete. The technology is attractive, but there are potential disadvantages, such as the possibility of inducing cracks to propagate. The need to dispose of large quantities of contaminated water also makes it difficult to satisfy environmental restrictions at many sites.

Concrete removal in other prestressed concrete components such as beams and piles must rely on small hand-operated equipment. This has traditionally been air-operated or electrically operated hammers, although small water-jetting equipment is becoming available. The key requirement is to provide a clean, sound concrete surface without damaging the exposed steel. Pretensioning strands are particularly vulnerable to being gouged or nicked. A rivet gun chipper is a useful tool for removing concrete from near strands without damaging them (*124*). Care is also needed around post-tensioning ducts in order to prevent the duct from being punctured or the grout shattered. The area should be shaped to prevent formation of pockets of entrapped air when the patch is placed.

Concrete should be removed until a sound surface is obtained. This may be readily defined during correction of accident damage or construction defects. When the concrete contains chlorides, the situation is much more complex; this is discussed later.

The final stage in preparing the existing concrete surface is cleaning the concrete and the exposed steel. Water blasting or sandblasting is normally used, although occasionally the steel is wire brushed. After completion of this operation, the concrete surface should be carefully inspected and any loose or cracked particles removed by hand chipping.

Many types of patching materials can be used, including conventional concrete, preplaced aggregate concrete (*125*), and polymer, usually epoxy, mortar. There is a multitude of commercial products available, many of which are rapid setting. Many of the products are, in fact, portland cement concretes with a high cement factor and, not infrequently, a chloride accelerator (*126*). If it is desired to place the patches without formwork, dry-pack concrete, pneumatically placed concrete (shotcrete), and some polymer mortars can be considered. High-range water reducers (superplasticizers) can be added to concrete to aid in its placement in heavily reinforced and confined areas; latex additions can be made to conventional concrete and shotcrete to improve bond and decrease permeability. Epoxy-modified shotcrete has also been used in Europe (*127*). In fact, there are so many variations of the materials that can be used for patching that the decision is often best made on the basis of local experience. If the materials and techniques are familiar to local personnel and are giving good performance, there is little incentive to change. The binder from the patch material (for example, neat epoxy if epoxy mortar is used or latex-modified cement slurry for latex-modified concrete) is normally applied to the existing concrete as a bonding agent. Where formwork must be used, use of a bonding agent may not be practical. In such cases it is important that the repair material have good wetting and workability properties to provide a good bond with the existing concrete. Latex-modified mixtures usually require that the existing concrete surface be wetted with water for a few hours, and when this is not practical they should not be used.

Shotcrete, which is pneumatically applied mortar or concrete, is often a feasible alternative to patching when access is difficult or forming costs are high—for example, in soffit repairs. Shotcrete can be applied by either the dry-mix process, in which water is introduced to the dry materials at the nozzle, or the wet-mix process, in which the materials are mixed and then blown through the nozzle. When properly proportioned and applied, shotcrete can provide a dense, strong repair. However, considerable experience, especially on the part of the nozzle operator, is required to produce the desired results of good compaction without voids or sand layers. The American Concrete Institute issues a guide for the certification of nozzle operators (*128*). Requirements for materials, equipment, procedures, and inspection are described in Reference *129* and specified in Reference *130*. In recent years, latex has been used to improve the adhesion and decrease the permeability of shotcrete. Steel fibers (without latex) have also been used and are claimed to improve finishability, reduce cracking, and increase tensile strength (*125*).

The dry-mix process produces better compaction, permits the use of longer hoses, and allows control of the consistency of the mixture during shooting. On the other hand, rebound is higher and a greater variation in quality can be expected. In the wet-mix process, the concrete is more uniform but strengths tend to be lower and it is difficult to interrupt shotcreting in mid-batch. The major drawbacks to the use of shotcrete are high mobilization costs and generally uneven appearance.

If the patch is to replace concrete that would normally be in compression, then the member should be preloaded to simulate the live load on the structure. If this is not done, the repaired member will crack under live load. Because patch materials are often stronger than the existing concrete, the crack will usually form at the bond line or in the existing concrete adjacent to the bond line. The preloading should not be removed until the patch attains the required strength. In practice, preloading is rarely used in conjunction with patching, though it is important to recognize the implications of not doing so. The technique is used more frequently in conjunction with the epoxy injection of cracks and is described in more detail in that section.

An interesting application of the use of patching was for the repair of deep honeycombing on the interior surfaces of a box girder in Germany (*131*). The prestressing ducts were exposed in the web of the girder as a result of the use of too stiff a concrete mixture and inadequate vibration. The method of repair is illustrated in Figure 8. The honeycombed concrete was removed and the top of the honeycombed area was cut out to

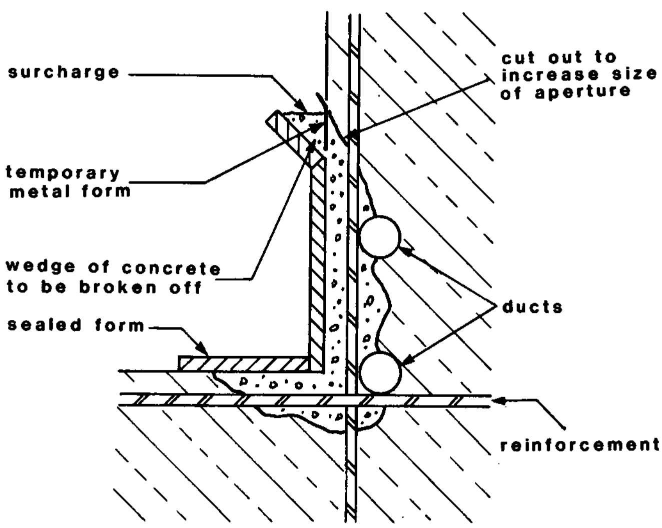


FIGURE 8 Example of patching to repair honeycombing in a box girder.

increase the aperture between the formwork and the web of the girder. A fluid concrete, containing a superplasticizer, was placed and consolidated using both internal and external vibration. A temporary metal plate was inserted flush with the web to facilitate the removal of the concrete used to surcharge the form.

Patch repairs can be used effectively for repairs when the cause of the damage is a "one-time" event such as accidental damage or honeycombing. They are a much less effective method of treating corrosion-induced damage, and will only provide a short-term extension of service life unless there is a commitment to continuing monitoring and further patching. In reality, patch repairs are common, and, given the resources available to most highway agencies for the maintenance of structures, there is often no practical alternative treatment.

If a member is patched, corrosion is likely to continue and may even be accelerated because the patches have a different oxygen and chloride content than the surrounding concrete so that strong corrosion cells may be established (*96, 126, 132, 133*). These corrosion cells may result in spalling of the patch itself or, more frequently, of the concrete around the patch. It is difficult to predict the effect of a patch on a chloride-contaminated member because it is a function of the change in potentials in the member as a result of the patching, the nature of the patch materials, and the future exposure conditions. If concrete is removed completely from around the reinforcement and replaced by conventional concrete, then the patch will be cathodic to the concrete surrounding the patch. The potential difference is often increased, and rapid corrosion adjacent to the patch occurs. If the steel in the patch area is only partially exposed so that it is partly embedded in chloride-contaminated concrete and partly in chloride-free patch material, a corrosion cell may be established within the patch area. If the patch contains chlorides, a strong anodic area may develop, and corrosion will occur in the patch soon after placement.

The use of a dielectric material, such as a polymer mortar, as the repair material should, in theory, diminish the possibility of developing concentration cells. However, cells can still develop if the repair is not impervious (because of either the selection of the material or workmanship) and because of potential differences along the steel, particularly if the steel was not properly cleaned. There is, in fact, no evidence that epoxy and polyester resins are any more durable than other materials (*134*).

Coating the steel exposed in the patch area is often advocated as a means of insulating the steel so that it cannot corrode or act cathodically to other steel. However, it is extremely difficult in practice to ensure that all chloride ions and all the corrosion products are removed from the steel in the cleaning operations so as to avoid continued corrosion beneath the coating. This is an extremely difficult task when exposed structural steelwork is painted and even more difficult when corroded reinforcement for which only part of the circumference is visible is cleaned. If the steel is not completely coated, corrosion cells may form at the small exposed areas, and corrosion is likely to occur at the edge of the patch because of the differing conditions along the surface of the bar (*92*). Further, because the rate of corrosion of steel in concrete is generally controlled by the reactions at the cathode, and most of the steel in a highway bridge is electrically connected, reducing the total cathodic area by the area of steel in a patch will have little effect on the overall rate of corrosion elsewhere in the structure. Consequently, an argument can be advanced that there is little, if anything, to be gained from coating the reinforcement with epoxy. The use of a portland cement slurry, which would create a strongly alkaline environment around the steel, and a portland cement-based repair material, which would be compatible with the existing concrete, is less expensive and at least as satisfactory.

Sealers have been applied over the entire surface of a component after patching in an attempt to prevent or reduce corrosion activity by reducing the availability of oxygen and moisture. Although the appearance of the component is often improved, there is no evidence to show that the life of the repair is extended. It is important to recognize that patch repairs cannot be used for the permanent repair of corrosion damage. Even when concrete is removed from around the steel, the concrete in adjacent, unrepaired areas remains contaminated. In bridge deck repairs it has been common to leave chloride-contaminated concrete in place, provided that it is physically sound, and such a practice is often cost-effective. Moreover, the consequences of continued corrosion activity on the service life of the deck can be predicted with reasonable accuracy and the safety of the structure is not compromised to an unacceptable degree. The situation is often quite different in prestressed concrete components where the effect of continuing corrosion on future performance and safety is far less predictable. Consequently, patching is not always an appropriate treatment for the repair of corrosion damage in prestressed components, though the statement must be tempered by recognizing that there is often no practical alternative. At best, patching will improve the appearance of the bridge, provide protection to the steel against atmospheric corrosion, and prevent continuing chloride ingress deep into the member, and these benefits may be sufficient to justify the cost. On the other hand, corrosion cells may be established that will accelerate corrosion in the member. Because it is difficult to predict the effectiveness of patch repairs, it is recommended that their performance be monitored regularly, preferably by both visual observation and sounding. Any additional patching should be done promptly until this method of repair is no longer feasible.

PERMANENT FORMWORK

One of the methods used to repair accidental damage on beams when several strands are severed and a substantial amount of concrete must be replaced is to fabricate a metal sleeve splice *(124)*. A typical splice is illustrated in Figure 9. This technique can also be used to repair other types of damage on beams and adapted for use on other members, such as columns, cap beams, and marine pilings. A further extension of the concept is to completely surround a component with new concrete (sometimes known as jacketing) and to leave the formwork in place to provide additional protection. This technique is quite common on marine pilings and has also been used on reinforced concrete columns.

When used to repair accidental damage, the concrete in the beam is first restored. The metal splice is custom fabricated to the shape of the beam, usually from galvanized ASTM A 36 metal not less than $^{5}/_{16}$ in. (8 mm) thick. Fabrication may be in the shop or the field, but at least one field weld is required in order to assemble the sleeve. The inside surfaces are scored

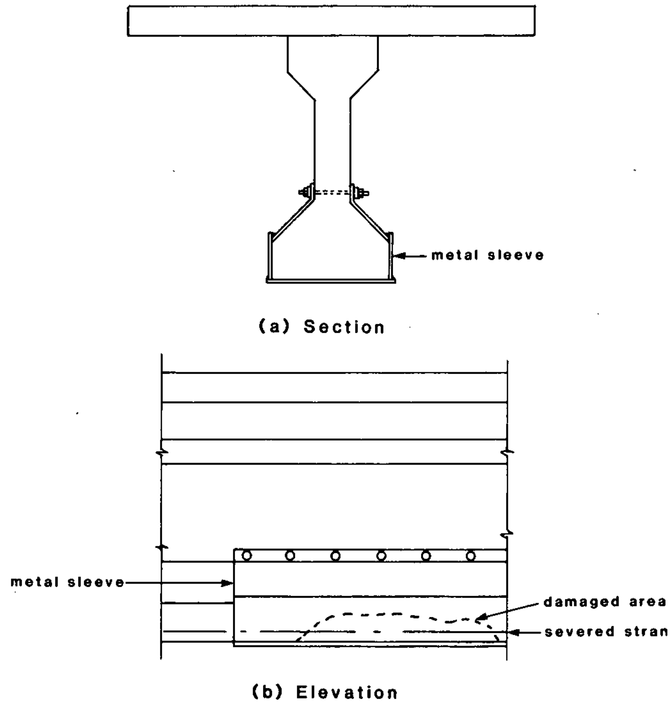


FIGURE 9 Metal sleeve splice.

by vertical brushing and $^{1}/_{16}$ in. (1.5 mm) spacers are attached before assembly. Field welds are painted with zinc-rich paint, and the entire splice is often painted "concrete gray" to improve its appearance. The splice is securely attached to the beam, usually by bolts passing through the web. Although the splice does not restore prestress, it is designed to have at least as much strength as any strands that were severed. Preloading is usually not necessary, because any hairline cracks are covered by the splice. The metal sleeve is bonded to the beam by the pressure injection of epoxy resin. The ends and top of the sleeve should be sealed with epoxy mortar, but inspection openings should be left to allow the progress of the grouting to be monitored. Several injection ports should be provided and injection started at the lowest point.

A metal sleeve splice for splicing concrete piles has also been tested *(124)*.

In jacketing repairs, the objective is not usually to provide external reinforcement but simply to replace deteriorated concrete. The formwork is not designed to increase the strength of the member and serves only to contain the repair material until it hardens. However, by selecting a form that is itself durable and impermeable, additional protection is provided. Fiberglass and different types of plastic forms are available. When the work is done by maintenance forces, it is common for materials already in hand, such as the plastic barrels used in energy-absorption devices, to be used. The forms are chosen to provide a gap of about 2 in. (50 mm) between the outside of the component and the inside of the form. On vertical elements, the bottom of the formwork is sealed with a gasket and the gap filled with concrete, usually superplasticized, with the use of an elephant trunk arrangement to avoid air pockets.

Although these techniques are relatively straightforward and can be very effective, the applications must be chosen with care. The techniques are essentially a means of constructing a large patch, and the advantages and disadvantages are the same as for patch repairs. Patch repairs are also well suited to damage resulting from "one-time" events, to repair of cavitation damage in marine pilings, or even as a means of repairing scaling damage. Permanent formwork is rarely a good solution for the repair of corrosion-induced damage.

CRACK INJECTION

The injection of epoxy resin as a method of repairing cracks has been in use for many years. The technique is well suited to the repair of structural cracks that are clean and that result from an overload condition that is unlikely to occur again. It can also be effective for shrinkage cracks once the cracks stop growing. It is not suitable for cracks in which dirt has accumulated, and should be used with caution on cracks that are filled with water. There is little point in injecting corrosion-induced cracks or active cracks (which move in response to live load or thermal effects), because both types are likely to reopen.

The material used for crack repair should be a low-viscosity epoxy resin capable of injection into and travel along a crack 0.002 in. (0.05 mm) wide. The cracks can be sealed on the surface in one of two ways. Fiber-reinforced resin can be glued to the concrete surface or, for a neater appearance, the cracks can be routed to a V-shaped cross section about 0.5 in. (12 mm) deep, along their entire length. After routing, the cracks are blown clean and sealed flush with the beam surface. Injection ports should be drilled with hollow bits with a vacuum attachment to remove the dust. The spacing of the injection ports should be selected to ensure that the cracks are fully injected. A useful rule of thumb is that the spacing should exceed the thickness of the member. Specific recommendations, such as a maximum spacing of 16 in. (400 mm) and a minimum of 6 in. (150 mm), have been suggested (*124*). The ports, which may be tire valves, copper tubing, or plastic fittings, are sealed in place. Several types of pumping equipment are available, and the best approach is often to use only companies having at least two years' experience in similar work. Pumping should begin at the low end of the crack and continue at each port in turn. When epoxy emerges from the adjacent port, the injection port should be sealed; the adjacent port then becomes the new injection port. This procedure is repeated until the crack is completely filled. Care is required when laminar cracks are injected to prevent spalling (*32*). The epoxy should be allowed to cure without disturbance for at least 24 h at 70°F (21°C), and longer if the temperature is lower. After curing, the valve stems should be flush. Where appearance is of importance, the sealing paste should also be ground flush and the entire beam coated with a pigmented sealer.

Repair of cracks by epoxy injection can be made underwater using methods similar to those described above, provided that an epoxy that is insensitive to water is used. The major difference in procedures is that the material used to seal the cracks before injection may take several days to harden (*135*).

Polymers other than epoxies have been tried for injecting delaminations in bridge decks (*136, 137*), but with mixed success. Several commercially available epoxy resins are available and have been used successfully for many years, so that there is little incentive to seek other materials.

One of the factors often overlooked when injecting cracks is that even though the beam is prestressed, the material in the crack is not precompressed. Consequently, some cracks experience tension under live load. Because the epoxy resin is normally stronger in tension than concrete, the crack usually occurs adjacent to the filled crack. This situation can be avoided by preloading the member so that the crack will not experience tensile forces in service. The preload can be applied by loaded trucks or by vertical jacking (*124*), but the preload must be maintained until the repaired cracks have gained the required strength.

CONCRETE SEALERS

Concrete sealers have been used in a number of different ways and with varying degrees of success. Generalizations are made difficult by the many different chemical types and the numerous chemical products. The following comments apply to those sealers that can be shown, on the basis of performance data, to be effective in resisting penetration by moisture and chlorides.

Concrete sealers are most effective when used as a postconstruction treatment to prevent deterioration rather than to correct a problem that has already developed. They can be very useful in preventing, or at least reducing, scaling damage in concrete that has an inadequate air void system or concrete placed late in the year so that it is immature when first exposed to freezing conditions. Sealers can also be effective in extending the time-to-corrosion of a member when applied before the chloride content has built up to threshold levels.

One of the most common uses of the sealers, but also one of the most questionable, is their application to corroding members, often after patching, with the intention of slowing down the corrosion processes by limiting the availability of oxygen and moisture (*138*). Field data and documentation of the long-term effects of this approach have not been found. The treatment is sometimes justified on the basis that it is inexpensive and cannot do any harm. However, this hardly constitutes a valid justification for its use, and performance data are badly needed. Sealers can have a negative effect if applied when the moisture content of the concrete is high, and can actually increase the moisture content if applied to one face of a member when the opposite face is exposed to moisture. Such situations are not common but can exist, for example, in a concrete abutment or a box girder in which there is leakage into the void.

Pigmented sealers are sometimes used to improve the appearance of a member that has been repaired by patching or epoxy injection. Such a use is legitimate, providing that the sealer is not also expected to solve the corrosion problem.

A further use of concrete sealers has been in an attempt to slow down the alkali-aggregate reactivity in pretensioned beams. The purpose of the sealer is to reduce the moisture content of the concrete and so slow down the reactivity and extend the service life of the beams.

A further discussion of concrete sealers, but in the context of their use in new construction, is contained in Chapter Four.

CATHODIC PROTECTION

The first known application of cathodic protection to a bridge structure was in California in 1958, when an experimental system was applied to the concrete beams in one of the structures crossing San Francisco Bay (*139*). The system appeared to be functioning effectively when the structure was replaced one year later. Further development in California led in 1973 to the first cathodically protected bridge deck (*136*). Over the next decade, cathodic protection systems were installed by only a few agencies (*140, 141*), and the total number of installations on bridges was small. More recently, there has been a renewed interest in cathodic protection with the introduction of several new anode materials into the marketplace and research studies to develop techniques for cathodically protecting substructure components (*142–144*). The major reason for the increasing attention being given to cathodic protection is that it is now generally accepted that it is the only repair technique that will stop active corrosion in existing structures.

The theory of cathodic protection is to apply sufficient direct current to the surface of the reinforcing steel to prevent it from discharging ions so that corrosion does not occur. The steel becomes cathodic with respect to an external anode; hence the term cathodic protection.

Cathodic protection has been widely used to prevent corrosion in such applications as pipelines, chemical plants, oil refineries, and underground storage tanks. Although the principles of cathodic protection are common to all applications, reinforced concrete structures present particular problems:

i) Concrete has a high resistivity.

ii) Concrete is attacked by acid, which may be generated at the anode.

iii) The potential of the steel must be maintained within a narrow range. If it is too low, corrosion can occur. If it is too high, the concrete around the reinforcing steel may deteriorate, anode life is shortened, and hydrogen may be evolved at the cathode.

The key to the successful cathodic protection of steel in concrete is to provide uniform current density to the reinforcement, thereby maintaining an even potential distribution. The resistance between the anodes and all the reinforcement must be kept uniformly low, and the anodes should be as large as possible to minimize the applied current density.

There are two methods of applying current to the structure: by using galvanic anodes or by an impressed current source. The source of impressed current may be a battery or a rectifier that converts alternating current line power to direct current. For galvanic anodes to function, a metal that is sacrificial to iron must be used as the anode material. Most investigations have used zinc anodes. The limiting factor in sacrificial anode systems is their low driving voltage. Although studies have indicated that galvanic cathodic protection systems are possible (*145, 146*), other work has indicated that insufficient protection is provided (*142*).

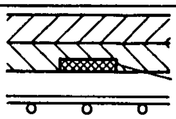
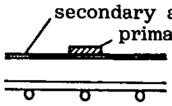
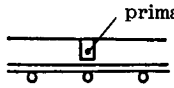
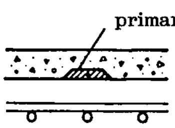
There are several different impressed current systems available, and it is convenient to recognize four generic groups:

i) Conductive overlays
ii) Conductive coatings
iii) Slot systems
iv) Distributed anodes (usually in a nonconductive overlay)

The success of any particular system is determined by the extent to which the requirements for a low uniform current density using durable components are compromised. The relative advantages and disadvantages of the generic systems are contained in Table 4. In addition, there are a number of proprietary systems in the marketplace and new product lines are introduced frequently into what has become a rapidly developing field. The most common of the proprietary systems could be classified in the distributed anode group and consist of line or mesh anodes embedded in a mortar or concrete overcoat.

TABLE 4

CHARACTERISTICS OF GENERIC CATHODIC PROTECTION SYSTEMS

Type	Features	Advantages	Disadvantages
Conductive Overlay	 wearing course conductive overlay primary anode (sometimes recessed)	Large secondary anode Low current density Good performance history Secondary anode dissipates gases	Suitable only for decks Additional dead load May cause concrete scaling if air-void system inadequate
Conductive Coating	 secondary anode primary anode	Low cost for conductive paints Easily installed Many coatings available Large secondary anode Low current density	Not suitable for decks Long-term durability questionable Circuit resistance varies seasonally Sometimes unattractive
Slot System	 primary anode	Low cost Surface profile unchanged	High current density Non-uniform current density Anode may touch rebar during installation Long-term durability questionable
Distributed Anode	 primary anode concrete overlay (non-conductive)	Overlay protects anode and reduces seasonal effects Well suited to badly spalled members Overlay can be used as a riding surface on decks Dissipates gases	Tends to be expensive Non-uniform current distribution

The cathodic protection of prestressed concrete components presents two additional problems. Hydrogen is generated at the cathode of an electrochemical cell when the potential of the cathode is depressed to the hydrogen evolution potential. The value of the potential at which hydrogen is formed is a function of the pH of the environment at the cathode. Thus the risk of hydrogen embrittlement is increased as the steel is cathodically protected. The main problem facing the practitioner is to determine whether it is safe to apply cathodic protection to prestressed concrete bridges. The secondary problem is to determine whether it is feasible to cathodically protect post-tensioned tendons because of the shielding effect of the duct.

These problems have not been widely addressed. One study (*147*) suggested that, because of the danger of embrittlement, it would be prudent to polarize the steel no more than the minimum necessary to arrest corrosion (i.e., without any factor of safety). A later investigation suggested that, although the cathodic protection of prestressed concrete pipe indicates that there is no inherent incompatibility between prestressing steel and cathodic protection, the applicability and design of cathodic protection systems must be evaluated specifically for each structure (*148*).

A laboratory investigation in Canada (*149*) concluded that cathodically protecting the mild steel reinforcement in a thick, post-tensioned deck slab would not adversely affect the post-tensioning tendons under normal operating conditions. However, the tendons would not be protected in a deck where the tendons and metal ducts were in electrical contact, which is the usual case. Cathodically protecting the mild steel reinforcement in thin slab decks supported on pretensioned girders was also judged unlikely to have any adverse effects on the girders, providing the applied current density did not exceed 2 mA/sq ft (20 mA/m^2). The study recommended that caution be exercised in any attempt to cathodically protect the prestressing steel in pretensioned beams. It was found extremely difficult to obtain a uniform current distribution, which not only could leave some of the reinforcement unprotected, it also could result in macro cell action between protected and unprotected steel, as well as hydrogen evolution in the higher current zones.

In a study carried out at Florida Atlantic University (*150*), a series of experiments was performed on a single tendon in a prestressed concrete slab. The procedure involved impressing an anodic current until active potentials were recorded along the length of the tendon. Subsequently the midsection of the tendon was cathodically polarized for a period of 36 days. The tendon was then removed and cut into sections. Each section was notched and strained to failure in a three-point bending test to determine any effect of cathodic protection on the mechanical properties of the steel. It was concluded that, under the conditions of the experiment, the cathodic protection did not affect the mechanical properties of the strand adversely or enhance its susceptibility to brittle fracture.

Although the cathodic protection of pretensioned strand may be technically possible, further research is required to show that this can be done safely over a long period of time. The feasibility of using cathodic protection to protect post-tensioned steel in existing structures is less promising, but cathodically protecting the mild steel reinforcement in such structures appears to be feasible.

REGROUTING DUCTS

When voids are discovered in grouted ducts, they should be regrouted wherever possible. In practice this is extremely difficult to do, because an injection port must be provided at one end and a venting port at the other end of each void in order to ensure that air and any water in the duct are displaced by grout. It is much easier to correct very poor grouting when the voids are continuous from one end of the duct to the other than it is to regrout a few isolated voids. In many cases, grouting of small voids—resulting from excessive bleeding, for example—is not practical. The materials and procedures are otherwise essentially the same as those described in Chapter Four for new construction.

A modification to conventional grouting procedures sometimes used in regrouting is to inject the grout under vacuum rather than pressure. The air is evacuated from the void at approximately 95 percent vacuum, and the quantity of air extracted is a measure of the volume of the cavity. The grout is then drawn into the vacuum, and by finishing with a positive pressure, it is believed that 99 percent of the volume of the void can be filled with grout (*101*).

Epoxy resin was used to inject incompletely grouted ducts in the Walnut Lane Bridge in Philadelphia when it was repaired during the winter of 1968–69 (*151*). Water was discovered in the ducts during a contract to repair cracks by epoxy injection. Although many commercial epoxies are claimed to be resistant to moisture, the presence of water reduces the effectiveness of the epoxy. It is difficult to ensure in this kind of repair that all the water is displaced by the epoxy and that the tendons are fully protected.

Polymer grouts have also been used in Europe to inject ducts in which voids were discovered. In one case (*152*) in the United Kingdom, voids were discovered, as a result of rust-staining, in a four-year-old bridge that had been grouted with a neat cement grout. The structure, a pedestrian underpass of a railroad, was regrouted with polyester resin using a vacuum grouting procedure. In another case (*153*), resin was used to inject voids thought to have originated because of the segregation of neat cement grout.

In 1974, extensive corrosion because of inadequate grouting was discovered in a 21-year-old viaduct in Stockholm (*154*). Major repair work was undertaken in 1975–76, including the injection of the cable ducts with epoxy grout. Holes were drilled into the ducts and compressed air was used to establish that the voids were continuous between adjacent holes. During injection, compressed air was blown into adjacent ducts to ensure that the epoxy did not flow into these ducts and block them. Where such flow occurred, the ducts were injected simultaneously. The amount of epoxy used amounted to 52.5 percent of the theoretical volume if the ducts had not been grouted. The costs were extremely high. Total repair costs were Kr 1.2 million, of which 0.8 million was for injecting the cables, compared with Kr 2 million (or approximately $300,000 at 1987 exchange rates) for the demolition and replacement of the structure.

METHODS OF STRENGTHENING

When the strength of a bridge must be increased to regain satisfactory ultimate resistance and serviceability performance or to correct design inadequacies, it may be feasible to provide additional reinforcement. Methods that have been used on bridges include:

i) Gluing or clamping of steel plates to the soffits and sides of the weakened member.

ii) Additional prestressing, or prestressing added to reinforced concrete structures.

iii) Replacement of corroded, unbonded tendons.

Other methods of strengthening that can be considered (*59*), but for which no documentation of use in bridges has been found, include:

iv) Addition of in situ concrete to the affected member to increase the capacity of the section. The new concrete must bond with the existing concrete to give composite action. One disadvantage of this method is that the dead load is increased.

v) Clamping of precast concrete members to the weakened unit. The additional members may be on the top, the soffit, or the sides, depending upon the available access. Short bolts attaching each precast member (or passing through the weakened member) are prestressed so that composite action is induced by friction across the contacting surfaces. Composite action and durability can be improved by grouting between the precast and the existing concrete.

vi) Where suitable conditions exist, induction of additional prestress by flat jacks in the suspect member. The effect of the force on other parts of the structure must, of course, also be considered.

The actual method adopted is often dictated by the particular circumstances in each case.

Bonded External Reinforcement

The use of bonded external reinforcement has been investigated in the United Kingdom, where it has been used in at least three major bridge-strengthening schemes in England (*155, 156*). Four bridges were repaired in 1975 and two in 1977. In 1982, steel plates were used to strengthen a prestressed box-girder bridge (*157*). In all cases, in order to increase live load capacity, steel plates were bonded with epoxy to the soffits of beams that were exhibiting cracking. The method has the advantage of minimizing both the interference with traffic and the reduction in vertical clearance. Because the new reinforcement can only be stressed under live load, it is necessary to determine the dead load stresses on the existing section and the live load stresses on the rebuilt section. The stresses must then be added algebraically to check that the original section is not overstressed.

Extensive laboratory testing was undertaken in which it was found that when the bond strength was fully developed, the strength of the epoxy joint exceeded that of the concrete, which failed in horizontal shear (*158*). The ultimate capacity of the beams was virtually unchanged, but stiffness increased significantly, cracking was reduced, and capacity under serviceability conditions was increased. The bond strength was found to be affected by the method of construction and was reduced by atmospheric contamination of the steel surface before bonding and by entrapped solvents from adhesive primers. In tests designed to simulate traffic-induced movements of the beam while the plate was being applied, bond strength was reduced when a stiff adhesive was used. There was no reduction when a more flexible resin system was used.

Long-term outdoor exposure tests were carried out on small-scale concrete beams to investigate the durability of the composite system and particularly of the steel-epoxy interface. Beams that were removed for evaluation after two years' exposure exhibited varying amounts of corrosion of steel that had been in contact with the adhesive. An average of 20 percent of the surface area had debonded (*159*). The corrosion was attributed to moisture penetrating the steel-resin interface. The 10-year exposure test results are not yet available, though the initial results give cause for concern for the long-term durability of the technique.

The application of external steel plates is also reported to have been used extensively in France and Japan (*158*).

A similar technique has been used successfully in Germany for the repair of concrete box girders (*138*). Steel plates were bonded to the interior surfaces, thereby reducing the severity of the service environment. A further advantage of this approach is that, because long-term performance is uncertain, the plates could be reapplied if necessary in comparative safety. This assumes that any debonding was identified before the performance of the girder was affected adversely. It was recommended that the surface of the concrete be cleaned but that roughening was unnecessary. The steel plates should be sandblasted and the exposed surfaces given suitable corrosion protection. Bonding adhesives are commercially available and specifically formulated for use in different temperature ranges.

Possible modifications to the method of using steel plates have been suggested (*156*). Attaching and shimming the steel plate followed by epoxy injection may simplify installation, especially where the concrete surface is uneven. Other reinforcing materials such as carbon strand might also be considered.

Tendon Replacement

The replacement of tendons is rarely an option in highway structures because of the overwhelming preference for bonded construction and the fact that the anchorages are rarely accessible. The technique is quite common in the nuclear industry and in the private sector. Because the clearance between the strand and duct is often very small, it is sometimes possible to replace the affected tendon by one of smaller cross section. Sometimes a higher-strength steel may be used to permit the same load to be applied; alternatively, a smaller prestressing force may be acceptable.

In the case of an externally post-tensioned bridge in the United Kingdom, all 120 prestressing cables were replaced eight years after the bridge was constructed. Numerous wire fractures were recorded beginning almost immediately after construction. Several studies into the cause of the fractures were made, but the results were not conclusive (*160, 161*). The replacement cost was approximately the same as the original cost of the structure (*49*). Removing the old strands proved to be difficult because, as the strands were cut, they became wedged in the saddles positioned within the diaphragms over each pier. A special jack was required to pull the strands through the saddles. The new tendons were encased in concrete.

Additional Prestressing

The effectiveness of external post-tensioning for the repair of accidental damage was demonstrated in Phase II of NCHRP Project 12-21 (*124*). External post-tensioning was used to restore loss of strength because of the severing of 4 out of a total of 16 strands in a pretensioned beam. It was also used to demonstrate how to increase the load capacity of a girder that does not have severed strands. External post-tensioning has been used to repair accidental damage in pretensioned beams in South Carolina, Connecticut, and Washington. The method consists of attaching jacking corbels or bosses to the girder and using high-strength rods or prestressing strands for the external post-tensioning. The corbel is normally attached to the sloping face of the bottom flange and the bottom of the web by expansion bolts or insertion of reinforcement into holes drilled for the purpose. The strength of the jacking corbels will generally control the amount of prestress that can be added. Typical details using prestressing strand are shown in Figure 10. In addition to the methods of corrosion protection shown, the corbel can be cast the full length between the jacking corbels after stressing, and this may, in fact, be the most satisfactory method of providing long-term protection. Although these techniques were developed for the repair of accidental damage, the concepts are equally applicable to the repair of other forms of damage. However, the amount of prestress must be carefully designed to provide sufficient capacity at the damaged section without overstressing sections that are already precompressed.

For some structures, the use of internal tendons or bars is a technique well suited to the control of unanticipated cracking. Additional prestress was applied through the anchor block and mid-span deflector diaphragm of a segmental bridge in the United Kingdom to prevent further crack growth (*67, 160*). The bridge developed hairline cracks six months after being opened, and the cracks continued to grow until, three years after construction, the bridge had to be closed for repair. Temporary repairs consisting of vertical prestressing were made to control cracking of the anchorage blocks in the worst condition. Subsequently, permanent repairs were made by the addition of both vertical and horizontal prestressing bars at the anchorage blocks and the mid-span diaphragm.

Examples of the use of this technique on a box-girder bridge are shown in Figure 11. Grouting of cracks should precede prestressing so that the compression being applied can be transferred through the member. The external profile can be straight or polygonal, depending on the nature of the defect. The straight tendon is easier to install but is not very efficient and has little effect on shear capacity. The polygonal profile is more satisfactory from the structural point of view but requires the construction of deviation blocks or saddles, as shown in Figure 11(c). In both methods the tendon should be attached rigidly to the structure to preclude buckling of the girder and prevent resonance in the tendon if the period of the structure is near that of the tendon (*32*).

One approach is to construct a massive reinforced or prestressed concrete distribution member across the end of the girder to avoid concentrated local stresses and to transfer the

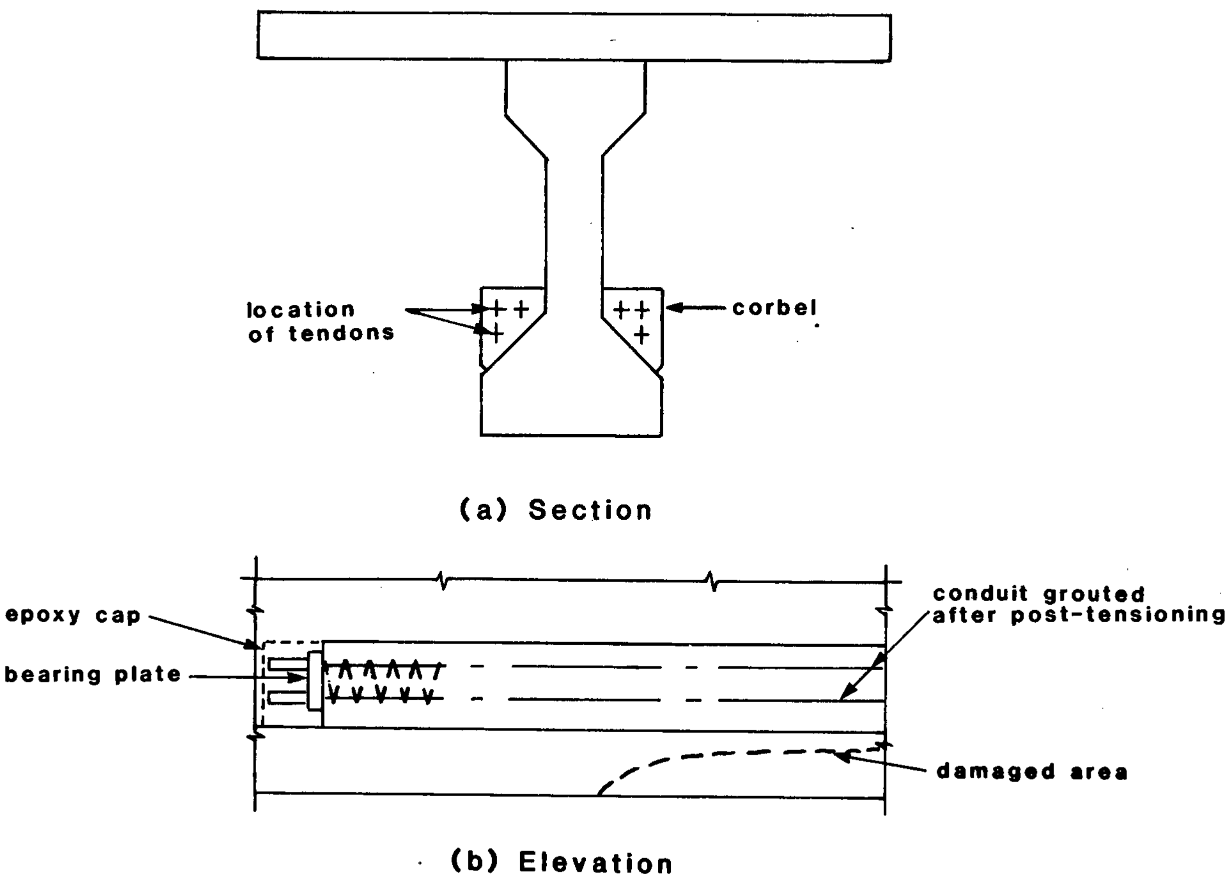


FIGURE 10 Example of a post-tensioned splice using strand.

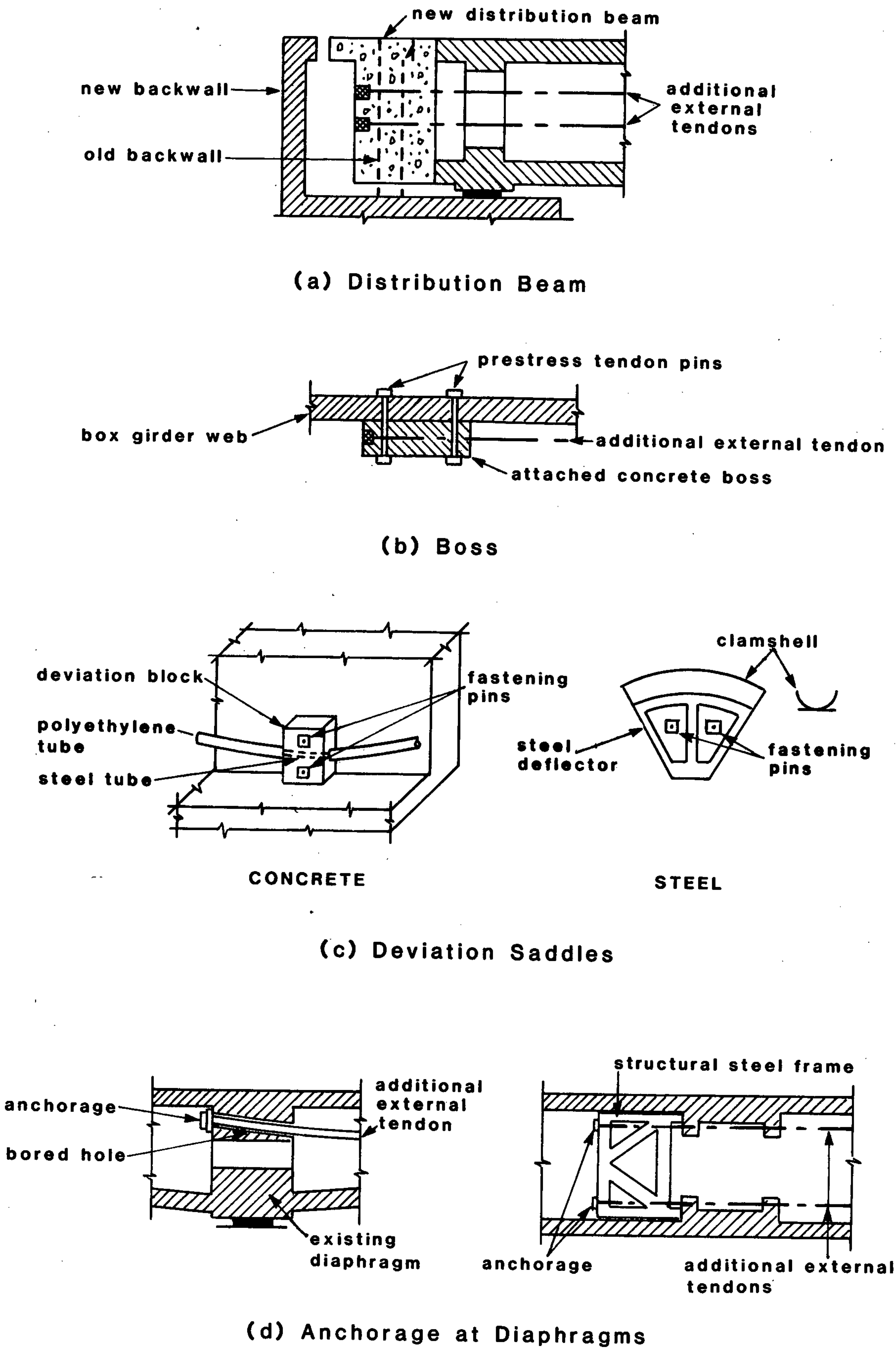


FIGURE 11 Examples of additional external tendons in box girders.

forces to the girder. The disadvantage of this method is that the existing abutment backwall must be removed and reconstructed as shown in Figure 11(a).

An alternative is to anchor the longitudinal tendons in a corbel attached by prestressing steel to the web as shown in Figure 11(b). This method is similar to that shown for I-girders in Figure 10, but has the disadvantage of inducing high local stresses in the web behind the corbel. It is also sensitive to losses in the short tendons, which could result in sliding of the corbel.

Tendons may pass through existing diaphragms, or, if they have sufficient strength, may be anchored as shown in Figure 11(d). Depending on the diaphragm detail, it may be necessary to provide a structural steel frame to transfer the longitudinal prestressing force.

The repair of shear cracks can sometimes be accomplished by adding vertical prestress to the web after injecting cracks, as shown in Figure 12. Although satisfactory from a theoretical standpoint, the approach has a number of practical disadvantages. In the case of the single tendon shown in Figure 12(a), the drawbacks are:

i) It is difficult to protect the anchorage near the roadway surface from corrosion.

ii) Because of the losses associated with short tendons, it is difficult to obtain the theoretical prestressing force, although the losses are less with bars than with strand.

iii) Boring through the web is extremely difficult.

If dual tendons are used, as shown in Figure 12(b), the boring problem is minimized but the problems associated with roadway anchorages and short tendons remain. In addition, there is the danger of punching shear through the flange and high secondary bending stresses in the flanges and web. The bending of webs is more likely to be serious in external webs, and thickening the web may be necessary before prestressing to avoid use of bars on the exterior of the girder (*162*).

One of the most extensive applications of additional prestressing was made to an 18-year-old bridge in Dusseldorf, West Germany (*163*). Cracks occurred because of restraint at the bearings, and this in turn led to problems of overstress, fatigue, and tendon fracture. Temporary supports were erected and the cracks injected with epoxy. Three tendons were added to the

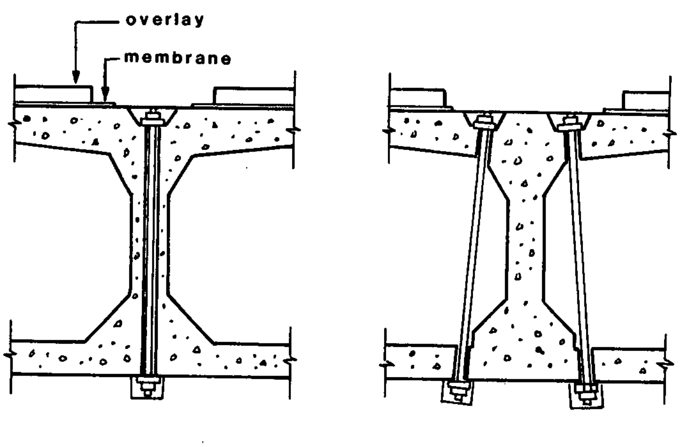


FIGURE 12 Retrofit shear tendons.

outer surfaces of the single-cell box girder and embedded in concrete 10 in. (250 mm) wide, 2 ft 4 in. (700 mm) deep, and extending the full length of the structure. The concrete was required not only for corrosion protection but also to carry the radial forces because the structure was curved in plan. Additional prestressing was installed at anchorage points and at large cracks. A steel plate was used to replace the tensile capacity of the broken tendons.

The rehabilitation and strengthening of the Boivre Viaduct near Poitiers in France (*164*) incorporated a number of innovative treatments. These included the use of stainless steel bars for vertical and horizontal prestressing, galvanized tendons for the full length of the 965 ft (294 m) long structure, and a wax-based material to protect the tendons from corrosion. Stainless steel was used for the external bars because standard prestressing steel installed previously had corroded. The stainless selected was a low-carbon austenitic steel containing 18 percent chromium and 14 percent nickel. The ultimate strength of 142 ksi (980 MPa) was achieved by nitrogen hardening. Special anchorages for the bars had to be developed. The 10 longitudinal tendons were installed in fiberglass ducts within the voids of the box girders. Although the tendons were galvanized, a wax-based grout was used. Special equipment for injecting the grout had to be developed, and both the material and the application techniques have been patented. The rehabilitation was completed in 1984 at a cost of $1.8 million.

Use of Prestressing in the Rehabilitation and Strengthening of Reinforced Concrete Components

The use of prestressing steel is often the only feasible method of rehabilitating or strengthening a reinforced concrete component. The technique has been used to strengthen reinforced concrete girders (*165*) and piers (*7*) and in conjunction with the replacement of floor beams and deck slabs.

There are a number of examples of the use of external tendons or bars to strengthen cross-head beams in pier bents. The force in the tendons is usually transmitted either by anchor plates bearing on the ends of the member or by anchorages clamped to the sides of the member by preloaded bolts passing through it. In some situations it may be possible to deflect the external tendons using a saddle clamped to the sides or soffit of the member to increase the midspan eccentricity. One such example was to strengthen a deteriorated crossbeam of one of the column bents in a section of the Gardiner Expressway in Toronto. Concrete was removed by chipping, and tendons were embedded in the sides of the crossbeam. The anchor plates were attached to the ends of the beam. After stressing, the prestressing system was protected by the application of shotcrete. External prestressing was also used to strengthen five cracked cantilever piers on the Route 695 Bridge over Route 151 in Baltimore (*7*). End plates were fabricated for each pier cap suitable for anchoring five post-tensioning bars on each face.

Precast, post-tensioned floor beams, and a post-tensioned deck, were used to repair and strengthen three open-spandrel, concrete arch bridges in Pittsburgh during the period from late 1981 to early 1983 (*18*). The rehabilitation consisted of replacing specific floor beams and columns, floor beam cantilevers, spandrel beams, and the deck. Construction was carried out in two stages while traffic was maintained at all times except during post-tensioning operations. The floor beams were precast in two pieces and post-tensioned together. The first-stage portion of the beam was temporarily supported before erection of the second stage. The floor beams were prestressed to the columns by high-strength bars. The precast deck panels were 80 in. wide, 30 ft long, and 10 in. thick (2.05 × 9.14 × 0.25 m), and transversely post-tensioned using high-strength bars to provide moment and shear transfer across the longitudinal joints between the panels (*166*).

Precast panels, usually incorporating prestressing in the panel or for joining the panels together, have been used in the replacement of bridge decks in Indiana, Illinois, New York, Pennsylvania, and California and by the New York State Thruway Authority, the Pennsylvania Turnpike Commission, and the Massachusetts Turnpike Authority, among others. The use of precast modular deck construction has the potential benefits of a reduction in the interruption to traffic, greater structural efficiency, increased load-carrying capacity without a commensurate increase in the dead load, and increased safety during construction, depending on the site conditions. The design and installation of precast panels has progressed from their use on short structures with simple geometry in the late 1960s to installations such as the Woodrow Wilson Bridge in 1983, which required lightweight concrete panels, sliding bearing surfaces to accommodate post-tensioning both transversely and longitudinally, and that all work be done at night. Case histories, typical sizes, details for leveling devices, and examples of methods of providing composite action and shear transfer between panels are given in References *16, 17, 19, 167,* and *168.*

SUMMATION

The rehabilitation of deteriorated, prestressed concrete bridges is a complex process that must be approached in a systematic manner on a case-by-case basis. Following a condition survey, the cause of the deterioration must first be determined. The selection of the method of repair, or the decision to replace, should be made by analyzing the technical and the financial implications of alternative treatments for a planning horizon appropriate to the future requirements for the structure.

The repair of damage resulting from "one-time" events such as accidental damage or construction defects can usually be repaired satisfactorily using such techniques as crack injection, patching, or jacketing. When structural capacity must be restored or increased this can be done by adding prestressing steel, metal splices, or steel plates.

The repair of environmentally induced damage, and particularly corrosion damage, is much more difficult and there is less assurance that a "permanent" repair will result. Much of the deterioration that occurs in highway structures results from the presence of chlorides in the service environment, and a program of regular inspection and preventive maintenance that reduces the degree of exposure to moisture and chlorides can be very cost-effective. Where patching is used to repair corrosion damage, there is the risk that corrosion will continue, though often there is no practical alternative treatment. It is therefore recommended that the performance of patch repairs be monitored periodically and additional patching be done as necessary. Cathodic protection is a feasible method of arresting the corrosion of mild steel reinforcement, but further research is required before its use can be considered for the protection of prestressing steel.

Several examples of the use of additional prestressing to strengthen structures are presented in the chapter. Prestressing has also been used to correct defects and increase the capacity of reinforced concrete structures. Specialized procedures are also available, such as filling voids in ducts by regrouting them with resin using vacuum-injection techniques.

CHAPTER FOUR

METHODS OF ENSURING DURABILITY IN NEW CONSTRUCTION

INTRODUCTION

Most prestressed concrete components of highway structures are difficult to inspect and repair. Consequently, a high priority must be given to the development of design criteria, the selection of materials, and the quality of construction to prevent the deterioration of prestressed concrete components. Fortunately, there are a number of methods of improving the durability of prestressed concrete, and several can be combined to provide redundancy. The methods of ensuring durability can be conveniently divided into two approaches: those that improve the quality of the concrete and those that are specifically designed to protect the prestressing steel and associated hardware against corrosion. This chapter will emphasize those techniques in the latter category, which includes measures to protect the prestressing steel before encapsulation in grout or concrete, grout formulations designed to prevent corrosion, coated strand, and coated or nonmetallic ducts and hardware.

Although a general treatment of the durability of portland cement concrete is beyond the scope of this report, its importance should not be diminished. A thorough review of the subject is available elsewhere—for example, in the reports prepared by committees of the American Concrete Institute (*31, 169*). The concern with the effects of corrosion in prestressed concrete components has tended to focus attention on measures for corrosion protection. However, it must not be forgotten that the concrete, the conventional reinforcing steel, the prestressing steel, and the ducts must be durable for the component to be durable. In other words, there is a danger of becoming preoccupied with corrosion protection only to find that the concrete deteriorates through such mechanisms as freezing and thawing, alkali-aggregate reactivity, or sulfate attack. This means that the ingredients of the concrete mixture must be selected carefully and properly proportioned to ensure that the concrete will be durable in the service environment. The goal is to construct a dense, low-porosity concrete through the use of a low-water-cement-ratio mixture, proper compaction, and curing. Such concretes not only have intrinsic high strength but are resistant to carbonation and penetration by aggressive ions. Furthermore, they have a higher resistivity, which reduces the rate of corrosion, should corrosion of the embedded steel begin. Finely divided mineral additions, such as good-quality fly ashes and silica fume, have been found effective in reducing the permeability and increasing the resistivity of portland cement concrete.

DESIGN PRACTICES

The design provisions for prestressed concrete components of highway bridges in the United States are contained in the AASHTO Standard Specifications for Highway Bridges. The purpose of any design specification is to ensure that a structure will be safe, durable, and economical. In recent years, increased emphasis has been placed on the inspectability and replaceability of individual components of structures, and several states have modified the AASHTO specifications, particularly for the design of segmental bridges. This has resulted not only in the provision of positive corrosion-protection methods but in design concepts such as external tendons within box girders and provision for the addition or replacement of tendons during the service life of the structure.

Design Details

The first line of defense against deterioration should always be to avoid design details that contribute to poor durability. Because moisture plays an important role in the dominant mechanisms of deterioration of reinforced and prestressed concrete (corrosion, frost action, aggregate reactivity, and sulfate attack), the structure must be designed to minimize those parts of the structure exposed to moisture and to avoid details that could cause the concrete to remain wet or cause the accumulation of moist debris. This requires a detailed design of the drainage of the structure. In those areas where the deck is exposed to deicing salts, a sufficient number of drains must be provided to prevent water ponding on the deck surface. The point of discharge of the drains must be considered to prevent the discharge of runoff on beams and substructure components. Deck joints will normally be sealed to prevent runoff through the joint because the ends of pretensioned beams and the anchorage zones of post-tensioned members are particularly vulnerable to deterioration. The durability of the structure in the region of the deck joint and bearing seats can also be improved by providing adequate ventilation between the ends of the beams and the abutment and by placing bearings on pedestals to avoid horizontal surfaces on which debris can accumulate. Other design details, such as the provision of effective drip checks, are easily overlooked but are extremely important in influencing the long-term performance of a highway structure.

An example of the application of the above principles in practice is illustrated in Figure 13. Most of the details are applicable to all structures and are not specific to prestressed concrete (*170*). In the case of the example given, the pretensioned strand is not coated but the mild steel stirrups within 9 ft (3 m) of a deck joint are epoxy-coated to prevent corrosion. Prestressing strands with debonding sleeves should be avoided at

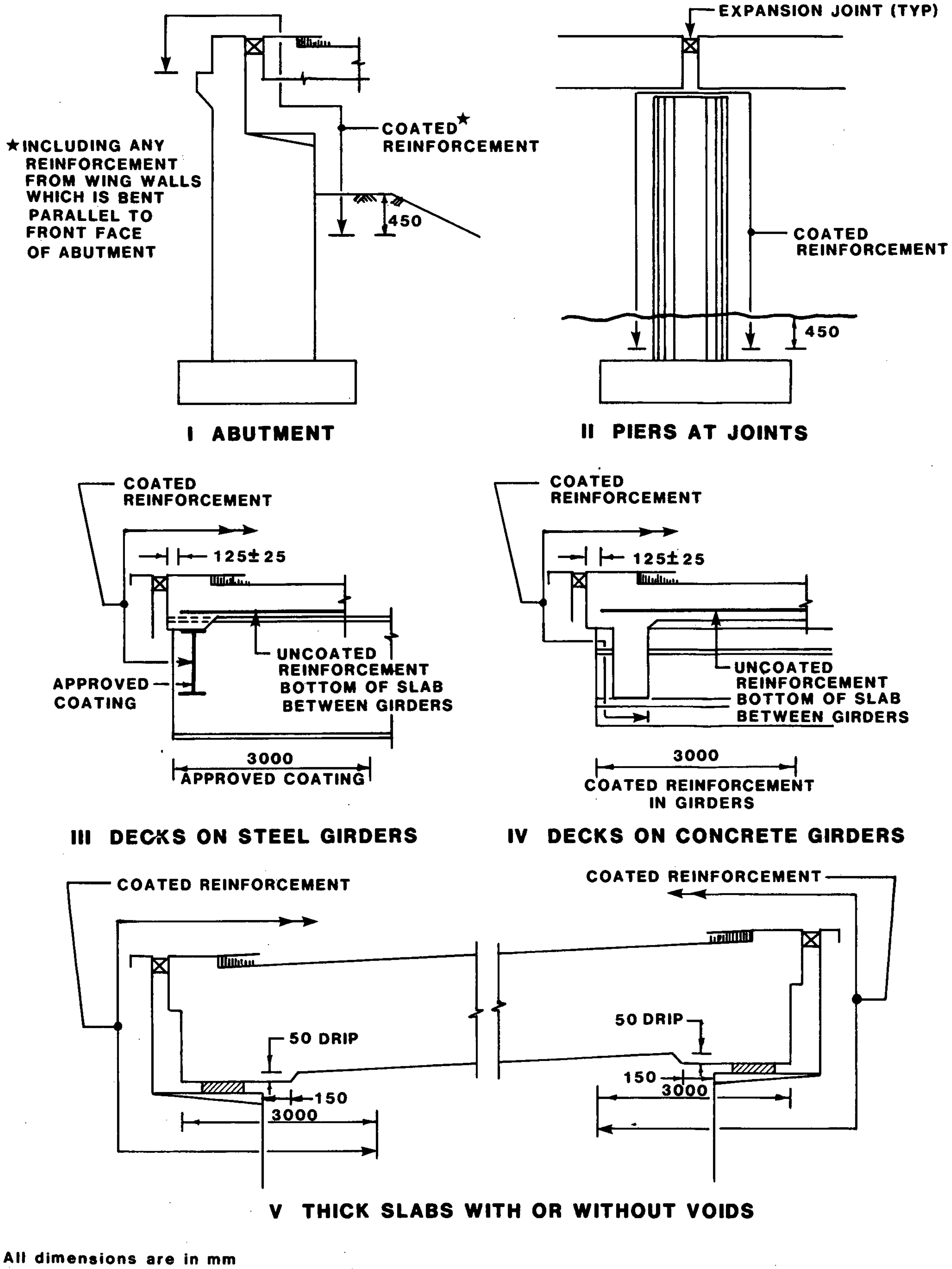


FIGURE 13 Examples of design details to improve corrosion protection.

expansion joint locations. Similar design details can be developed by each agency to take local conditions into account.

Concrete Cover, Water-Cement Ratio, and Control of Chlorides

The second line of defense has traditionally been to maximize the protection provided to both the mild and high-strength reinforcement by the concrete through the use of increased cover and a low water-cement ratio in the concrete. The role of concrete cover in protecting reinforcing steel from corrosion has been recognized for many years. Provisions for increasing the cover in corrosive environments have been in existence for about 50 years, and the resurgence of interest in the corrosion of bridge decks in the 1970s spawned research that quantified the relative roles of cover and water-cement ratio (*171*). Other studies (*172, 173*) have demonstrated the importance of both the quality and the quantity of the concrete cover in providing protection against corrosion.

From the standpoint of design, the increase in concrete cover has practical limitations. Dead load is increased for little or no increase in the capacity of the section, cracking of the unreinforced concrete may occur because of shrinkage or temperature effects, and surface crack widths are also increased. Furthermore, it has now been recognized that in the most severe exposed conditions, such as a highway bridge in an area where deicing salts are used or in a marine environment, concrete will not provide protection against corrosion throughout the life of the structure and additional protection must be provided.

Although the opportunities to provide large covers for conventional reinforcement are limited, it is often practical to provide substantial cover to the main longitudinal tendons, especially in segmental and slab bridges. The specifications of the Pennsylvania Department of Transportation for segmental bridges require that the longitudinal post-tensioning ducts be at least 10 in. (250 mm) below the top of the deck slab. In some cases, designers have placed the tendons entirely within the hollow core of a bridge. The use of external tendons is discussed in more detail elsewhere in this chapter.

Control of the mixture ingredients, and particularly the chloride content, is extremely important. ACI Committee 222 recommends that for prestressed concrete, the acid-soluble chloride content of the concrete be limited to no more than 0.08 percent of the mass of the cement in order to minimize the risk of chloride-induced corrosion (*169*). Higher permissible values have been suggested (*174*), but it would appear prudent to take a conservative approach for highway structures, most of which are exposed to chloride in service. It is advisable to maintain the lowest possible chloride levels in the mixture ingredients to maximize the service life of the structure before the chloride threshold level is reached and a high risk of corrosion develops. Consequently, chlorides should not intentionally be added to the mixture ingredients, even if the chloride content is less than the limit given above (*169*).

Other Approaches

As an insurance against the corrosion of tendons, as well as reflecting a trend to provide greater flexibility for future needs, the design criteria for some projects require that provision be made for the installation of additional tendons after the structure has been placed in service. Providing for 10 percent of the design longitudinal prestressing force has been suggested (*32*). This includes providing anchor blocks, tendon saddles, and holes for the tendons in diaphragms. The cost of these items is relatively minor during initial construction but can be expensive and very difficult as a retrofit measure. This approach is not without its disadvantages, as the provision of voids in a concrete member can have serious ramifications if water enters the voids (*175*). Because temporary grouting of vacant ducts is not practical, a decision has to be made when the unused ducts are to be permanently grouted, and this negates the ability to provide increased structural capacity to meet future needs. Consequently, the provision of additional prestressing is most useful for overcoming problems that arise during or shortly after construction, such as blocked ducts or unanticipated cracking.

Corrosion protection also can be provided by systems more tolerant of corrosion damage. The substitution of high-strength bar for high-strength wire results in more than 50 percent more steel area for the same prescribed final tensioning force as a result of the lower strength of prestressing bar (150–160 ksi or 1.03–1.10 GPa) compared with strand or wire (240–270 ksi or 1.65–1.86 GPa). The additional area means that the bar can tolerate more loss of section before breaking than the equivalent strand or wire. Further, because the bars have a much larger diameter than the individual wires or the wires composing prestressing strand (and hence have a much smaller surface-to-volume ratio), they are also much less susceptible to substantial strength reduction from pitting corrosion. Examples of bridges using this approach are the Kishwaukee River Bridge on Route 42 in Illinois and the Muskegon River Bridge on Route 131 in Michigan (*176*).

Design for Constructability

Designs that are easy to construct and that recognize the site conditions and the sophistication of the contractor increase the likelihood of a durable structure. Practical items, such as the spacing between pretensioning strand to allow for proper compaction of the concrete, and the relative diameters of a strand, wires or bars, and the duct to permit grouting, are covered by the design specifications. However, other practical details, such as ensuring that tendons have the minimum possible changes in direction and that details for duct venting and draining are provided, are the responsibility of the designer. It can be difficult to maintain the correct cable profile when flexible ducts are specified. Semi-rigid ducts are generally preferred for long-span bridges (*34*). On complex structures it is prudent for the designer to provide for grouting trials, which should be conducted so that the results can be verified by postmortem examination of simulated sections or by nondestructive testing of the actual structure by such techniques as radiography (*59*).

A further important role of the designer in the construction of a durable structure is to specify the quality-assurance measures and the appropriate records that are needed at all stages of the construction.

Unbonded Tendons

In bridge construction, unbonded tendons are used less frequently than bonded tendons because of the difficulty of ensuring adequate corrosion protection, because of the reduction in ultimate strength capacity, because of the susceptibility to catastrophic failure, and because wide cracks might develop under overloads unless beams are adequately reinforced with additional bonded steel (*177*). Unbonded tendons are widely used in buildings because of their economy, but there have been a significant number of cases in which unbonded tendons failed because of insufficient corrosion protection. Unless the anchorage is properly sealed against penetration by damp air, moisture can condense between the center wire and the circumferential wires of a strand (*22*). The water will then flow to the lowest part of the strand, where corrosion is initiated and spontaneous rupture may result. Although grease is the material most commonly used to fill the space between the tendon and the sheathing, most greases are not impervious to moisture, and aggressive substances can migrate through the grease and come into contact with the steel (*178*). Special greases that contain corrosion inhibitors are commercially available.

Unbonded tendons can be conveniently divided into those that are embedded in the concrete and those that are external to the member. Unbonded, embedded tendons are not well suited to the hostile service environment of most highway bridges, and their use is not recommended.

External tendons have been used in highway bridges, especially inside box girders, where the tendons can be inspected (and the risk of corrosion is diminished) and where the greater ease of construction has resulted in a more economical structure. The Long Key Bridge in Florida is perhaps the best-known example of the use of external tendons in a bridge in North America. The primary longitudinal tendons were encased in polyethylene ducts in the box-girder void (*179*). A cementitious grout was used. A draft of the specification being developed under NCHRP Project 20-7 Task 32 "Design and Construction Specifications for Segmental Concrete Bridges" requires that external polyethylene duct be made from high-density polyethylene conforming to ASTM D 3350 and that the ratio of the minimum external diameter to the wall thickness not exceed 21. External tendons have also been used in strengthening and rehabilitating a number of bridge substructure and superstructure components in which they may represent the only feasible structural solution. Nevertheless, the use of external tendons needs to be considered carefully and, as noted in the failures cited in Chapter Three, particular attention must be given to providing adequate protection against corrosion.

CONSTRUCTION PRACTICES

The corrosion of strand or wire in post-tensioned concrete can be substantially affected by its condition during shipping and storage and by the conditions between the time it is placed in the duct and grouted.

Protection of Prestressing Steel During Shipping and Storage

The strength of prestressing wire and strand is as sensitive to corrosion before placement as it is during its service life. Corrosive environments encountered during shipping and storage may cause pitting corrosion, which, if undetected, could severely impair its strength and ductility. Cases have been reported (*34*) in which prestressing steel failed after tensioning because the wires were exposed to sea spray during storage or because of long delays between tensioning and grouting. It is therefore necessary to protect the steel adequately against corrosion from the time it is manufactured until it is encapsulated in concrete grout.

Prestressing steel is generally packaged in a waterproof wrapping for shipping. When specified, a vapor phase inhibitor is incorporated in the packaging. A commonly used vapor phase inhibitor consists of dicyclo-ammonium nitrite crystals, which sublime to create a vapor that prevents corrosion by forming an insoluble ferric oxide coating on the steel. Even when a vapor phase inhibitor is used, the prestressing steel is vulnerable to corrosion because of tearing of the paper wrapping during unloading and storage in the open.

Several recommendations have been developed for minimizing the risk of contamination and corrosion of the steel during storage and handling (*34, 59, 176*):

i) storage time should be kept to a minimum,
ii) prestressing steel should be stored in dry conditions, indoors if possible and avoiding direct contact with the ground,
iii) the diameter of the coils should be large enough to prevent a permanent set in the tendon,
iv) when site storage is unavoidable, all the prestressing material should be coated with a rust-preventive oil.

If the prestressing steel must be stored on site for a week or more:

i) the storage area should be well-drained,
ii) the steel should be stored on metal or concrete supports and never be dragged on the ground,
iii) a waterproof cover should be provided, but not in direct contact with the steel, and installed to ensure adequate ventilation,
iv) the steel should be inspected regularly.

Temporary protection can be provided by coating with a corrosion inhibitor. The inhibitor ions are typically adsorbed on the metal surface and repel or neutralize corrosive ions that would otherwise attack the substrate. In the process, they may be consumed or neutralized themselves.

Two temporary coatings, a water-soluble oil and a film produced by sodium silicate solution in water, were investigated in an early NCHRP study (*3*). The water-soluble oil was found to offer superior corrosion protection, but a residual film was left on the steel after rinsing with water and this film impaired the steel-concrete bond. The presence of the sodium silicate film or its residue did not adversely affect the bond. Although the study concluded that the sodium silicate coating was the most suitable temporary coating available, an emulsifiable (so called "water-soluble") oil has historically been the only type of temporary coating used (*176*). This product has been used in Europe and North America for more than 15 years, though improved emulsifiable oils are being developed. Water-soluble oil has also been found useful as a lubricant when long tendons are threaded through a duct (*180*). In this case, the oil also acts to reduce the friction factor during stressing. Other materials that appear suitable for use as temporary coatings on prestressing steel have been identified but are not in common use (*176*).

Protection in Ungrouted Ducts

Although strands should be grouted soon after placement in the ducts, this is rarely practical. Sometimes the strand is placed in the duct before placement of concrete. The structure cannot be stressed until the required concrete strength has been developed, and grouting is often done several days later. Consequently, the prestressing steel remains ungrouted in the ducts for a period of 10 to 20 days or more. For segmental bridges it can be much longer. Rarely is any form of temporary protection used. Grouting should not take place immediately after stressing because the stress in each tendon requires time to equalize (typically 6 to 24 hours), depending on friction (*181*).

Long periods during which tendons remain ungrouted are more likely to occur in cold climates where structures may be stressed in the fall but cannot be grouted until the following spring. An investigation of the corrosion risk resulting from this practice was conducted by the Swedish National Road Administration during the period 1969–1972 (*182*). Five different corrosion-protection methods were applied to a total of 26 ungrouted tendons at three sites. After three years the tendons were removed and tested. Careful sealing of the ducts combined with drains at the low points of the duct resulted in acceptable corrosion protection. The protection was improved if there was a continuous flow of dried air through the ducts. A further improvement was the use of a vapor phase inhibitor in the ducts. An emulsifiable oil also produced good results, though this approach may impair the bond between the steel and the grout (*183*). The only method found to be impractical was filling the ducts with nitrogen, because gas leakage occurred.

The use of a vapor phase inhibitor to protect ungrouted strands in a French nuclear reactor was not satisfactory, and some wires fractured two years after stressing (*184*). This experience emphasizes the temporary nature of the protection provided by vapor phase inhibitors and the need for periodic replenishment.

During construction of the CN Tower in Toronto in 1973–75, concrete placement continued throughout the year, though grouting was suspended during the winter months. The ungrouted strands were protected against corrosion by the pumping of sufficient dry air at ambient temperatures to provide three complete air changes per hour (*185*). Examination of the visible portion of the strands before grouting indicated no corrosion had occurred.

Quality Control on Site

The construction of prestressed concrete structures on site includes a number of relatively sophisticated processes that require that the contractor exercise stringent quality-control procedures in order to produce a durable structure. A working party of the Concrete Society (United Kingdom), set up to report on the deterioration of tendons in prestressed concrete, included the following recommendations (*59*):

- ensure that supervisors and personnel are suitably qualified and trained and are aware of the need for vigilance to prevent deterioration or corrosion of prestressing materials
- check all material inspection certificates and regularly examine quality-control records
- inspect tendons and any special protective coating to make sure that they are not damaged
- make sure that the concrete is correctly proportioned and cured
- ensure that only materials approved by the specifications are used
- ensure that the specified quality of grout is used and the grouting technique has been approved by prior trial
- have flushing equipment available in case grouting is interrupted
- check to see that dimensional tolerances are maintained.

SPECIALTY CONCRETES

As noted in the section "Design Details," the traditional approach to the durability of prestressed concrete components has been to specify a durable concrete (low water-cement ratio, properly compacted and cured; air entrained where necessary) and relatively large amounts of concrete cover as a means of protecting embedded steel against corrosion. Although it is good engineering practice to maximize the protection afforded by the concrete, normal portland cement concrete will not provide sufficient long-term protection in the most severe service environments. There are, however, additives that can be incorporated in the concrete mixture that will increase the degree of corrosion protection provided.

In general, any ingredient that decreases the permeability of the concrete and increases its resistivity will improve corrosion performance in two ways. The decrease in permeability slows the penetration of aggressive ions and the increase in resistivity means that, should corrosion occur, the rate of corrosion is, in most cases, reduced. There are also secondary benefits to the reduction in permeability, such as the lowering of the moisture available within the concrete and an increase in strength to resist cracking. Mineral additions, such as pozzolans, fly ash, or silica fume, or supplementary cementitious materials, such as blast furnace slag cement, can be very useful in decreasing permeability and increasing the resistivity of the concrete.

The use of condensed silica fume, which is a by-product in the production of silicon and ferro-silicon alloys, has increased substantially during the 1980s. However, because it is a by-product, it is available in limited quantities. Condensed silica fume particles are spherical in shape, and about 100 times finer than portland cement or fly ash particles. The material has superior pozzolanic properties, permitting the production of high-strength, low-permeability concretes (*186, 187*). The silica fume is typically added at 5 to 15 percent by mass of cement and is used in conjunction with a high-range water-reducing admixture. An important property with respect to corrosion protection is that water-saturated cement products containing silica fume typically have electrical resistivities about two orders of magnitude greater than comparable products without it. Although the use of silica fume is increasing rapidly, it does not appear to have been used in significant quantities in prestressed concrete construction.

The use of blended cements (comprising ordinary portland cement and blast furnace slag cement in ratios up to 50 percent slag cement) has also been found to increase the resistivity of the concrete and to decrease corrosion (*188, 189*). Earlier studies (*190*) found that there was no difference in the susceptibility to corrosion of prestressing wires in slag cement concrete and portland cement concrete. However, it is now known that corrosion resistance is appreciably affected by the fineness of the slag cement (*191, 192*). The corrosion resistance increases with fineness of the slag cement.

High-range water reducers can also be used to reduce permeability by allowing a very low water-cement ratio to be used. This is especially true with precast products, in which the use of the admixtures is more easily controlled than it is in in situ construction.

An alternative approach to increasing the degree of corrosion protection afforded by the concrete is to use corrosion-inhibiting admixtures. Although there are numerous candidate chemicals (*193, 194*), only calcium nitrite has been used in significant quantities. Calcium nitrite is an anodic corrosion-inhibiting admixture in which the nitrite ion is believed to oxidize the ferrous ions to form an insoluble ferric coating on the steel surface (*195*). It has been shown to be effective in reducing the corrosion of uncoated steel in chloride-contaminated concrete in laboratory and exposure-plot tests (*196–198*). As the chloride content increases, the ability of a given dosage of calcium nitrite to maintain complete passivity is reduced because the ratio of nitrite is slowly consumed. Consequently, the dosage of calcium nitrite is extremely important. It was originally suggested (*197*) that Cl^-/NO_2 ratios of up to 1.25 would increase protection levels at least 10 times on the basis of rate of corrosion measurements in slabs stored outdoors. The manufacturer's recommendations for dosage are based on the Cl^-/NO_2 ratio not exceeding 1.5 during the life of the structure. In a more recent laboratory investigation (*199*), it was found that calcium nitrite did not significantly delay the initiation of corrosion, but that the rate of corrosion on both mild steel reinforcement and prestressing strand was reduced substantially. This led to the conclusion that calcium nitrite can provide significant protection to prestressing strand, but that the quality of the concrete (water-cement ratio 0.32 to 0.44) and the design cover (not less than $2^1/_4$ in. or 57 mm) should not be reduced. The cost of using calcium nitrite is a premium of about 50 percent on the base delivered price of the concrete.

Concern for the long-term performance of calcium nitrite, and the focus on bridge deck problems in which epoxy-coated reinforcement was the first form of corrosion protection in the marketplace, have meant that use to date in highway applications has been limited. The major user has been the state of Illinois, which requires calcium nitrite be used in all prestressed, precast box beams that do not utilize a cast-in-place overlay or other protective coating.

Other specialty concretes, such as latex-modified concrete, which is widely used, or polymer-impregnated concrete, which has been extensively evaluated (*200*) as a means of impeding the intrusion of chloride ions in reinforced concrete, do not appear to have been used in prestressed concrete applications.

CONCRETE SEALERS

Sealers, which are applied to the surface of concrete to retard the penetration of chloride ions and moisture, have been available for many years. Engineers familiar with the times when linseed oil was viewed as the panacea for concrete scaling problems have been skeptical about using sealers for corrosion protection. This skepticism has not been diminished by aggressive promotion as a large number of products of widely varying quality contest a limited market. Laboratory studies (*201*), however, have shown that certain specific formulations of a number of chemical materials were effective in reducing water absorption and chloride intrusion on nontrafficked surfaces. Good-quality sealers can decrease chloride ingress by 75 to 95 percent compared with uncoated control specimens. It has been reported that these materials can be applied for $0.35 to $0.75 per sq ft ($3.75 to $8.05 m^{-2}), including preparation costs, materials, and labor (*125, 201*).

Notwithstanding the promising performance data and attractive costs, there are a number of caveats that should be attached to the use of concrete sealers:

- Not all sealers are equal. A sealer must be chosen carefully on the basis of performance data and applied in accordance with the manufacturer's instructions.
- Verification of the laboratory performance under field conditions is generally lacking.
- Requirements for concrete quality and cover should not be relaxed.

DUCTS

In bonded post-tensioning systems the primary purpose of the duct is to form the void into which the high-strength bar, strand, or wire can be installed and stressed. Ducts should have the following properties (*176*):

i) imperviousness to the intrusions of paste or mortar during placement of the concrete
ii) sufficient strength to prevent crushing, puncture, or other damage during installation of the duct or placement of the concrete
iii) sufficient abrasion resistance and stiffness to prevent the prestressing steel from cutting or crushing the duct walls during tensioning
iv) chemical stability to avoid destructive reactions with the cement, grout, or prestressing steel
v) ability to transfer bond between the grout and the surrounding concrete

Current practice typically utilizes continuously wound, corrugated ferrous metal ducts to meet these requirements. In some cases the duct is also required to be galvanized to improve its resistance to corrosion. In recent years, as attempts have been made to improve the corrosion protection afforded post-tensioned strand in harsh service environments, coated metal ducts or plastic ducts have been specified. If ducts are themselves noncorrosive and are also impervious to chloride ions and/or water, then this is an effective way of protecting the strand, provided that protection can also be provided in the anchorage zones.

Plastic Ducts

Spirally corrugated and rectangular hoop-corrugated plastic ducts have recently been introduced to the marketplace as a means of providing both a more durable duct and improved protection to the prestressing steel.

The major concern in the use of plastic ducts is the ability of the duct to transfer stresses to the concrete. In three recent projects in which the use of plastic duct was permitted, the duct had to withstand a "pull-out" load equal to 40 percent of the ultimate strength of the enclosed tendon. The embedment length of the duct during the test was equal to the development length of the tendon. Failure at either the grout-duct or the duct-concrete interface before attainment of the proof load constituted failure of the test. In two of the three cases, testing of the plastic duct was undertaken; in the third case, the alternative epoxy-coated metal duct was selected.

Both series of tests involved nominal 2 in. (50 mm) diameter, corrugated polyethylene tubing. In one series, high-strength bar was grouted into the duct; in the other, four 0.6 in. (15 mm) diameter strands were used. All test specimens with the bar tendon failed, with the bar pulling out of the grout. The 12 specimens with strand tendons failed at loads ranging from 38 to 51 percent of the catalog ultimate strength of the tendon, with only 2 specimens failing at less than the specified 40 percent.

Spirally corrugated polyethylene duct has been used for the transverse deck prestressing in the Doraville Interchange near Atlanta. The duct had a wall thickness of 48 mils (1.2 mm) and was accepted on the basis of pull-out tests described above. Smooth, rectangular polyethylene ducts were used for the transverse deck prestressing in the Wiscasset-Edgecomb Bridge in Maine. The prestressing was designed as an unbonded system. A draft of the specification being developed under NCHRP Project 20-7 Task 32 requires that internal polyethylene ducts be corrugated and wall thickness be 50 ± 10 mils (1.27 ± 0.25 mm).

Aside from the ability of plastic duct to provide proper bond between the prestressing steel and the concrete, questions have also been raised concerning the chemical stability of certain plastics and the possibility of destructive reactions between the components of the plastic and the strand. Primary concern is concentrated on polyvinyl chloride (PVC) and other plastics containing chloride compounds. The concern is based on the possibility that the plastic may decompose and release chlorides in close proximity to the prestressing steel. If this concern is proved to be unfounded, one advantage of PVC is that waterproof splices can be made with adhesives that produce chemical bond. Polyethylene does not have this property, and splices must be made by heat sealing or with threaded couplers.

Other concerns with the use of plastic duct involve the abrasion resistance and stiffness of the material. The basic reason for using plastic duct instead of metal is to prevent chlorides and moisture from reaching the prestressing steel. However, if the duct is punctured or a hole is abraded during threading or tensioning the prestressing steel, the purpose is defeated. In addition, the question of the steel crushing the duct walls over a period of time, either directly or through bond stresses in the grout, has been raised. Further work is required to examine the performance questions raised above before plastic ducts are widely used in North America for the primary prestressing in a component. They do, however, appear to be much more promising for use with the transverse prestressing system in bridge decks because the tendons are usually straight or only slightly draped. This prestressing steel is closest to the deck surface, and therefore most in need of protection. In addition, the nature of the structural system generally precludes its replacement.

Corrugated polyethylene ducts were first used in Europe in bonded prestressing systems in 1967 and have been used successfully on several bridges in Belgium and Switzerland since that time. Ducts with a diameter of $3^1/_8$ to $3^1/_2$ in. (80 to 90 mm) have been profiled to a minimum radius of curvature of 39 in. (1 m) and coefficients of friction and wobble factors have been established. These factors are comparable with the factors for corrugated metal ducts.

Coated Metal Ducts

Coating the external surface of metal ducts with the same epoxy materials used to coat reinforcing steel has been investigated. Smooth, epoxy-coated metal ducts, with a 1 × 3 in. (25 × 75 mm) rectangular cross section, were used for the transverse post-tensioning in the top 4 in. (100 mm) of the deck of the Duwamish River Bridge on the West Seattle Freeway (*202*). Additional corrosion protection was provided by a 2 in. (50 mm) thick concrete overlay on the deck slab. Before acceptance, the ducts were subjected to a "pull-out" test load equivalent to 40 percent of the ultimate strength of tendon, as described for plastic ducts.

The epoxy coating is not sufficiently flexible to bridge the spiral seam in a corrugated duct when it is flexed, and it would appear that only semi-rigid ducts should be considered for coating.

BAR, STRAND, AND WIRE

In much the same way that a duct can be coated, coatings can also be applied to the prestressing steel. A stable coating isolates the steel from contact with oxygen, moisture, and aggressive ions, thus preventing corrosion. This approach has the advantage that the corrosion protection is provided directly to the component most vulnerable to corrosion rather than indirectly, such as is the case with coating of the duct or sealing of the concrete.

NCHRP Project 12-5, "Protection of Steel in Prestressed Concrete Bridges," identified 10 requirements for an ideal permanent coating for wire and strand (*3*):

1. No adverse effect on the strength or ductility of the steel.
2. Ability to withstand a wire elongation of 0.6 percent without cracking or spalling.
3. High bond strength to the concrete.
4. Ability to transmit shear from the steel to the concrete; also, no creep of a type that would partially relieve stresses in a pretensioned member.
5. Good long-term stability in a cement environment as well as in a cement environment contaminated by chlorides and other aggressive ions.
6. Sufficient wear resistance to withstand normal handling without loss of coating.
7. Sufficient flexibility to permit coated wires to be twisted into strand without spalling or cracking of the coating.
8. Sufficient ease of application so that good coverage and good bond to the steel could be assured.
9. No embrittling action on the steel from hydrogen pick-up during coating application.
10. Reasonable cost.

Although the study did not identify an ideal coating, there have been several developments in recent years, largely because of research to identify suitable coatings for mild steel reinforcement. In particular, fusion-bonded epoxy coatings have seen widespread use, and three metals (zinc, stainless steel, and copper) have received serious consideration.

Metallic Coatings

A metallic coating may be noble or sacrificial. Sacrificial coatings are anodic to the base metal. If a break occurs in the coating a galvanic couple is formed and additional protection is provided. The most commonly used sacrificial coating for steel is zinc. Noble coatings, on the other hand, act as a barrier to the base metal and are selected because of their superior corrosion resistance. However, if the base metal is exposed it becomes the anode, and because the coating provides a large cathodic surface, accelerated corrosion may occur in the base metal at the break in the coating. Noble coatings that have been considered for prestressing strand include stainless steel and copper.

Zinc has been used for coating strand for many years in both North America and Europe, though many of the applications are outside the highway field. It is interesting that galvanized strand was used in the first prestressed bridge in the United States, in Madison County, Tennessee, in 1950 (*1*), though it is unclear why its use did not become widespread. In addition to being sacrificed, it has the advantages that it is not easily damaged in handling and installation and is relatively inexpensive. Hot-dip galvanizing the wires for galvanized strand both coats and stress-relieves the wires. However, the galvanizing process also reduces the ultimate tensile strength and increases the ultimate elongation and long-term relaxation of the resultant strand. Special anchorage devices are also required (*203*). In fresh concrete, zinc reacts with the alkalis in portland cement to release hydrogen gas. Concern has been raised about the danger of hydrogen embrittlement or bond reduction because of the increased porosity at the steel interface. Traces of chromate will passivate the zinc, and coated wires can be dipped in a chromate bath at the galvanizing plant or potassium bichromate can be added to the concrete (*204*).

Galvanized seven-wire strand with ultimate strengths ranging from 130 to 225 ksi (0.9 to 1.5 GPa), based on total cross-sectional area, is currently produced by a number of manufacturers in the United States. This strand is not typically used for prestressing, although 225 ksi (1.5 GPa) strand has been produced for such applications.

Galvanized prestressing strand was used experimentally when a bridge in France was stiffened by additional prestressing in 1976 (*205*). The two galvanized cables, which were in addition to the cables required, were stressed and left unprotected in rigid sheathing. One of the cables was removed in 1979 and found to be in good condition. Losses experienced in the three years were 7.4 percent, compared with a forecast loss of 6 percent.

External galvanized tendons were also used in the Sermenaz Viaduct in France (*206*). This structure is of a rather daring design consisting of prefabricated match-cast segments, which were assembled on site by free cantilever construction without any grout or epoxy resin in the joints.

The wide variety of stainless steels and their corrosion-resistant properties have raised interest in their use either in the solid form or as a coating for steel in concrete. Solid stainless steel reinforcing bars have been manufactured in South Africa and England for many years, but their high cost has tended to restrict their use to special applications, such as the attachment of cladding panels on buildings (*96*). Solid stainless steel prestressing bars were used to strengthen the Boivre Viaduct in France, as described in Chapter Three. In 1984, solid stainless steel reinforcing bars were used on an experimental basis in a structure in Michigan (*207*). Stainless-steel-clad bars have also been evaluated in exposure-plot studies.

Solid stainless steel, stress-relieved strand was used by the U.S. Navy in marine pilings, though the primary reason for the use of stainless steel was its nonmagnetic, rather than its corrosion-resistant, property. The stress-relieved strand had an ultimate tensile strength of 228 ksi (1.57 GPa), an ultimate elongation in 24 in. (0.6 m) of 1.5 to 2.6 percent, and a modulus of elasticity of 24,200 ksi (167 GPa). The cost of the strand was roughly 10 times that of 270 ksi (1.86 GPa) carbon steel strand.

At the present time, no North American manufacturer produces high-strength stainless-clad strand or wire suitable for prestress applications, though product development is being pursued. The technology for producing such strand is known, and at least one manufacturer has produced a small quantity of strand experimentally.

Recent interest has also been expressed in the use of copper-clad reinforcement in concrete. Copper was dismissed in earlier studies because it is strongly cathodic to steel and because of concern for its stability in an alkaline, chloride environment (*3, 208*). The strong cathodic action means that the underlying steel may corrode rapidly at any break in the coating. Furthermore, all other steel in the structure must be copper clad or electrically isolated to prevent galvanic action. Nevertheless, copper-clad bars appear to be performing well in FHWA outdoor exposure tests (*209*), and there is interest in copper-coated prestressing steel. Although prestressing steel is not available, nominal 0.5 in. (12 mm) diameter, copper-clad, seven-wire strand with a minimum tensile strength of 150 ksi (1.03 GPa) is produced for grounding and guy-cable applications.

Before either stainless-steel-clad or copper-clad strand could be used, considerable research would be required to determine the relaxation properties, bond characteristics, fatigue properties, and resistance to stress-corrosion. In addition, questions relating to handling characteristics and costs would need to be answered.

Nonmetallic Coatings

In general, a nonmetallic coating is preferable to a metallic coating because it does not form a galvanic cell with the base metal. Nonmetallic coatings can be conveniently divided into inorganic and organic materials.

The only inorganic material that has been found to be feasible for coating prestressing strand is a neat cement slurry. Following an investigation in which the cement slurry was applied to strand in pretensioned and post-tensioned beams and tested for bond and corrosion performance, the coating was found to be suitable for use on pretensioned strand (*210*). It was too brittle for use on post-tensioned strand and susceptible to cracking. The brittleness could be reduced by addition of a plasticizer, but the material was not as versatile as the coal-tar modified epoxy coating with which it was compared.

Research studies in the early 1970s led to the identification of formulations of epoxy that could be fusion bonded to mild reinforcing steel and were effective in preventing corrosion of the steel (*211*). There was rapid acceptance of the product by the highway industry. The same materials used to coat mild steel reinforcement have been used to coat prestressing anchorage hardware, and this application is gaining in popularity, especially in rehabilitation projects. The protection technique has also been applied to high-strength prestressed bars used in the rehabilitation of concrete and timber (*212*) bridges.

Nonmetallic coatings, particularly epoxies, have been considered for use on prestressing strand for many years (*3*). As with metallic coatings, the principal concerns are with the effect of the coating on the bond and elongation characteristics of the strand. In the research study mentioned previously (*210*), a coal-tar modified epoxy was found to offer good protection against corrosion, but it was suggested that longer transfer lengths may be necessary.

At least one U.S. manufacturer produces an epoxy-coated, seven-wire, low-relaxation, 270 ksi (1.86 GPa) prestressing strand (*213*). Coated wire is also available. The coatings used on mild steel reinforcing bars were not sufficiently ductile for the elongation of the strand or they did not bridge the gap between the outer wires. A more flexible epoxy coating was developed that is applied at a thickness over the crowns of the outside wires of 30 $\pm$ 5 mils (0.76 $\pm$ 0.13 mm). The coating thickness is greater where it fills the spaces between the wires. There is no coating on the center wire or the inside surfaces of the outer wires, but this is not viewed to be a problem provided the end of the strand is properly sealed.

Two variations of the coated strand are marketed: epoxy-coated strand designed for use with end anchors when bond to the concrete is not required and bond-controlled strand in which grit is embedded in the coating. The size and concentration of the grit can be varied to provide very short to very long transfer lengths, but the strand that is normally fabricated is designed to have a bond transfer length approximately the same as that of uncoated strand.

The strand has been extensively tested for corrosion resistance, strength, flexibility, bond, fatigue, and relaxation. It is also claimed to be resistant to damage during handling. The coated strand can be gripped for tensioning and anchoring. Pretensioned bonded strands may require more concrete cover than equivalent uncoated strand because of splitting failures in laboratory bond tests. Other concerns that must be answered are the creep and slippage characteristics of the strand and the ability to provide adequate corrosion protection to the bare ends. The cost of the coated strand is approximately double that of the uncoated strand it replaces.

The product has been tested in precast, prestressed bridge deck panels and a utility pole. Coated strand was used in the stay cables of the Bayview Bridge at Quincy, Illinois (*214*), and has been specified for use in the precast concrete piles and the hollow box beams on a bridge replacement in Baltimore.

Nonmetallic Strand

The concept of using tendons of a noncorroding material is not only attractive from the standpoint of improved durability, but the freedom from cover requirements places fewer restrictions on the designer and encourages techniques such as external prestressing. The idea of using nonmetallic reinforcement, especially glass fibers, for prestressed concrete is not new (*215–217*) and has been investigated in the Soviet Union, Europe, and the United States.

In the 1970s work at the University of Stuttgart in Germany showed that the use of glass-fiber rods for conventional reinforcement was not practical because of their comparatively low modulus of elasticity but that the prospects for use in prestressing were encouraging. A project was initiated in 1978 to develop a process for the industrial-scale manufacture of glass-fiber tendons (*218*). The result of this development was a product known under the trade name Polystal, which consists of unidirectional glass fibers protected by a highly filled polyamide coating. The tendons have a high glass-fiber content of about 80 percent by mass, or 65 percent by volume. The purpose of the resin is to prevent shear between the fibers, to protect them, and to provide dimensional stability to the tendon. It also provides protection against the alkalinity of the concrete. Typical properties of the tendons are given in Table 5. The stress-strain relationship is linear to failure. A disadvantage of the tendons is that the resin around the fibers loses strength above 100°C (212°F) and consequently must be adequately protected in case of fire.

The first application of the glass-fiber tendons occurred in 1980 with the construction of the Lunensche Gasse Bridge in Dusseldorf. The bridge was designed as a footbridge suitable for occasional use by emergency vehicles and was constructed as a research project. One of the major difficulties in the practical implementation of the tendons was to develop suitable anchorages. Various types of anchor were tested in the bridge, including anchorages of cast-in-place polymer concrete.

In 1985, construction began of the two-lane, two-span Ulenbergstrasse Bridge in Dusseldorf. The longitudinal prestressing consisted of 59 Polystal prestressing tendons, each consisting of 19 0.2 in. (5 mm) diameter rods to produce a calculated initial stressing force of 3500 tons (31 MN). After stressing, the cable ducts were injected with a resin mortar grout. The bridge was opened to traffic in July 1986, and when inspected in early 1987 displayed no unusual behavior. The bridge is considered experimental by German authorities, and, as a precaution, nine empty ducts have been installed to allow for additional prestressing, should this be necessary. The bridge will be monitored regularly, with particular attention being given to the performance of the anchorages.

An interesting application that is being investigated is the incorporation of optic fibers in the tendon to monitor the strain in the tendon over time.

Tendons made from high-modulus aramid fibers encased in a thermoplastic sheath have been developed and tested in the United Kingdom (*219*). Although well suited to applications in which high strength and light weight are important, aramid fibers degrade in the presence of ultraviolet light and lose strength in the presence of strongly alkaline material; hence, the requirement for the protective sheath. Typical properties of the tendons are given in Table 5. The stress-strain relationship is also linear to failure. A variety of laboratory test programs have been carried out to determine the properties of the tendons that are relevant for prestressing. These include not only strength testing but work on stress-rupture, creep, stress relaxation, and

TABLE 5

MATERIAL CHARACTERISTICS OF STEEL AND NONMETALLIC TENDONS AND FIBERS

Material	Tensile Strength ksi	Tensile Strength GPa	Elastic Modulus ksi	Elastic Modulus GPa	Ultimate Strain %
High-Strength Steel Strands (ASTM M 203)	>250	>1.72	28,000	193	6
Polystal[a] Glass Fiber Tendons (218)	220	1.52	7,400	51	3.3
Parafil[b] Type G Aramid Fiber Tendons (219)	280	1.93	18,000	126	1.5
Carbon Fibers	400	2.80	58,000	400	0.7

[a]Polystal is a registered trademark of Bayer AG, West Germany

[b]Parafil is a registered trademark of Imperial Chemical Industries, U.K.

thermal effects. Stress-rupture is the loss in strength under the action of long-term loads and is a common phenomenon in organic fibers.

It has been found that the creep and relaxation of the aramid tendons is higher than that of steel but not so high as to preclude their use in prestressing. However, because of the lower modulus, the total losses because of creep, shrinkage, and elastic shortening of the concrete are similar to those in a comparable beam stressed with steel. An anchorage system has been developed, based on a conical steel spike inserted in the center of the fiber core so that the fibers (but not the sheath) are pressed against the surface of a mating conical hole in the steel termination block. Because the sheath will not bond to concrete, the tendons must be assumed to be unbonded for design purposes.

Tendons based on aramid fibers are also being developed by a joint venture of a German and a Dutch company, but in this case the fibers are contained in an epoxy matrix (*220*). Other interesting differences are that the tendons are made in flat strips to improve the transfer of forces at the anchorage and they can be embedded in the concrete for pretensioning. Although numerous potential applications have been suggested, including their use in aggressive environments with only sufficient cover for stress transfer, their use has been limited to laboratory testing.

Carbon fibers, which are widely used in advanced composite materials, are not suitable for use in prestressed concrete because of the combination of a very high elastic modulus with a low strain at failure, as shown in Table 5.

Although it will be some time before nonmetallic tendons can be considered for routine use, their development represents one of the most innovative advances in prestressed concrete. The degree of acceptance will ultimately depend on their performance and price and the ability of designers to take advantage of their different properties. For example, the low modulus and excellent corrosion resistance are very desirable properties when using short tendons for rehabilitation and strengthening.

GROUTS

It is generally accepted that the use of a good-quality grout, properly injected, is one of the best means of protecting post-tensioned steel against corrosion. Laboratory tests to simulate voids in a steel duct have demonstrated that significant corrosion of steel can occur even when the duct is sealed (*3*). Incompletely grouted ducts can also result in cracking or spalling of the concrete as a result of freezing of any water trapped in the duct (*18*). Voids in grout appear to be a widespread and potentially serious problem and have been reported in the United Kingdom (*57*), France (*105*), Czechoslovakia (*221*), Japan (*222*), and Canada (*60*).

Most grouts consist of portland cement and water, often with chemical admixtures. The alkaline portland cement paste affords excellent protection to the steel, and a dense paste is resistant to penetration by aggressive ions.

Field observations of structures in service have confirmed the efficacy of portland cement grouts in protecting prestressing steel against corrosion. Although an investigation carried out in the United Kingdom in 1979 of tendons in 12 bridges built between 1958 and 1977 found voids in most of the ducts examined (*57*), there was usually a film of cement paste over the tendon sufficient to protect the steel against corrosion at the time of the survey. Further evidence of the importance and effectiveness of good-quality grout was demonstrated in the results of a postmortem examination of two post-tensioned highway bridges in Canada (*60*). In one structure, the quality of the grout was very variable. Corrosion of the tendons had taken place where the grout did not fill the duct or was of a very poor quality. By contrast, all the ducts examined in the second bridge were found to be filled with a dense, dark gray grout and the cables were free from corrosion, despite the fact that the bridge, which was 23 years old at the time of demolition necessitated by road widening, had been subject to heavy salt use throughout its service life.

A grout for use in post-tensioned cable ducts requires the following characteristics:

1. Adequate fluidity so that it can be pumped into all the available space in the duct.
2. Little or no segregation.
3. Noncorrosiveness and, preferably, the ability to inhibit corrosion of the steel.
4. Little or no shrinkage in the plastic or hardened state.
5. Thermal compatibility with the concrete component.
6. Long working time followed by rapid strength gain.
7. Ability to transfer stresses between the tendon and the duct (adequate strength).
8. Preferably a low permeability and high resistivity.
9. Tolerance to site conditions, including installation by unskilled labor.

Despite the complexity of the requirements, most specifications contain only a few simple acceptance requirements. Flow, bleeding, and strength (usually at different ages) are the most common requirements with a specified expansion when an expansive agent is required. A limit on water-cement ratio is often stipulated, and some jurisdictions have other requirements. For example, French specifications include a shrinkage requirement and, for special applications, a requirement that the grout remain fluid for six hours. In North America, flow is measured by the U.S. Corps of Engineers Method CRD-C79-58, "Method of Test for Flow of Grout Mixtures (Flow-Cone Method)," whereas in Europe the Marsh flow cone is normally used. Because the dimensions of the cones are not quite the same, flow requirements in North America and European specifications cannot be compared directly.

Grouting Procedures

Grouting is a dirty and unpleasant task, often carried out under difficult conditions and under pressure to complete the job quickly. A survey in the United Kingdom (*223*) showed that grouting specifications were often not followed, the work was poorly organized, and there were no contingency plans when problems occurred. Inspection was uneven, the owners' representatives often relying on the grouting "specialists" who themselves were not always adequately trained and often did not appreciate the importance of fully grouted ducts. Serious deficiencies noted were poorly maintained equipment, inadequate cleaning of equipment (which could result in hardened grout becoming dislodged and causing a blockage), and unreliable metering of water, which resulted in wide variations in the quality of the grout. Although it would appear at first sight that mixing cement, water, and admixtures and injecting the resulting grout into a duct are simple procedures, such is not the case under the realities of most construction sites. Procedures that will result in successful grouting are known and are presented in this section. However, it must be recognized that efforts spent on improving grout composition and developing better test methods are wasted unless the more basic problems of equipment maintenance, operator training, and site organization are first resolved.

A high-speed colloidal mixer is essential for the production of flowable, low-water-cement-ratio grout. Most mixers have two containers; one is for mixing and the other is a storage hopper that feeds the pump inlet. The hopper must be kept at least partially full at all times during the pumping operation to prevent air being drawn into the duct, and preferably should provide continuous agitation of the grout. The pump should be a positive displacement type. It should be able to provide an outlet pressure of at least 150 psi (1.0 MPa), though grouting pressures are usually in the range of 80 to 100 psi (550 to 690 kPa). Under normal conditions the grouting equipment should be capable of continuously grouting the largest tendon in no more than 20 min. It is prudent to have standby water-flushing equipment available in case it is necessary to flush out any partially grouted ducts because of blockage or the breakdown of the grouting equipment. The flushing equipment should have a different power source from the grouting equipment and be capable of developing a pressure of at least 300 psi (2.0 MPa). Despite these precautions, the view has been expressed that in the absence of drain vents from the bottom of the duct through the soffit of the structure, which are rarely used, flushing is unlikely to be successful (*223*).

All ducts should have grout openings at both ends. A grout vent is usually provided at the high points of all draped cables, although work in California has shown that high-point vents are required only on ducts more than 400 ft (122 m) long if rigid conduit is used (*224*). Drain holes are required at low points if the tendon is to be placed, stressed, or grouted when temperatures are below freezing. All the grout openings or vents need to have provisions for sealing against grout leakage.

Historically, flushing with water has been used to clear the duct of foreign materials and to wet the duct and tendon surfaces so that the grout flows more easily. Some jurisdictions—California, for example—require that the water used for flushing ducts contain 0.1 lb per gallon (12 g/L) of quick lime (calcium oxide) or slaked lime (calcium hydroxide). After flushing, the water may be removed by oil-free air or displaced by the grout. This practice has been called into question (*225*) because it is difficult to remove all the water from the duct and between the strands. Furthermore, there is a danger of splitting thin sections if the water hits an obstruction. The latest specifications of the Post-Tensioning Institute for stay cables forbid flushing with water (*226*). If the ducts are not flushed, compressed air should be injected to check for obstructions before grouting.

All grout and vent openings should be open when grouting starts. Grout is injected and allowed to flow from the first vent until any residual flushing water or entrapped air has been removed, at which time the vent should be closed. The remaining vents should be closed in sequence in the same manner. Grout should be pumped through the duct and wasted at the outlet pipe until there are no visible air bubbles or water in the grout and both the quality and the quantity of grout ejected are the same as that injected. All ports should then be closed securely and not opened until the grout has set. Alternatively, standpipes may be attached at the high points of the tendon. A study in the United Kingdom suggested that often insufficient grout is allowed to flow from the vents before each vent is plugged because rarely is there adequate provision for the collection of the grout that is expelled (*223*).

It is important that a one-way flow of grout be maintained throughout the grouting operation. If the grouting pressure exceeds the maximum permitted, any of the vents that have been capped may be used as the injection port. However, if one-way flow cannot be maintained, the grout should be flushed out of the duct immediately with water.

If the structure has features that would affect the grouting operation in a nonroutine way, full-scale trials, preferably on a mock-up section, should be carried out using the actual materials, equipment, and procedures intended for use in the construction.

Control tests are required on site throughout the grouting operation. In addition to checking for flow and bleeding, expansion should be measured when appropriate, and specimens made for the measurement of compressive strength. When the grout contains an expansion agent, it is important that the cubes for strength determination be restrained against expansion. Full and accurate records should be maintained of all the control tests. More complete recommendations for grouts and grouting procedures are contained in a number of references (*223, 225, 227, 228*).

Chemical Admixtures to Cementitious Grouts

A number of admixtures are available to improve the characteristics of grouts consisting of a mixture of portland cement and water. These admixtures can be classified as fluidifiers and water-reducers, thixotropic agents, expansion agents, frost-resistant agents, and corrosion inhibitors.

Fluidifiers and water-reducers improve the ability to pump the grout and encapsulate the prestressing steel. In practice, most grouts include a water-reducing admixture in order to satisfy specification requirements for flow, maximum water-cement ratio, and strength. Restrictions are normally placed on the chloride content of admixtures used in grout. For example, the California specification prohibits the use of admixtures containing more than 0.25 percent chloride ions by mass. One of the most comprehensive sets of limits on anions in grouts is contained in the specifications of the Canadian Standards Association (*229*) and reproduced in Table 6.

It should be noted that fluidifiers and water-reducers do not eliminate bleeding and may increase it.

Thixotropic agents are proprietary formulations that thicken the grout, thereby preventing sedimentation and eliminating or reducing bleeding (*228, 230*). Although bleeding is most serious in vertical tendons, it has been suggested that bleeding will occur within the height of a horizontal duct (*228*).

TABLE 6

MAXIMUM CONCENTRATIONS OF ANIONS IN GROUT

Anion	Maximum Concentration mg/L
Chlorides	300
Fluorides	300
Sulfates	400
Sulfides	400
Nitrates	500

The most common expansive agent is aluminum powder, which reacts with the alkalis in the cement to produce hydrogen gas. The purpose of the expansive agent is to counteract bleeding and ensure that the duct is filled with grout. It also acts to reduce pumping pressures (*225*). There are a number of difficulties associated with the use of aluminum powder. The dosage required is extremely small (typically 0.005 to 0.01 percent by mass of cement, which corresponds to roughly one tablespoonful per bag of cement), so that addition rates are difficult to control on site, especially under windy conditions, unless the aluminum powder is premixed with a portion of the cement. In addition to dosage, the amount of expansion is a function of the particle size and the surface characteristics of the aluminum (*231*). Because the expansion begins as soon as the aluminum powder is added to the grout in the mixer, and most of the expansion takes place in the first half hour, it is important that the grout be injected into the ducts immediately if it is to achieve the intended purpose. Specifications that include a requirement for expansion typically require an unrestrained expansion of 5 to 10 percent (*227*).

Concern has also been expressed that the hydrogen may cause embrittlement of the steel (*59, 232*). However, grouts containing aluminum powder have been used for about 30 years and there are no documented cases of these grouts causing embrittlement. Furthermore, the hydrogen is released in molecular form, whereas embrittlement is caused by atomic hydrogen attacking the grain boundaries of the steel.

Calcium sulfoaluminate has also been investigated as an expansive admixture (*233*). Despite producing higher bond strengths it has not been widely used.

There are a number of commercially available grout admixtures that are various combinations of expansion agents, fluidifiers, water-reducers and thixotropic agents. Several manufacturers also market cementitious grouts specifically formulated for grouting post-tensioning tendons. These proprietary products are advertised as pumpable, non-shrink, non-bleed high-strength materials but have not received widespread use in highway structures.

Air entrainment of grout is generally not required, even in cold climates, because the hardened grout will not become critically saturated in service (unless a serious flaw occurs). However, grouts in post-tensioning ducts can cause cracking of the structural member if the grout freezes at an early age (*234–237*). Dilation of the grout can be prevented by entraining an adequate amount of air in the grout, and this is also beneficial in reducing bleeding. Consequently, air entrainment is recommended for grout placed under conditions in which freezing may occur at early ages. The partial replacement of water by methyl alcohol to depress the freezing point of the grout is not recommended because hydration is retarded and the strength permanently reduced (*238*).

Although no record has been found of the use of corrosion inhibitors, and specifically calcium nitrite, in grout, this would appear to be a good use of the admixture because it places the inhibitor inside the duct, where it can be of the greatest benefit. Because the volume of grout used is relatively small, the impact of the cost of the inhibitor on the total cost of the structure would also be small. Development work would be required to develop an optimum grout formulation, and it is likely that a retarder would be needed to offset the accelerating properties of calcium nitrite.

Mineral Admixtures

Condensed silica fume, which was identified as a material that could be used advantageously in specialty concretes, also imparts superior properties to grout. When used in combination with a high-range water-reducer, silica fume enhances the pumpability and reduces the segregation and bleeding of grout in the fresh state. In the hardened state, strength and resistivity are increased and permeability is substantially reduced.

Laboratory studies in Canada (*239*) and Europe (*240*) have investigated the properties of grouts containing silica fume, but neither study examined the potential for grouting post-tensioned cable ducts. However, the work to date suggests that this would be a promising application to maximize the benefits from the relative expense and limited availability of the product.

A later laboratory study in Canada (*241*), which specifically investigated grouts for post-tensioned highway bridges, identified the most promising grout formulation as a combination of normal portland cement, silica fume, aluminum powder, and a high-range water-reducing admixture. The study also recommended that the dry ingredients be blended and packaged off-site to improve quality control in the field.

Polymer Grouts

Although the possibility of using a polymeric grout to protect prestressing steel has been raised, no record of use in North America has been found. Two examples of polymer grouts being used to inject voids in partially grouted ducts (*152, 154*) were identified. These were discussed in Chapter Three. A description was also located (*221*) of the use of a polyurethane foam grout in a rehabilitated bridge in Czechoslovakia in 1982. The grout was used to protect replacement tendons that were placed in PVC pipe. An inspection two years later at four locations showed the pipe to be filled with the foam and the surface of the wires to be free from corrosion.

Despite the fact that surrounding the prestressing steel by an inert, impervious material may, at first sight, appear attractive, the use of a polymer-based grout would have a number of serious drawbacks. Not only are such materials expensive and intolerant of poor mixing procedures but it would be extremely difficult to abort an unsuccessful grouting operation. Other potential difficulties include excessive exothermic heat, creep, and thermal incompatibility with the concrete. It would therefore appear that work to develop improved grouting materials should be based on an investigation of cementitious products.

CHAPTER FIVE

FUTURE RESEARCH

INTRODUCTION

Many of the future research needs follow logically from the discussion of the disadvantages of existing practices in Chapters Two, Three, and Four. Although much progress has been made in improving the durability of new construction, considerable work is needed to develop techniques for investigating the conditions of prestressed concrete components and methods of repair.

The technical research area "Bridge Protection" of the Strategic Highway Research Program (SHRP) will result in the expenditure of $10 million over a five-year period to address the problems of corrosion in existing concrete bridges (*242*). Although many of the projects in this program can be expected to benefit prestressed concrete bridges, only one project, determining the feasibility of applying cathodic protection, applies specifically to prestressed concrete components. This project has been transferred to an FHWA contract study within the NCP Program D4 (Corrosion Protection). Investigating the condition of prestressing steel was considered to be of major importance in the development of the SHRP research plan but could not be accommodated within the budget. NCHRP Project 10-30, "Nondestructive Methods for Field Inspection of Embedded or Encased High Strength Steel Rods and Cables," will establish the design requirements for a prototype system of nondestructive inspection.

For the purposes of discussion, it is convenient to divide the identification of research needs into those that apply to inspection and assessment, to repair, and to new construction. In identifying these needs, the approach has been to identify topics of high priority rather than create a list of researchable subjects.

INSPECTION AND ASSESSMENT

The scientific principles on which the various methods of detecting deterioration are based are well known, and it is therefore unrealistic to anticipate entirely new methods of investigation (*56*). Furthermore, the simple methods have been thoroughly investigated. Consequently, the majority of future developments will result from applications of penetrating or reflected waveforms or magnetic disturbances. It is reasonable to expect significant improvements in existing equipment as electronic circuitry becomes more advanced. Such improvements could be in the form of enhanced signal processing or in the construction of equipment that is more discriminating, rugged, and portable for field use and is also less expensive.

Although there is a demand on highway agencies for test methods that are rapid (especially when traffic control is required) and that automatically reduce the data to common civil engineering terms, high technology has not been embraced by the highway industry to the same degree that it has by most other industries. The sheer number of bridges and limited budgets discourage the use of investigative techniques that are costly to perform or lengthy. Procedures must also be compatible with the expertise commonly available in highway agencies, or the service must be readily available from specialized contractors at reasonable cost. Specific research needs are listed below:

- The most important requirement is for a nondestructive method of detecting corrosion on prestressing steel. NCHRP Project 10-30, "Nondestructive Methods for Field Inspection of Embedded or Encased High Strength Steel Rods and Cables," addresses this need. The research will investigate, under laboratory conditions, the use of remote transducers to excite embedded or encased steel so that the resulting signals can then be analyzed using ultrasonic or acoustic techniques. However, considerable additional work will be necessary for equipment development, and extensive field validation studies will be required to advance the conceptual prototype unit to the stage at which it is suitable for use in routine field operations.
- A nondestructive method is also required to measure the rate of corrosion of steel in concrete. Such a technique would be applicable to pretensioned components, but it is unlikely that it could be used on post-tensioned strand without opening the duct. Existing half-cell tests measure the presence of corrosion but give no information about the rate of corrosion. Such information is needed to predict the effects of corrosion on a structure and to monitor the effectiveness of repair techniques.
- Other needs that are related to predicting the susceptibility of a component to the corrosion of embedded steel are techniques for measuring the depth of chloride-ion penetration and the permeability of the concrete, its moisture content, and its resistivity.
- Techniques that examine the overall response of a structure to an external stimulus, such as load or vibration, offer promise for assessing the significance of deterioration on a structure. Measurement of the overall response of a structure to known forces enables both capacity-reducing effects, such as loss of prestress, as well as strength-enhancing features, such as composite action or the stiffening effect of parapet walls, to be quantified. Existing methods require further development to produce practical and economical procedures.
- To complement the above item, methods of analysis that can better predict the effect of deterioration on a structure need to be formulated and embodied in appropriate codes.

REPAIR

The major difficulty in conducting repairs is that except for cathodic protection, which may not be appropriate for prestressed concrete members, there is no method of arresting ongoing corrosion. The ideal solution would be a simple and inexpensive material or process that could be applied to existing structures and stop corrosion (by neutralizing the effects of the chloride ions, for example). Decision-making is complicated by the lack of data on the service life of repair techniques and a knowledge of the optimum time to take action. A list of research needs dealing with these issues follows:

- At the present time there is no method that is technically and economically feasible for removing chloride ions from concrete. Pilot studies have shown promise (*243, 244*), and a project to investigate electrochemical removal is included in SHRP. However, this does not specifically include prestressed components, and work would be needed, if the technique proves feasible for reinforced concrete, to determine whether it can be applied safely to prestressed concrete.
- The removal of concrete is one of the major components of cost in making repairs. Techniques are needed for the controlled and rapid removal of concrete without damage to the concrete and reinforcement left in place. A related study is also included in the SHRP, but additional work is needed to deal with the additional complexities of removing concrete from around prestressing steel.
- Data on the service life of repair techniques are lacking, and this prevents comparison of alternative repair and replacement strategies on the basis of cost-benefit analyses.
- A further development of service-life prediction methods is the formulation of an overall decision model that would result in identification of the most appropriate repair strategy for a particular structure and the optimum time for the repairs.
- It is necessary to investigate the application of cathodic protection to prestressed concrete from two aspects. Is it feasible? Is it safe? These questions will be addressed by an FHWA contract research study. The study will also apply cathodic protection to full-size test specimens. However, if the results are positive, a substantial research program will be required to develop methods that are practical, durable, and economical for field use.
- New approaches to arresting corrosion are required in existing structures. Such work requires an examination of the corrosion of steel in concrete at the fundamental level and identification of the approaches that could be taken to arrest the corrosion processes. Although the work would inevitably be a high-risk endeavor, the rewards for success would be substantial.

NEW CONSTRUCTION

The need for research on improving the durability of new construction has a lower priority than for inspection and repair because there are a number of techniques available that provide good durability. The greater challenge is often to ensure that these techniques are well known and are implemented properly in the field. This is not to suggest that no further work is required in this area, because such is clearly not the case. The deterioration of highway structures has been a sobering experience, and continued research, as well as new materials, design procedures, and methods of construction, is required to ensure that the infrastructure crisis of the 1970s and 1980s is not repeated. It should be noted that the scope of the Strategic Highway Research Program is limited to existing structures. NCHRP Project 4-15, "Corrosion Protection of Prestressing Systems in Concrete," includes the investigation of corrosion-protection methods in new construction. Among the subjects requiring further study are:

- Improved recognition and definition of the service environment of the individual components in a structure. In this way a "zone defense" concept can be developed in which protection is required according to the severity of the exposure conditions. In other words, rather than adopt generalized rules, additional protection is provided only where it is needed, thereby optimizing the cost of protecting a structure against deterioration. Such an approach needs to be incorporated in design codes.
- A study that complements the above item is an investigation of the influence of design practices on rehabilitation. It is envisaged that such a study would include the identification of design details that are most durable and also those that would facilitate rehabilitation, should it be necessary. In order for the product of the investigation to be most useful, typical standard details would need to be prepared.
- Because a number of studies have emphasized the importance of good-quality grouting in protecting post-tensioning steel against corrosion, this would appear to be a fruitful area for research. The research should include an examination of grout materials (including inhibitors), grouting methods, and simple field tests and inspection procedures to ensure that the desired quality of grouting is achieved.
- The introduction of polyethylene ducts is seen as a major improvement in protecting post-tensioning strand against corrosion, and is particularly well suited for transverse ducts, which tend to be straight and nearer to the concrete surface than the primary prestressing steel. However, the ability of plastic ducts to withstand the rigors of construction and their effect on the ultimate capacity of a member need to be investigated.
- With the increasing use of concrete sealers, test procedures are required to detect their presence in the field and to predict their in-place performance.
- Further work is required to investigate the effectiveness and economics of metallic coatings for prestressing steel, including detailed study of the interaction not only with the prestressing steel but with all the metallic components in a typical member, including anchorages, ducts, and mild steel reinforcement.
- Further work is also needed to determine the long-term performance of nonmetallic tendons, particularly with respect to the possibility of slippage at the anchorage and to fatigue and aging effects. It has been suggested that the cost of such tendons will be high (*245*), and their cost-effectiveness needs to be established.

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