Bridge Rehabilitation

Given the country's aging infrastructure, bridge rehabilitation provides an economical and environmentally sound solution to a massive national problem.

FRANK STAHL

Our shrinking resources have begun to drastically affect how we live. For example, recycling has stopped being just a fashionable battle cry of environmentalists. Nowadays, the recycling of common household materials such as glass bottles, metal cans and newspapers is becoming mandatory in many parts of the country. The rehabilitation of our bridges, for whatever reason, is but another aspect of recycling or preservation of existing and scarce infrastructure resources.

In the past, when a bridge was severely damaged or had outlived its functional usefulness, it was abandoned and replaced with a new structure at the same or adjacent location. Today, such action is very rare, notwithstanding such prominent projects in New England as the replacement of the Charter Oak Bridge in Hartford, Connecticut, and the proposed replacement of the Boston Central Artery. Instead, it has now become standard practice to make every effort to rehabilitate rather than replace bridge structures.

Reasons for Rehabilitation

There are many reasons why rehabilitation or replacement of a structure becomes necessary — no structure has an infinite lifespan. Chief among these reasons are structural damage, structural inadequacies and functional inadequacies.

Structural Damages. The most obvious reason for rehabilitation or replacement is damage to a structure caused by the forces of nature or man. For example, the Niagara arch bridge, built in 1898 at the site of an early Roebling suspension bridge, was completely destroyed by ice flow in 1938 and was replaced by the present structure. Probably the most famous bridge failure was the collapse of the Tacoma Narrows Bridge in heavy winds on November 7, 1940. This bridge was completely rebuilt, utilizing only the foundation and portions of the approach viaducts of the original structure. San Francisco's Golden Gate Bridge suffered considerable damage to its stiffening trusses and lateral wind system during a severe storm in December 1951. Subsequently, in addition to the necessary repairs, the bridge was rehabilitated in 1954 by the addition of a lower lateral bracing system.

In addition to windstorms and floods, earthquakes can inflict considerable damage to bridges as was dramatically attested to recently by the damage to the San Francisco Bay Bridge and the Oakland Viaduct during the Loma Prieta earthquake on October 17, 1989. In addition, the 1971 San Fernando, California, earthquake caused serious damage to nearly 70 highway bridges. Seven of these bridges either collapsed or were sufficiently damaged to warrant their replacement. Earthquakes occur more frequently than is generally realized, constituting a serious problem in many parts of this country and throughout the world.

Bridges are often key targets in armed conflicts. Hundreds of bridges were severely damaged or destroyed in Europe during World War II. Many had to be completely replaced, but just as many were built and rehabilitated using existing foundations and substructures, as well as portions of the superstructure. The historic Chain Bridge across the Danube in Budapest and the sixteenth-century Ponte a Santa Trinità in Florence were two such bridges that were rehabilitated. More recently, the unique cable-stiffened San Marcos suspension bridge built across the Lempa River in 1953 and the Cuscatlan Suspension Bridge on the Pan American Highway built in the 1940s were casualties of the fighting in El Salvador.

Not infrequently, the loss of, or damage to, a bridge results from accidents such as a ship or truck colliding with a bridge superstructure, a ship colliding with bridge foundations, or an overheight truck colliding with truss portal bracing.

A more recent phenomenon is the sudden failure of individual bridge components, or even entire bridges, due to a local fracture initiated at a metallurgical defect produced during the fabrication of the detail or in the steel fabrication process itself.

Damages to bridge structures caused by erosion and corrosion are much more widespread, but far more subtle in their effects. Recent notable examples of the effects of longtime deterioration are the collapse of the Schoharie Creek Bridge on the New York Thruway that was caused by scouring erosion and the undermining of a pier foundation, and the collapse of the Mianus River Bridge on the Connecticut Turnpike that was caused by corrosion build-up in the pin-and-hanger assembly. In addition, New York's East River bridges, in particular the Williamsburg Bridge, have suffered extensive corrosion damage that was caused not so much by old age as lack of proper maintenance.

The lack of proper maintenance, or "deferred" maintenance, is a chronic problem that affects both steel and concrete bridges, and the concrete components of steel bridges. The longstanding habit of unrestricted use of de-icing salts and other chemicals is taking a heavy toll on bridges. Bridge deck repairs and replacement have become a routine cause of traffic delays not only in this country but also in Europe.

Structural Inadequacies. Structural inadequacies result from changes in design codes or traffic loads that have occurred since the original design. Many major bridges in this country were constructed in the nineteenth century or early in this century. The Benjamin Franklin, George Washington, Triborough and Golden Gate Bridges were built in the 1920s and 1930s. Hundreds of smaller bridges were built during the Works Progress Administration (WPA) days of the Roosevelt Administration. There was a tremendous increase in automobile and truck traffic shortly after World War II that brought about the creation of a national highway network. This evolution has been accompanied by improved design theories, a better understanding of the strength of materials and the discovery of new materials. Allowable stresses in current codes vary substantially from those used in the original designs of these structures. In addition, entirely new concepts such as fatigue and earthquake parameters must now be taken into account in evaluating structure safety.

Similarly, vehicle loads have increased dramatically since the turn of the century. The only reason that such structures as the Brooklyn Bridge, Williamsburg Bridge and Eads Bridge — all of which were designed prior to the advent of the automobile — can still serve traffic is that they were designed to carry railroad traffic in addition to horse-drawn vehicles.

Functional Inadequacies. Functional inadequacies also result from code or usage changes that have occurred since the structure was originally designed. However, code changes generally result directly from usage changes. Some older bridges were built for a mixture of railroad traffic and horse-drawn carriages. In the early twentieth century, relatively light and slow-moving automobiles started to compete with horse-drawn vehicles. Soon thereafter, the automobile took over the road and after World

War II motorized vehicle dimensions, speed and traffic density began to seriously affect design code requirements. Prior to World War II, a lane width of 9 feet sufficed; a 10-foot lane was considered the norm and there were few other geometric or safety requirements.

Today, a lane width of 12 feet is standard, with 13-foot lanes desired for truck lanes. Functional design requirements include minimum dimensions to side or overhead obstruction; cross slope, superelevation and sight distance specifications; and curb and centermall traffic barrier standards. In determining the design plan for the rehabilitation of a structure, all such requirements must be carefully considered, since the Federal Highway Administration (FHWA) rarely provides funding for structures with substandard features.

Rehabilitation vs. Replacement

Not infrequently, the merits of rehabilitating a structure must be weighed against those of complete replacement. Factors that have to be considered in such an evaluation fall into four basic categories:

- Economic
- Environmental
- Historic
- Political

Economic considerations cover construction costs (including design, construction management and financing costs), traffic maintenance costs during rehabilitation, the cost of temporary repairs to keep a structure serviceable until the replacement structure is completed, and land acquisition and business and private relocation costs. A thorough economic evaluation should be based not on a first-cost basis comparison but on a reasonable life-cycle basis that includes one or more rehabilitation cycles and maintenance costs. In addition, the costs on businesses in the area affected by traffic detouring, the costs of moving businesses and residents from land acquired for new construction as well as the costs of removing tax-producing property should also enter into this equation.

Environmental factors, most of which have a large cost component, affect the quality of life in the area in a way that cannot be completely evaluated in monetary terms. Typically, they include the air-pollution and other effects of backed-up traffic on detour roads, or exposure to lead paint during rehabilitation. For a replacement structure, especially if on a new alignment, the effects of the loss of business establishments, churchs, schools or other public facilities on the remainder of the neighborhood, as well as the removal of homes on nearby schools and churches or on the ethnic composition of the area, must be evaluated somehow.

Older bridge structures must be given some consideration with regard to their historic significance. In some instances, bridges have been made historical landmarks and are, thus, protected by laws that make it difficult or impossible to change features, or replace parts, even if it is necessary for safety's sake. Other structures are eligible for landmark status and, consequently, responsible authorities are reluctant to permit alterations that would endanger this status. A replacement bridge on a new site might impinge on nearby historic structures. Or, the rehabilitation design might not "fit in" with nearby historic structures.

Political considerations include all of the above factors since all public actions fall into the sphere of politics, especially in an election year. No matter how detached and professionally the arguments are presented, occasionally the political process will take over and force decisions to be made not so much based on the facts presented, but on what satisfies a vociferous segment of the population. Engineers are challenged to persuasively recommend the best solution based on engineering, economic and environmental considerations.

Design Approach to Rehabilitation

Bridge rehabilitation is normally preceded by a thorough inspection and structural rating. Detailed instructions and regulations for these operations are now available. However, when the need for engineering inspections was first recognized, engineers had to develop their own program, criteria and test procedures. Probably one of the earliest such efforts was the inspection of the Brooklyn Bridge, performed from 1943 to 1945 by a Board of Consulting Engineers that had assistance from a team of engineers from the New York City Department of Public Works. Their report, "Technical Survey of the Brooklyn Bridge," remains an appropriate model for all engineers engaged in this type of work.[1]

The Golden Gate Bridge was the subject of an in-depth inspection from 1967 to 1968. The work included removing various materials such as cable wires and several suspender ropes for testing. While standard ASTM test methods and material specifications intended for new materials were employed, testing methods and acceptance criteria had to be established for materials that had been in service for more than thirty years. Much path-breaking work was accomplished by the inspecting engineers who worked in collaboration with the steel industry in developing the methods and criteria.

The collapse in 1967 of the Point Pleasant Bridge (Silver Bridge) across the Ohio River in West Virginia with the loss of 46 lives provided the catalyst for a national policy on bridge inspection. As a result, the first national specification, the *Manual for Maintenance Inspection of Bridges*, was issued by American Association of State Highway and Transportation Officials (AASHTO) in 1970.[2] This specification replaced the many different, and mostly nonmandatory, specifications and guidelines for bridge maintenance and inspection in use by various highway authorities.

Code Requirements

The *Manual for Maintenance Inspection of Bridges* is now in its fourth edition and has become a standard reference for the engineering profession. The manual covers two basic topics:

- Inspection
- Capacity rating

Procedures for correcting deficiencies have been specifically excluded from the manual since they must be addressed on an individual basis.

Inspection. Specifications for inspection fall into three basic categories:

- Personnel qualifications
- Frequency of inspection
- Inspection procedures

The manual requires that the individual in charge of inspection operations be a registered professional engineer or have a minimum of ten years' experience in bridge inspection in a responsible capacity and have completed a comprehensive training course based on the United States Department of Transportation's "Bridge Inspector's Training Manual."[3] For a bridge inspection team operating under the general supervision of a professional engineer, the manual requires that the team leader have a minimum of five years of responsible experience and have completed the training course.

Lately, these minimum requirements have been superseded by many agencies that now require that each individual inspection team be headed by a registered professional engineer and that each team have an assistant team leader with a bachelor's degree in engineering or equivalent experience. In addition, the assignment of an independent quality control engineer who is not part of the inspection team is now frequently required. This quality control engineer must be a professional engineer.

The manual specifies that each bridge must be inspected at regular intervals that do not exceed two years. Interim inspections are required for any bridge with known deficiencies or in questionable condition. A stretch-out of inspection schedules, or having initial biennial inspections performed by maintenance personnel rather than an engineering team, are acceptable provisions for new structures.

Inspections should be conducted in a systematic and organized fashion. The manual presents a comprehensive general listing of the items that require inspection from foundations to superstructure and railings. This listing must be modified or supplemented to conform to the actual condition of the structure. The project manager and the chief inspector should visit the site at the beginning of an inspection project to perform a reconnaissance examination that would be used to determine the schedule, type and experience of personnel needed, means and equipment for access and whether any special tools are required. Drawings and past inspection records that are needed to prepare sketches and forms for use by the inspectors should be requested.

Necessary preparations should be completed in the office before the start of field operations. Since inspectors frequently have to work in exposed areas, from scaffolds and in inclement weather, any unnecessary movement to record inspection data will not only delay the project but also may affect the safety of the operation. The use of voice-recording devices is sometimes helpful, especially if the equipment is not hand-held and there are sufficient resources to transcribe the recordings afterwards.

Rating. A check of the load capacity or rating of the structure is normally an integral part of any inspection assignment. This effort requires careful evaluation of many conflicting factors in an effort to extend the useful life of the structure and to safeguard the public. The more questionable the condition and capacity of a bridge, the more detailed an analysis will be required. Not only the physical condition of the bridge as determined by the inspection, but the governing laws and legal requirements of the local jurisdiction, the degree to which bridge load restrictions can be enforced and the interest of the public in obtaining the maximum safe utilization of the facility must be considered.

Ratings must be performed for the "as-built" and "as-is" condition of the structure. The as-built calculation is based on the original design dimensions and member sizes, including any later modifications, but using current design load requirements. For older structures, sufficient field checking must be done to assure that the plans are truly representative of the structure's current status. If no plans are available, sufficient field measurements must be taken to permit an adequate as-built analysis. This as-built design check normally need be performed only for the initial bridge inspection and should be available as a reference for any subsequent inspection and rating.

The design check of the structure in the as-is condition is based on the results of the current field inspection — *i.e.,* considering member sizes reduced by corrosion and wear, members damaged by accidents, and other defects affecting the capacity of the structure unless such defects are scheduled for immediate repair.

Each bridge must normally be rated for two load conditions. The first, or upper, load level determines the absolute maximum permissible safe load level to which the structure may be subjected and is referred to as *operating rating.* The second, or lower, load level determines the permissible load level at which the structure can be safely utilized for an indefinite period of time. This level is called the *inventory rating.* Either the load factor or the working stress method can be employed to determine these ratings.

The manual permits a certain degree of independent judgement in such areas as allowable unit working stresses and assumed loading conditions. The engineer may want to modify the allowable material stresses based on judgements of its quality — as is most commonly done in the case of timber structures — or based on actual tests such as concrete cores. A reduced or increased load impact factor may be assumed based on road alignment, traffic speed and pavement condition. It may also be advisable to use a higher safety factor for a bridge carrying a large volume of traffic as compared to a structure carrying only light traffic.

Reports. The preparation of an all-inclusive report is one of the most important functions of the bridge inspection program, since the usefulness of the information gathered in the field depends on its current and future availability to the bridge operator.

For each structure there should be an inventory that contains complete information on the bridge, including a general description, history, plans, inspection reports, a stress analysis with data on the capacity of the structure, and recommendations for repairs and improvements. Once a basic inventory has been established for a bridge, succeeding inspection reports need only provide updates of the original inventory report to reflect the conditions found during the current inspection, or to record any modifications made to the structure since the last previous inspection. Many agencies have developed their own standard structure inventory and appraisal form that must be filled out by the inspector and made part of the permanent bridge inventory. A sample form is shown in Figure 1.

TP 350d (7/81)

NEW YORK STATE DEPARTMENT OF TRANSPORTATION
BRIDGE INSPECTION AND CONDITION REPORT

SHEET 6 OF 20

REGION N | COUNTY 3 | BRIDGE IDENTIFICATION NUMBER 2240019

FEATURE CARRIED: Bklyn. Br. & City St. FEATURE CROSSED: 1278,City Sts., FDR Dr., & E. River

INSPECTED BY ______ TITLE Team Leader DATE MO. 10 DAY 15 YEAR 88

SPAN NO. (10–12)	DECK ELEMENTS: WEARING SURFACE (19)	MONO DECK SURFACE (20)	CURBS (21)	SIDEWALKS & FACIAS (22)	RAILINGS & PARAPETS (23)	RAILING PAINT (24)	SCUPPERS (25)	GRATINGS (26)	MEDIAN (27)	RECOMMENDATION (28)	SUPERSTRUCTURE: DECK STRUCTURAL (29)	PRIMARY MEMBER (30)	SECONDARY MEMBER (31)	PAINT (32)	JOINTS (33)	RECOMMENDATION (34)
041	5	8	5	6	6	6	8	8	5	5	4	4	5	4	2	4
042	5	8	5	6	6	6	8	8	5	5	6	5	8	8	8	5
043	4	8	5	6	5	5	8	8	6	5	8	5	8	8	8	5
044	4	8	5	6	5	5	8	8	6	5	8	5	8	8	8	5
045	4	8	4	6	5	5	3	5	6	5	8	5	8	8	5	5
046	4	8	5	6	5	5	3	5	6	5	8	5	8	8	5	5
047	4	8	4	6	5	5	3	5	6	5	8	5	8	8	5	5
048	4	8	5	6	5	5	8	8	5	5	4	2	5	4	5	3
049	4	8	5	6	5	5	8	8	5	5	8	5	8	8	8	5
050	4	8	5	6	5	5	8	8	5	5	8	5	8	8	8	5

SPAN NO. (10–12)	PIER: BRG. ANCHOR BOLT. PADS (35)	PEDESTALS (36)	TOP OF PIER CAP OR BEAM (37)	STEM SOLID PIER (38)	CAP BEAM (39)	PIER COLUMNS (40)	FOOTINGS (41)	EROSION OR SCOUR (42)	PILES (43)	RECOMMENDATION (44)	UTILITIES: LIGHTING STANDARDS (45)	LIGHTING FIXTURES (46)	SIGN STRUCTURE (47)	UTILITIES (48)	UTILITIES SUPPORT (49)	RECOMMENDATION (50)	CONTRACTOR CODE (116–117)	REC. CODE (118–119)	TX CD (120)
041	5	5	8	5	3	5	9	8	8	5	5	5	5	5	5	5	99	16	3
042	8	8	8	5	8	8	9	8	8	5	5	5	8	5	5	5	99	16	3
043	8	8	8	5	8	8	9	8	8	5	8	8	8	5	5	5	99	16	3
044	8	8	8	5	8	8	9	8	8	5	8	8	5	5	5	5	99	16	3
045	5	8	8	5	8	8	9	8	8	5	5	5	8	5	5	5	99	16	3
046	5	8	8	5	8	8	9	8	8	5	5	5	8	5	5	5	99	16	3
047	5	5	5	5	8	8	9	8	8	5	6	8	8	5	5	5	99	16	3
048	5	5	5	5	5	8	9	8	8	5	5	5	8	5	5	5	99	16	3
049	8	8	8	5	8	8	9	8	8	5	8	8	8	5	5	5	99	16	3
050	8	8	8	5	8	8	9	8	8	5	8	8	8	5	5	5	99	16	3

REMARKS:

see sheet 17

BD 188 (3/81)

BRIDGE INSPECTION AND CONDITION REPORT

SHEET 17 OF 20

BIN 2240019

FEATURE CARRIED: Bklyn Br. & City St. FEATURE CROSSED: 1278,City Sts.,FDR Dr. & E.Riv.

INSPECTED BY [illegible] TITLE Team Leader DATE 10/15/88

REMARKS:

TP 350, refer to sheet 6 of 20

041 - [30] Same conditions as 039, 040 - [30].

041 - [32] Same condition as 039, 040 - [32].

041 - [33] Heavily corroded and cracked support angles of the roadway joint. West bound roadway (Flag No. 89-150)

043 to 045 - [19] Roadway wearing surface has poor rideability and extensive map cracking.

045 - [21] North outer curb is spalled and cracked, reinforcing steel is exposed.

045 - [25] Scupper at Brooklyn bound outer curb is clogged with dirt and debris, no longer operational.

046 to 050 - [19] Roadway wearing surface has poor rideability and has extensive map cracking.

046 - [25] Scupper at Brooklyn bound inner curb is clogged with dirt and debris. No longer operational.

047 - [21] North outer curb spalled and cracked, reinforcing steel exposed.

047 - [25] Same as 046 - [25], on Manhattan bound outer curb.

048 - [29] Bridge underdeck has efflorescence, random cracking and wetness throughout. Photo No. 39.

048 - [30] Crack in floorbeam to girder connection (Flag report no. 88-904). Fascia stringer has moderate to heavy corrosion full length. Photo No. 40.

048 - [32] Paint on fascia stringer in poor condition. Paint is also chalky. Moderate corrosion, full length.

FIGURE 1. Sample bridge inventory form.

Strength Assessment

Materials Testing. Determining the actual strength of the materials incorporated in a bridge structure, especially in older structures, is an important part of the inspection and rating process. Normally procedures call for the testing of samples that are removed from convenient places in the structure.

The taking of concrete test cores usually presents no problems except on heavily travelled roadways where interfering with traffic should be avoided. Concrete cores are tested not only for their compressive strength but can also be subjected to a variety of other tests that help to establish the future serviceability of the concrete. These other tests include petrographic and chemical analysis to determine the basic composition of the concrete materials, cement and air content analysis, chloride ion analysis to establish the level of chloride contamination of the concrete (a chloride concentration of 1.3 lbs/yd^3 is normally considered the level above which the corrosion of reinforcing steel becomes irreversible), and freeze-thaw cycle tests (since freeze-thaw deterioration is the most prevalent and most pervasive type of deterioration in the northern area of the country, this test is very important in determining whether to repair or replace a deck slab).

Steel samples are usually taken from low-stress members such as stiffener plates, gusset plates and diaphragms; or, if necessary, they can be obtained from the edge of main carrying members in a section along their length where calculated stresses are lowest. Normally, the size of the removed material is not sufficient to make a test coupon with standard dimensions as used in the mill testing of new material and on which all ASTM and AASHTO requirements are based. The testing laboratory normally has to revert to so-called sub-size specimens that, for some properties, require the application of correction factors for comparison with the standard mill test samples that form the basis for acceptance requirements. Mill test samples are taken from specified locations in a new plate or rolled member; samples from other randomly selected locations can produce strength results as much as ten percent below those obtained from the specified location in the same piece of steel.

Load Testing. The calculated strength of a structure, whether based on specified allowable material strength or on strength established by materials tests, often produces results that require restrictions on the use of the facility. When such restrictions, such as load or speed posting or complete closure, impose unnecessary hardships on the public, actual load testing provides an alternative method to establish a structure's acceptability.

Load tests are most commonly performed on structures where the condition of the deck slab or the supporting superstructure produces calculated stresses that require posting or closure. The test is usually performed by operating a truck of known dimensions and weight over the questionable portion of the structure which is instrumented with strain gages. Stress readings normally fall considerably below calculated values. The lower readings are most likely due to frame action in the superstructure members and/or composite action between the deck slab and superstructure, even where no built-in provisions for composite action exist. The AASHTO code permits acceptance of such tests under certain operating and monitoring conditions.

In extreme cases, the load testing of an entire structure can be performed. This was done twice recently on the Williamsburg Bridge in New York City to verify the deflection of the cables and their interaction with the stiffening trusses. The load test results were then compared with the results of a three-dimensional analysis of the cable and stiffening truss system.

Service Life Assessment

Functional Capacity. The service life of a bridge does not depend solely on its strength. The question whether to rehabilitate or to replace often depends on whether a structure is, or will soon be, functionally obsolete and on whether it is economically and environmentally possible to sufficiently improve its functional capabilities.

Many bridges were built at a time when cars and trucks were smaller and lighter, speeds were considerably slower and traffic density was but a fraction of today's. Ten-foot wide

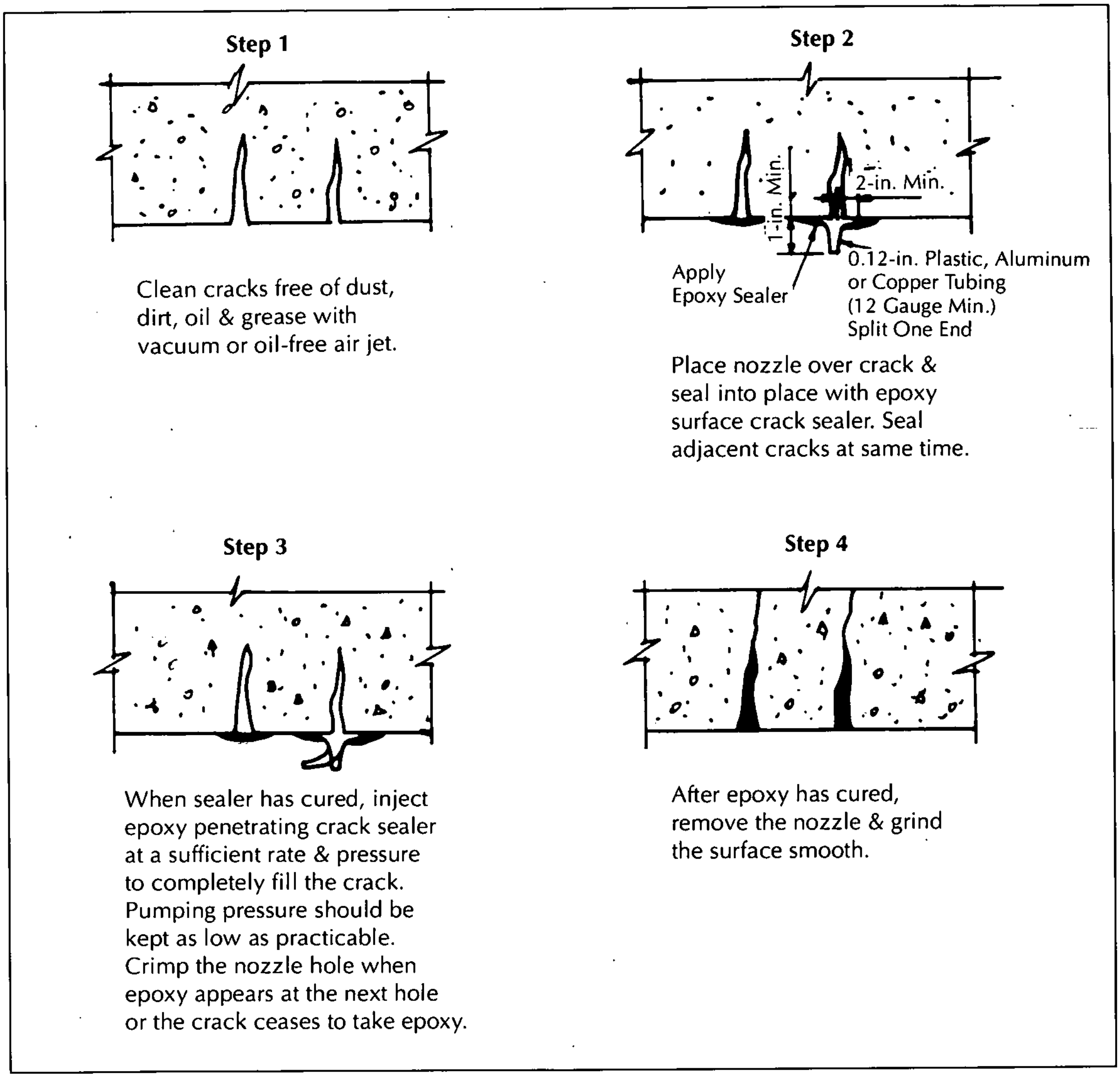

FIGURE 2. Typical crack repair.

lanes are no longer satisfactory. Traffic studies, based on anticipated regional and local changes in population, business and real estate development, are a prerequisite for the proper assessment of a structure's anticipated service life.

Fatigue. In rating a structure, fatigue strength is determined based on fatigue cycles and allowable stress at the time of rating. For a service life assessment, fatigue cycles and allowable stresses have to be projected into the future and the often drastic reduction in fatigue strength, especially for riveted construction, must be taken into account.

A considerable improvement in a structure's fatigue rating can be obtained by replacing rivets with high-strength bolts since a higher fatigue stress range is permitted for mechanically-fastened connections. Replacing rivets with high-strength bolts also provides a convenient method to increase the load capacity of an old structure because of the higher allowable stresses permitted for bolted construction. For structural or economical reasons, however, this solution is not always feasible.

Repair of Structural Systems

Typical Repair Details. The rehabilitation of

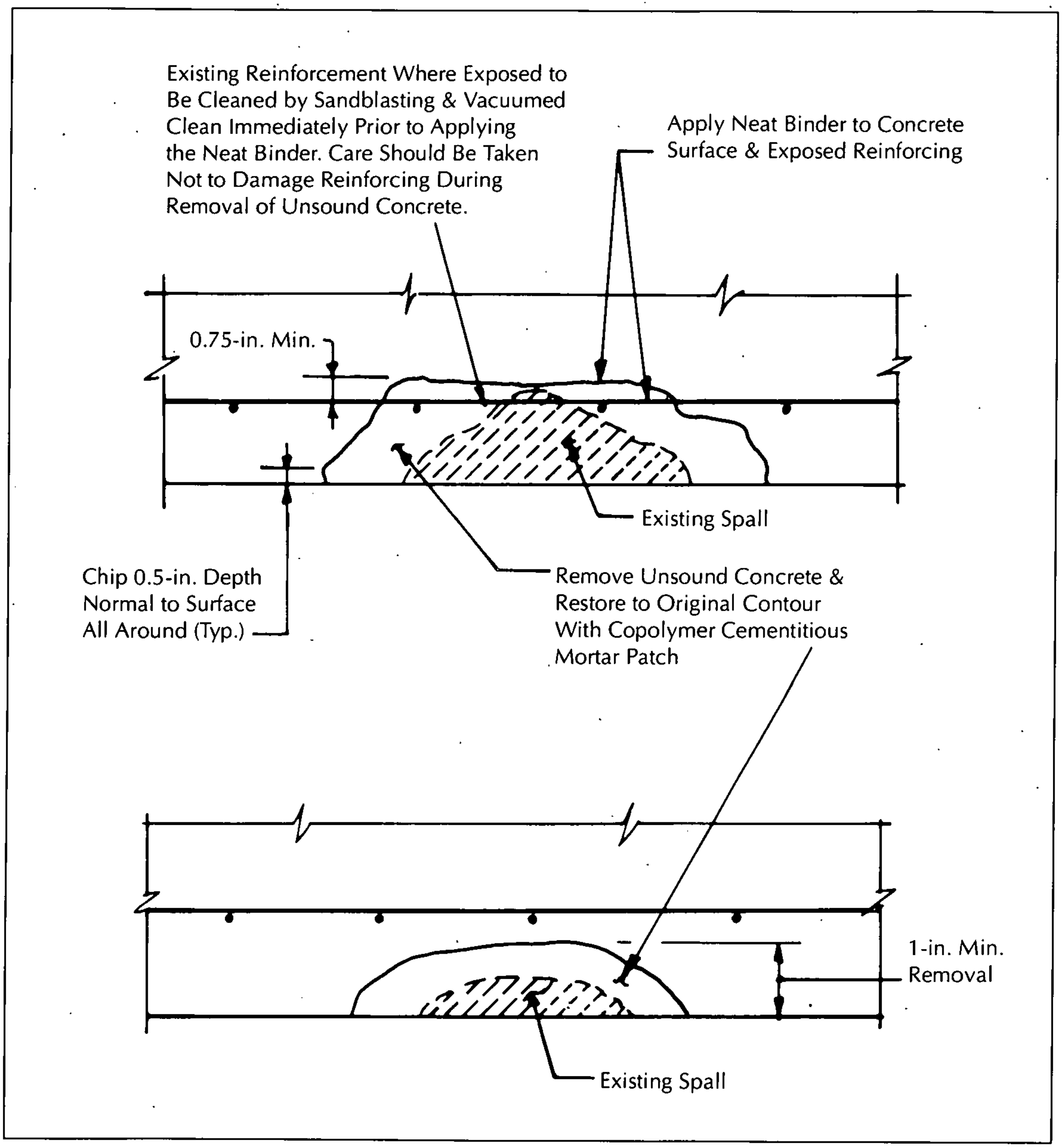

FIGURE 3. Typical spall repair.

structures requires a high degree of engineering expertise and creativeness as well as a thorough knowledge of materials and construction procedures. Design codes and standards, which provide the designer of a new structure with guidelines within relatively narrow limits, must be applied very judiciously in the rehabilitation design. The designer must judge whether material condition, construction detail or actual usage load can justify deviations from specified standards that often can make the difference between rehabilitating or replacing a structural member or the entire structure.

Nevertheless, a certain standardization of procedures covering repair methods and details has occurred over the years, primarily as a result of the extensive bridge rehabilitation programs already carried out by the various state highway departments.

Foundations. Since foundations normally are below ground or water level and out of sight, little thought is often given to their condition. The recent collapse of the Schoharie Creek Bridge, however, has highlighted the consequences of insufficient attention to subsurface structures. In-depth inspection of a bridge should include test pits or inspection by divers to ascertain the condition of the foundation structure. Any deficiency found should be immediately repaired. However, conditions rarely allow for the typification of repair details and the designer must rely heavily on ingenuity to solve the problem.

Substructure. Cracks and spalls are a common occurrence in concrete substructures (abutments, walls and piers). Typical details have been developed by most agencies. Static, narrow cracks are generally repaired by epoxy urethane or conventional grout pressure injection (see Figure 2). If the crack penetrates the full thickness of the structure and the pressure-injected material cannot be contained, epoxy mortar treatment, chemical or portland cement is used. Typical repair details for two conditions of spalls are shown in Figure 3.

Bearing shoes connect the superstructure to the substructure. Corrosion and deterioration of bearings occurs typically under roadway joints, caused by the accumulation of moisture and dirt on abutment and pier seats. The replacement of such deficient bearings requires the careful jacking of the bridge superstructure to avoid local overstressing and the cracking of the roadway deck. Good maintenance, including timely repair of joint seals, can prevent bearing deterioration.

Steel Superstructure. Prior to making decisions on the repair of corroded steel members, representative parts of the structure should be blast-cleaned of all corrosion products, since its appearance usually exaggerates actual damage, particularly to corroded rivet heads.

In addition to rivet heads, corrosion usually affects the edges of plates and rolled members and areas where dirt can accumulate. Feathered or knife-edges should be ground smooth. The acceptable metal loss in thickness or width of a member should be determined by calculations or experience and standardized on repair plans. Losses exceeding these standardized limits should be repaired by such means as adding welded patch plates, or removing and replacing a member locally. Excessively corroded rivets are normally replaced by high-strength bolts.

Major repairs, including rehabilitation requiring a necessary increase in the strength of a member, can be provided by adding coverplates, replacing rivets by high-strength bolts in connections, or completely replacing individual members. Occasionally, prestressing or post-tensioning with cables or high-strength rods has been found feasible. Some typical details are shown in Figures 4 and 5.

Concrete Superstructure. Concrete superstructure members (cast-in-place and precast beams, prestressed beams and boxes) frequently develop cracks; spalls are a less common occurrence. Cracks can be repaired by injection similar to the procedure shown for substructure components in Figure 2. For the repair of spalls in cast-in-place or precast beams, shotcrete is the preferred material (see Figure 6).

Deck Slabs. The deck slab is the bridge member most susceptible to wear, deterioration and corrosive attack. Outward signs of trouble are cracks, spalls and potholes. Determining the reasons for these defects is essential to arriving at a proper decision concerning the rehabilitation or replacement of the deck structure. Other factors entering into this deliberation are the extent of damage, cost of repair, cost of future repairs for any continuing or expected additional problems, and the structure's anticipated service life. Non-destructive testing — such as infrared thermography, ground-penetrating radar, or electric potential measurements (saturated copper-copper sulfate half cell method — see Figure 7) to determine laminations and chloride ion content of the concrete — is often warranted to assist in the decision-making process.

If the decision is made to rehabilitate the existing deck slab, several rehabilitation methods and materials are available, depending on the nature of the problem to be repaired:

1. Epoxy injection of cracks. This method is recommended only if the chloride content of the concrete is at a low level.

2. Removing and replacing concrete in

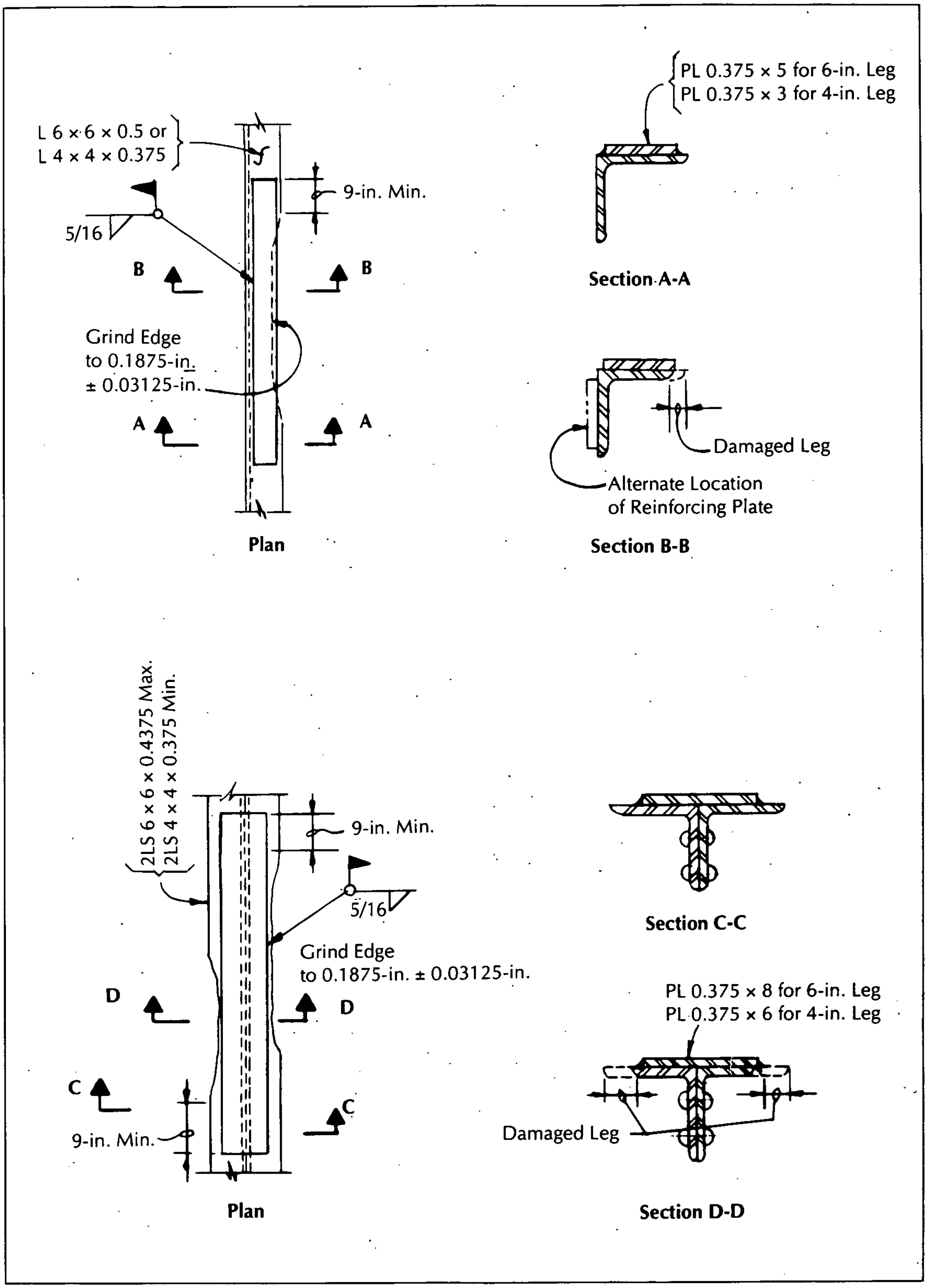

FIGURE 4. Typical structural steel repair — angle repair.

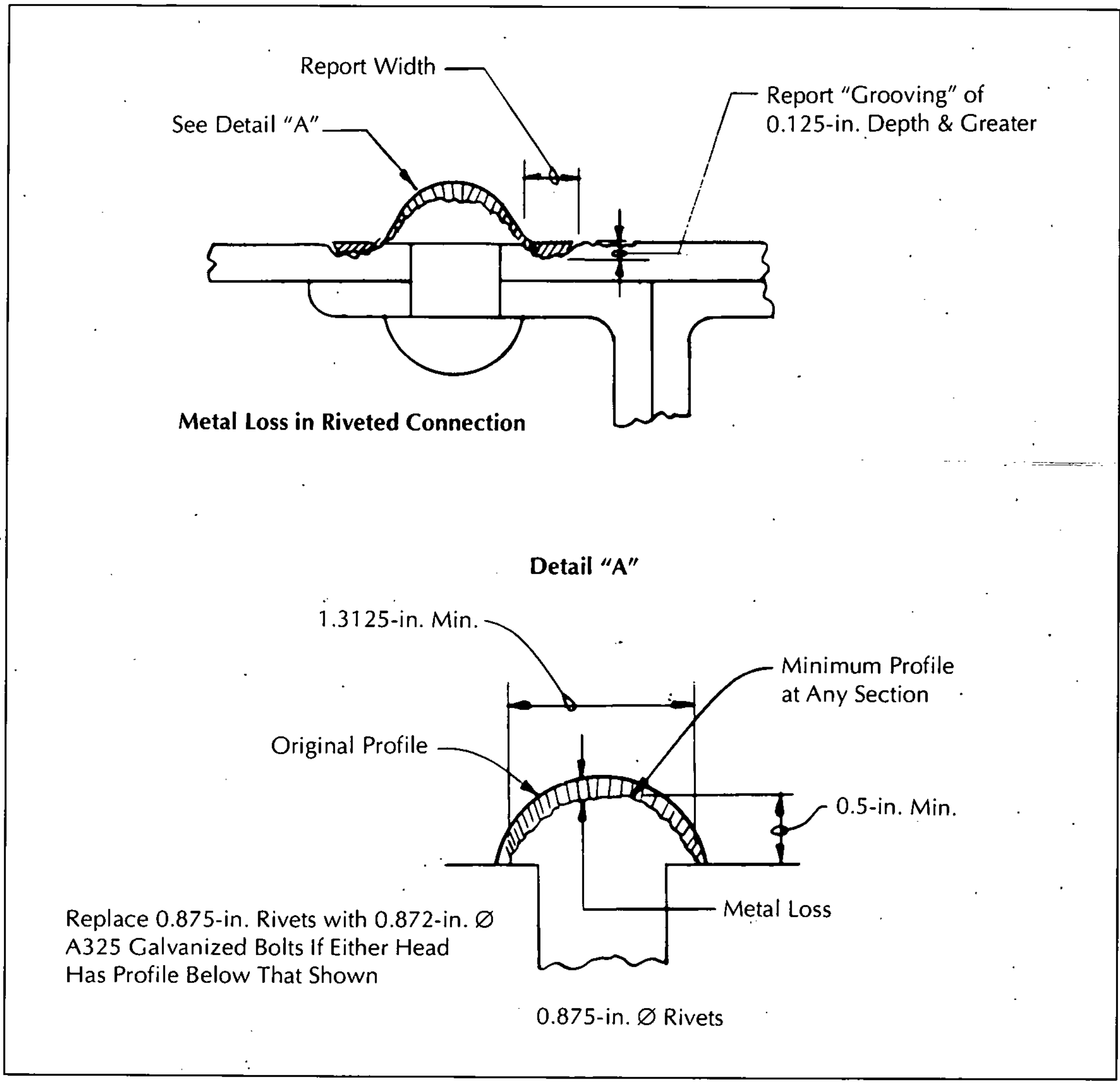

FIGURE 5. Typical structural steel repair — rivet replacement.

spalled areas. This method is recommended only if the spalled areas are limited in number and size, and if the chloride content of the concrete is at a low level. Various materials with different drying times can be used for the patching of spalls depending on the time available for keeping traffic off the repaired area.

3. Removing concrete above the top layer of reinforcing steel (where chloride contamination is generally at its worst) and replacing it with normal concrete, latex-modified concrete (LMC) or silica fume concrete.

4. Scarifying the concrete surface and applying thin overlays (LMC, silica fume or bituminous concrete).

5. Installing a cathodic protection system.

The variety of repair methods and materials is almost limitless and greatly varies from state to state. However, caution must be exercised in selecting and applying the more exotic materials such as LMC or silica fume. These materials require proper mobilization, equipment, trained personnel and supervision that are normally available only on large projects.

Reinforcing steel corrosion as a result of chloride contamination is the most frequent

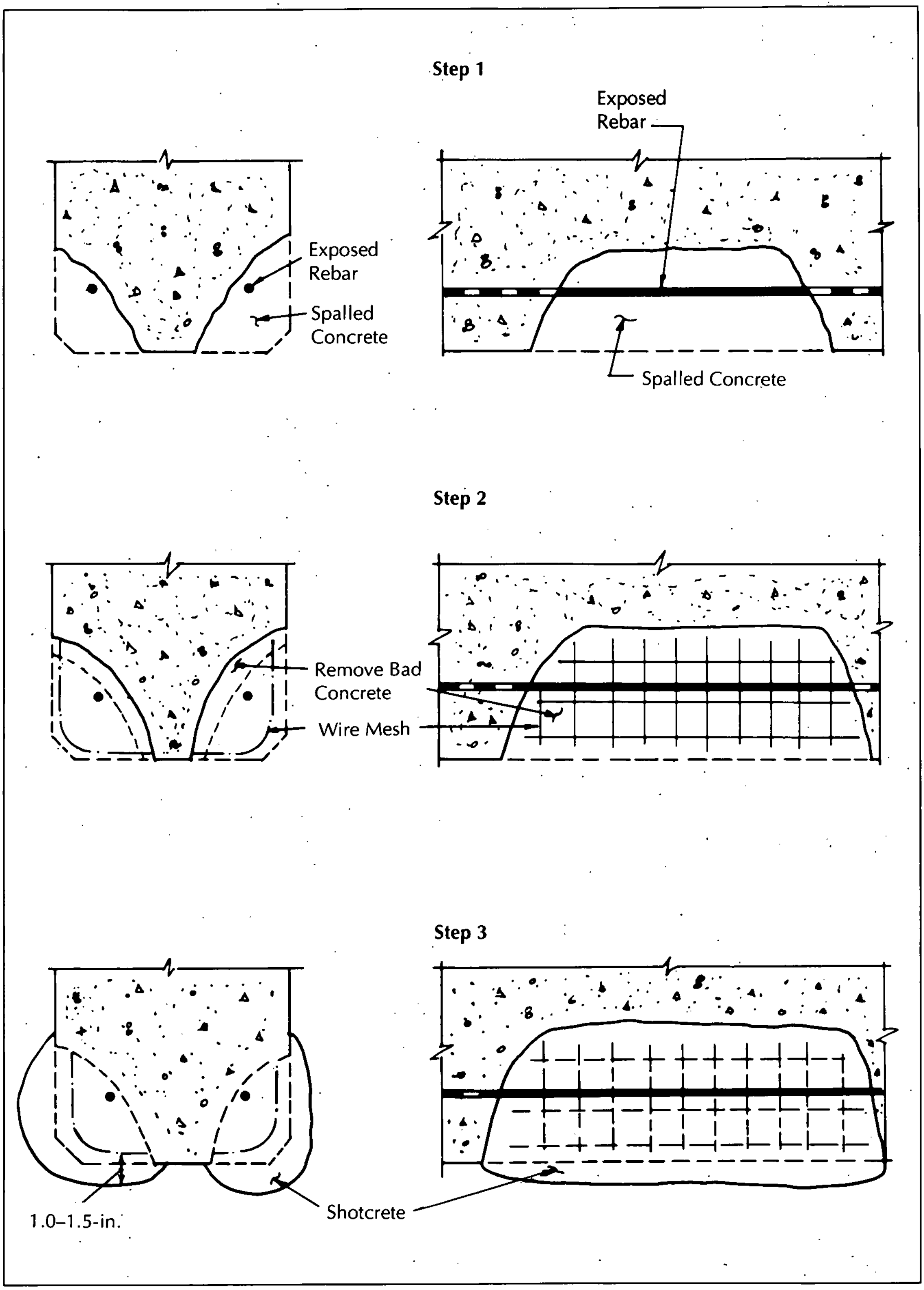

FIGURE 6. Typical shotcrete repair.

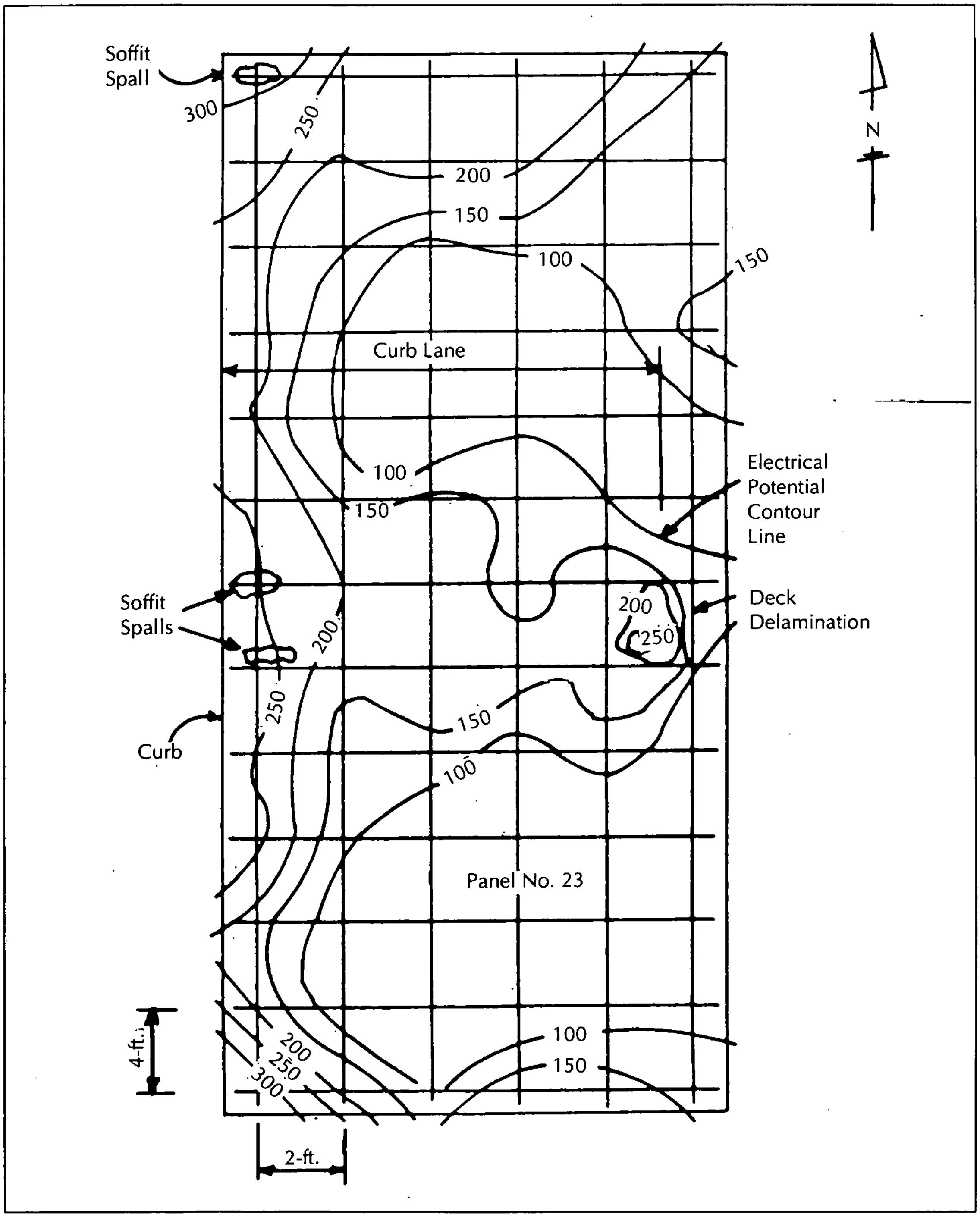

FIGURE 7. Electrical potential measurements.

cause for the deterioration of bridge decks. Cathodic protection is the only known system that will effectively stop this corrosion after it has started. Cathodic protection systems were first installed on bridge decks in California in 1974. Since then, demonstration projects have been installed in many states and in Canada, and are encouraged by the FHWA. Initial cost and necessary continued maintenance are factors that have prevented the widespread use of this method so far.

Suspension System. Suspension bridges make up only a relatively small fraction of bridges in the United States. However, because of their prominence, it is worthwhile to list a few common maintenance and repair problems.

Where problems exist, their solution normally requires engineering imagination and innovative ideas. The main cables are, as a rule, little affected by corrosion. However, suspender ropes are susceptible to the accumulation of moisture and dirt and corrosion at their bottom attachment detail to the suspended structure. The replacement of suspender ropes has become almost a routine operation. Nevertheless, check calculations are necessary to determine whether the suspended floor can be left unsupported during rope replacement or whether installation of temporary suspender ropes is necessary.

Cable band castings are the one location on the cables where routine maintenance and repair will most likely be required during the rehabilitation of a suspension bridge. Unless they are properly caulked around their uphill circumference and along the top joint between casting halves, they provide entry ports for water running down along the cable. Such entry must be avoided at all costs. On the other hand, the bottom joint between casting halves must be kept open and all caulking, if present, should be removed to permit any water that has penetrated into the cable to exit at this point. The bolts that hold both halves of the band together and provide the friction for holding the bands in place and preventing sliding downhill must be checked occasionally and be retightened if necessary. This retightening is now performed with hydraulic jacks.

Seismic Retrofitting. The risk of earthquakes severe enough to affect the safety of bridge structures is not limited to California. The AASHTO seismic risk map identifies large areas in the East and Southeast, in addition to the West Coast and Alaska, where earthquakes of sufficient force to create damage must be expected.

The major damage caused by earthquakes usually results from shifts of the substructure supports or from heavy vibrations of the superstructure, causing the dislocation of bearings and loss of support. The total or near total collapse of bridge superstructures in the San Fernando earthquake as a result of loss of bearing supports started a serious research effort, backed primarily by the California Department of Transportation, to find ways to confine and control damage to predictable levels and locations. That research indicated that restrainers utilizing cables, or rods with springs or neoprene compressive end details, would help to keep structures from vibrating apart or falling off their bearing supports, while permitting sufficient movement for temperature expansion and contraction.

Guidelines for the seismic retrofitting of highway bridges have been published by the FHWA.[4] A number of preventive retrofit projects have been completed with the goal of increasing the resistance of bridges to seismic forces and to minimize the possibility of total collapse. The cost of such retrofit measures is justified by the avoidance of possible loss as a result of an earthquake.

Limitations and Precautions

Maintenance of Traffic Flow. Other factors than design-related ones need to be considered in the planning of bridge repair and rehabilitation. Probably the most important of these factors are the maintenance and protection of traffic. Traffic impacts not only the repair and replacement of roadway slabs (where traffic demands affect construction schedules as well as construction sequences and details), but also substructure repair (which can impact adjacent roadways) and superstructure repair work above the travelled lanes of a roadway spanned by the bridge (where detouring traffic below, or adequate shielding or netting, is required). Occasionally, traffic in the lanes carried on the bridge may have to be interrupted to permit local jacking to replace bearings, temporary disconnection of joints to replace corroded members or replace rivets with high-tensile bolts, or similar operations that could temporarily weaken the portion of structure being repaired.

Systems Compatibility. The compatibility of the repair with the original construction should

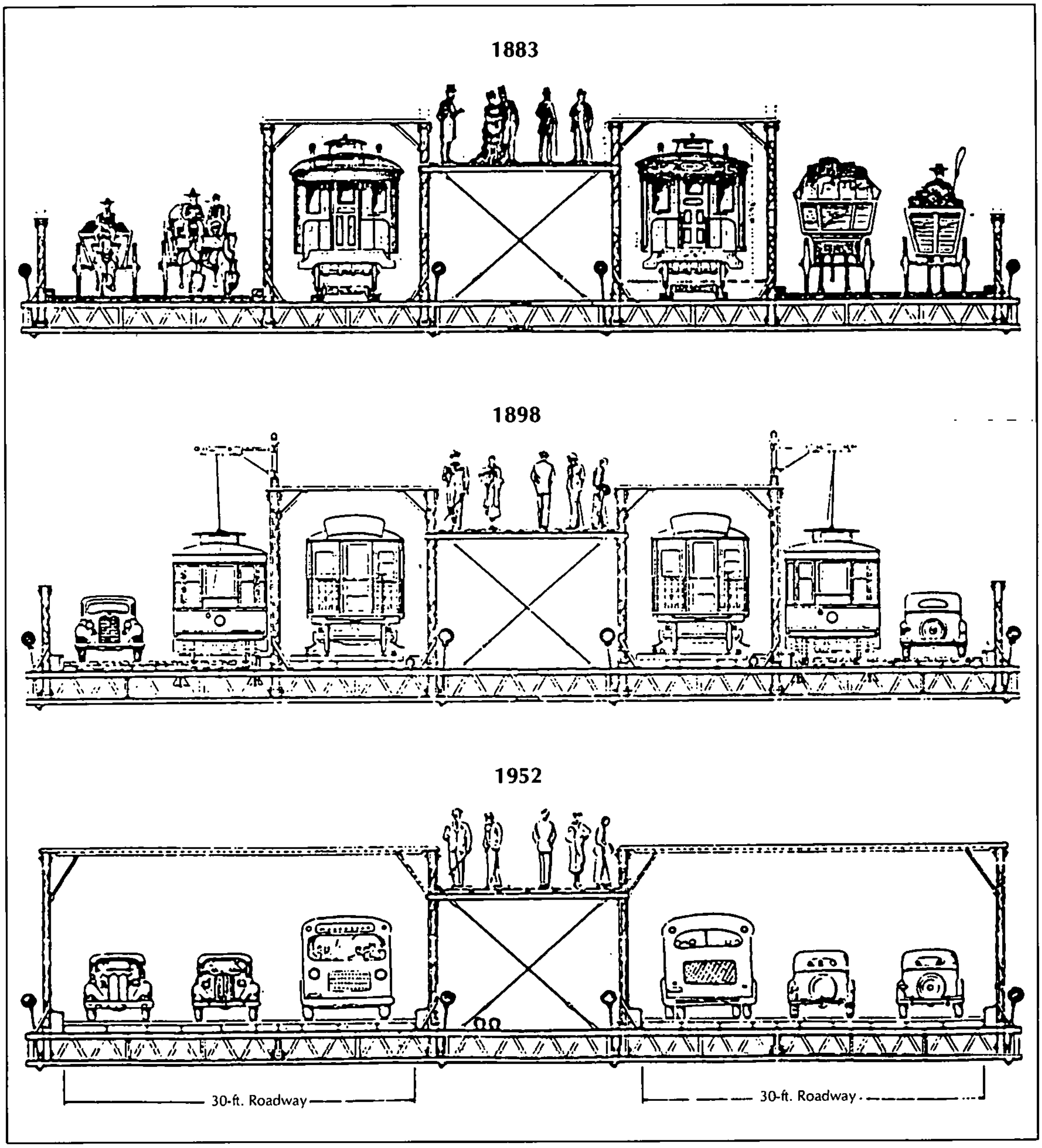

FIGURE 8. Brooklyn Bridge rehabilitation stages.

be considered. For instance, field welding of high-carbon steel found in old structures can cause more problems than it would solve. Weld repair to riveted or bolted construction can create fatigue problems because it changes the relative stiffness of the assembly.

In concrete construction, attention must be given to the joining of new concrete to old concrete. Rather than relying on some instantaneous decision-making by construction personnel at the site, proper details should be included in construction plans and specifications. Some of the overlay materials are not compatible with remnants of previous, removed paving material or with the curing compound applied to fresh concrete. The manufacturer of the overlay material should always be consulted.

Case Studies

Over the last thirty years many bridges have been rehabilitated. In order to gain a better understanding of the finer points of the choice of rehabilitation over replacement, and since the decision-making process is in many ways site-specific, it is best to review some of the significant bridge rehabilitation projects.

Brooklyn Bridge

The Brooklyn Bridge has gone through several phases of rehabilitation to keep up with its age and the changing demands on its services. When the bridge was opened to traffic in 1883, it carried on each half of its cross section a 16-foot, 7-inch wide outer roadway for two horse-drawn vehicles and one railway track for a Pullman railroad car. An elevated pedestrian promenade occupied the space between the two interior cables (see Figure 8). In 1898, the outer roadways were modified to carry one lane of automobile traffic and one trolley car track each. In 1944, the elevated railway was discontinued and their tracks were taken over by the trolley cars. The space formerly used by the trolleys was now available for a second lane of automobiles.

Following the recommendation made in the "Technical Survey" report released in 1945, the intermediate trusses were removed in 1952, the outer trusses were rebuilt and the overhead bracing was extended to the outer truss.[1] These changes resulted in the present two 3-lane, 30-foot wide roadways for automobile traffic (see Figure 8). Trucks are not permitted on the span.

As part of the federally-mandated inspection program, the Brooklyn Bridge was subjected to a complete and detailed inspection in the late 1970s. The extent of deterioration and corrosion found during this inspection resulted in the recommendation for a fifteen-year rehabilitation program (1980-1995) that includes:

- Rehabilitating the cable anchorages, including enlarging the anchorage chambers, realigning the cable splay strands and replacing badly corroded cable wires or strands.
- Replacing suspender ropes and diagonal stay ropes.
- Strengthening the suspended bridge structure, particularly the roadway trusses.
- Rehabilitating the pedestrian promenade.
- Rehabilitating approach ramps.
- Replacing roadway decks on approaches.
- Improving lighting and bridge drainage systems.
- Rebuilding the protection system at the base of the Brooklyn Tower.

At this time, rehabilitating the cable anchorages has been completed and all suspender ropes have been replaced. The replacement of the diagonal stay ropes is in progress. Strengthening the suspended structure and replacing the roadway decks is scheduled for the near future.

George Washington Bridge

The George Washington Bridge was initially conceived as a double-deck structure with the upper deck accommodating seven lanes of vehicular traffic and the lower deck for four tracks of heavy rapid transit trains or additional vehicular lanes (see Figure 9). The lower deck was to be added when the need for such additional capacity arose. Owing both to the economic conditions at the time of bridge construction in the late 1920s and real traffic demands, the upper deck and the approaches were built to accommodate a total of six traffic lanes, three on each outer roadway. The center portion of the roadway deck was left open, to be completed at a later date when traffic volume demanded it. This demand came with the automobile explosion following World War II, and the center portion of the upper deck was completed in 1946 to accommodate a reversible two-lane roadway.

A future expansion of the bridge to serve the ever-increasing metropolitan traffic was recommended in the "Joint Study of Arterial Facilities" performed by the Port Authority of New York and the Triborough Bridge and Tunnel Authority in 1954.[5] This study also included a recommendation for the construction of the Verrazano-Narrows Bridge and the Throgs Neck Bridge. The lower deck was added to the bridge in 1962 and provided two additional three-lane roadways for a total bridge capacity

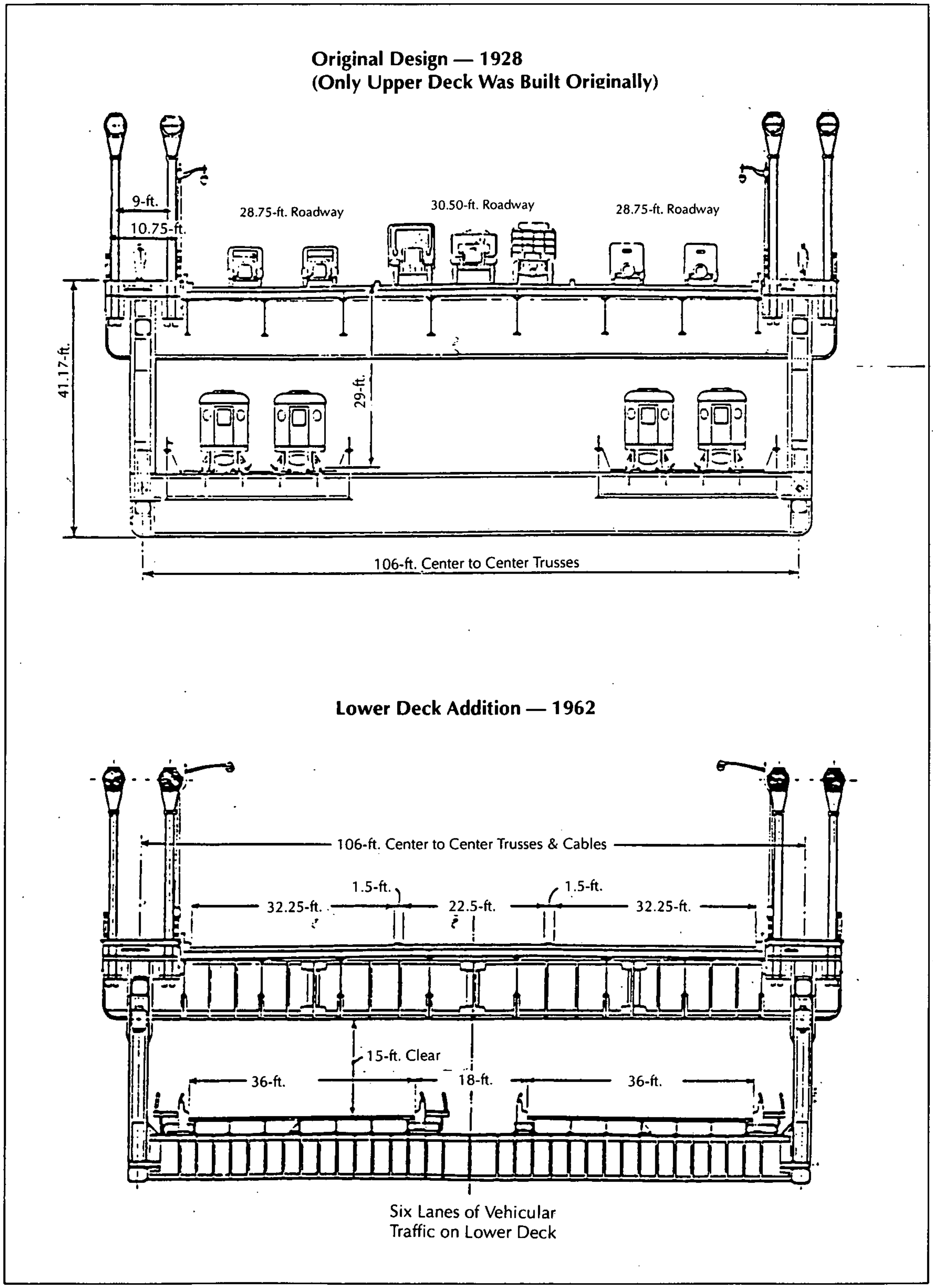

FIGURE 9. George Washington Bridge rehabilitation stages.

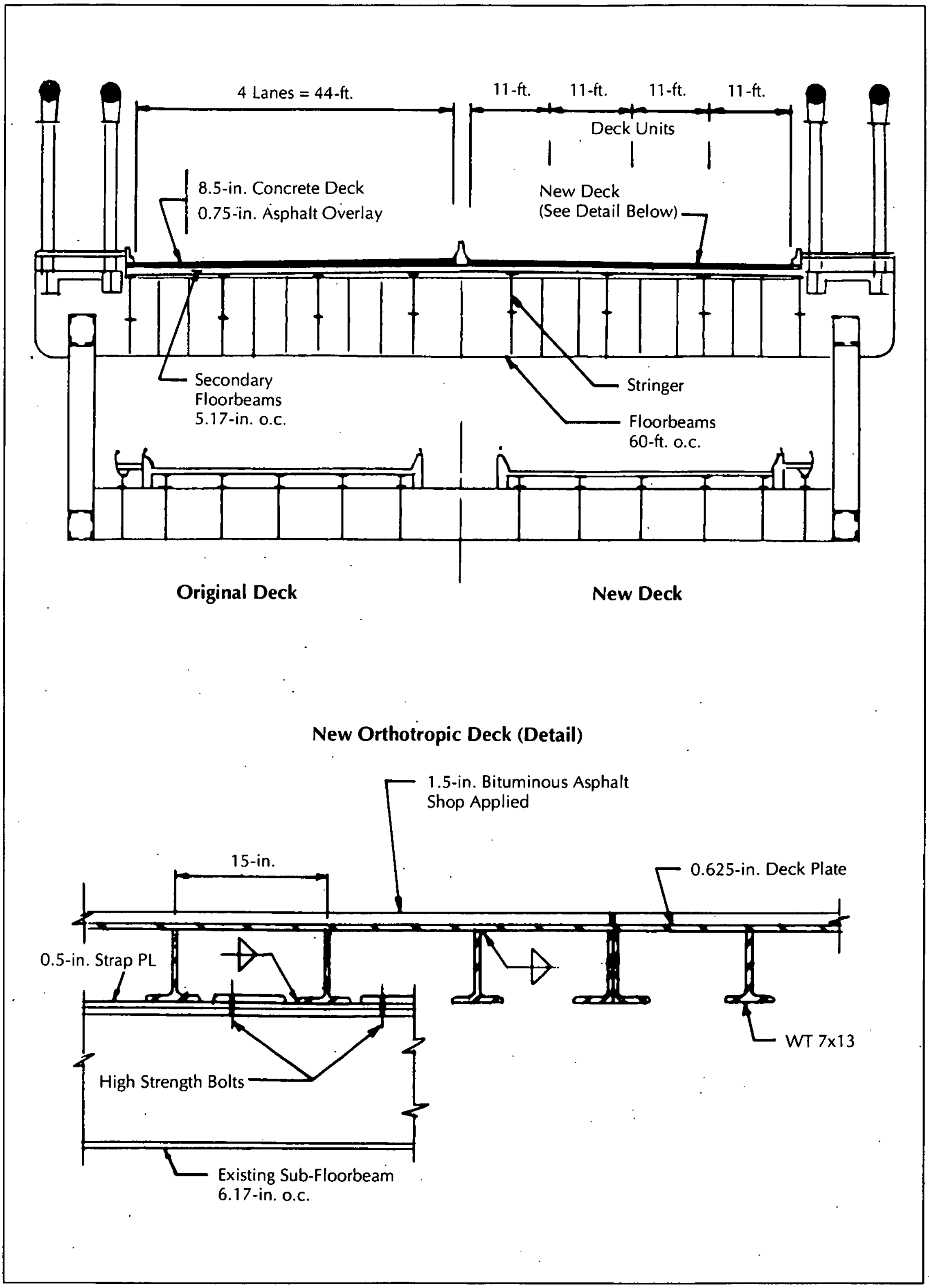

FIGURE 10. The George Washington Bridge orthotropic steel deck.

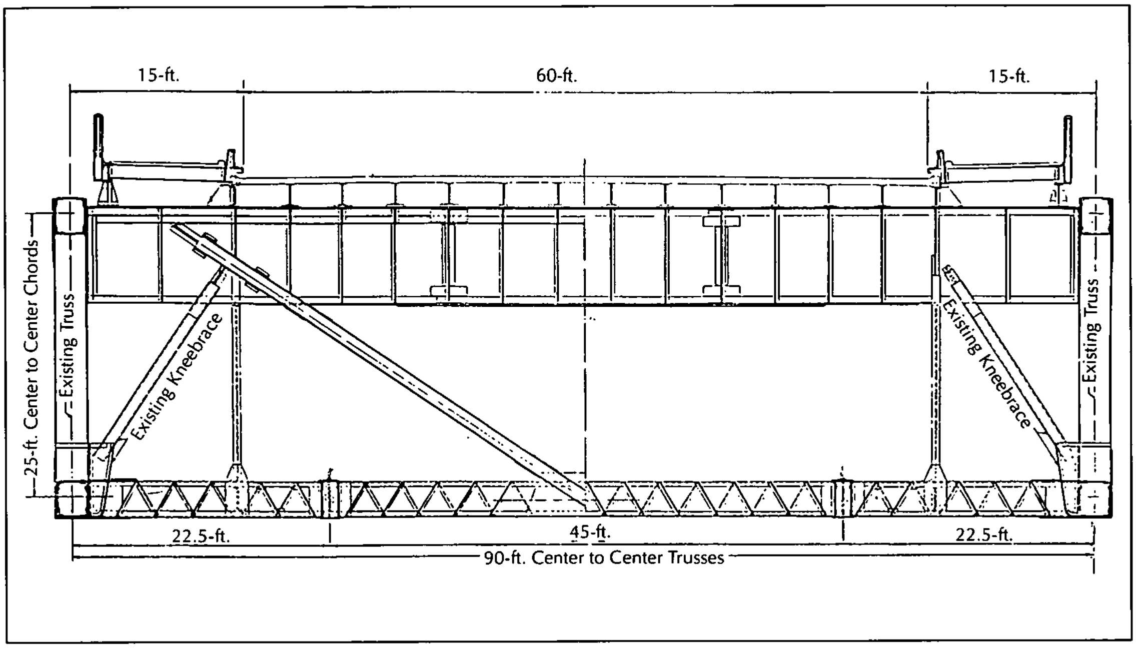

FIGURE 11. The lower lateral system of the Golden Gate Bridge.

of fourteen lanes (see Figure 9). Rehabilitation work was performed on portions of the upper deck roadway concurrently with the lower deck construction. This rehabilitation included work on the roadway joints and fingerdams of the upper deck. In addition, several suspender ropes were removed for testing and replaced by new ropes, and all cable bands were retightened and recaulked.

In the following years, more and more maintenance repairs were required on the upper deck roadway. A study performed by the Port Authority showed that in the ten years between 1961 and 1970, 25 percent of the total deck area had to be patch repaired. As a consequence, it was decided to replace the deck rather than keep on repairing the rapidly deteriorating concrete slab.

The new orthotropic deck that was installed from 1977 to 1978 consisted of a 0.625-inch thick plate of ASTM A588 weathering steel, stiffened by WT7x13 ribs welded to the deck plate and connected with high-strength bolts to the existing subfloorbeams (see Figure 10). The project required replacing each night a 44-foot wide by 60-foot long section of the roadway (or one half-width of the bridge between existing roadway expansion joints). Adjacent sections of the deck were not disturbed during this nightly operation. The new deck modules consisted of four 60-foot long by 11-foot wide panels that were installed separately and then bolted together along their longitudinal edges. The panels were prepaved with an 1.5-inch thick asphaltic concrete pavement and were immediately ready for traffic after installation.

To maintain traffic flow, New Jersey type barriers were installed for the full length of the bridge. During peak hours, all eight lanes were available to carry traffic. During off-peak hours, and at night, all traffic was diverted to one of the two four-lane roadways. The other roadway was prepared for slab removal during off-peak hours; actual deck removal and replacement took place at night. The well-orchestrated use of the various trades permitted maximum utilization of workers and machines in the limited space available. By 6 A.M. the new panel would be in place to permit flow of the daily rush-hour traffic. The entire deck replacement project was completed well within the specified two-year period.

Golden Gate Bridge

The location of the Golden Gate Bridge where the Pacific Ocean enters San Francisco Bay exposes it to fierce winds, salt spray and almost daily fog, all of which have an extremely detrimental effect on the condition of the bridge. With its seismic exposure to the movements of the nearby San Andreas and Hayward faults, and ever increasing traffic demands added to its problems, the bridge has become a textbook example for bridge rehabilitation.

Aerodynamic Improvements. Heavy winds began to barrage the Golden Gate Bridge even during construction. Severe storms during the first decade of its operation culminated in the storm of December 1, 1951, when a strong southwest wind peaked at 69 miles per hour and created double amplitudes of 130 inches at the southeast quarter point and 108 inches at the southwest quarterpoint of the mainspan. During the height of the storm, which lasted in great intensity for more than six hours, the suspended structure oscillated longitudinally through the full length of travel permitted by the expansion provisions at the towers. Considerable damage was caused by this storm; parts of the lateral wind system connections at the towers had to be replaced.

As a result of this storm, the District Board of Directors authorized a Board of Engineers to investigate the feasibility and desirability of adding a lower lateral bracing system to the bridge that would effectively stiffen the original longitudinal trusses at each side of the bridge (see Figure 11). This system, which was installed in 1954, has solved the wind stability problem on the bridge and, for all practical purposes, has reduced the motions to values unobjectional to the traveling public. The bridge experienced a storm in December 1982 that greatly exceeded the storm of 1951 in intensity and length. While the bridge had to be closed to traffic because of the danger to automobile traffic (one light truck was actually blown over on its side), bridge movements were well within acceptable limits and no damage to the bridge was reported.

Concurrently with the installation of the lower lateral bracing system, travelling maintenance platforms were installed in the mainspan and both sidespans that now greatly facilitate bridge inspection and maintenance.

In-depth Inspection. In the early 1960s a comprehensive painting program was initiated that was supposed to provide the entire bridge structure with a paint system that was guaranteed to last at least 20 years between repainting cycles. After extensive tests, a system of inorganic zinc-rich primer with vinyl top coat was selected. However, as soon as painting operations were started, it was recognized that widespread and severe corrosion had taken place in many parts of the structure. Consequently, a complete inspection of all structural components of the suspended structure and approaches was authorized and undertaken between 1967 and 1969. This program included the inspection of the main cables and the removal for testing purposes and replacement of several suspender ropes. As a result of this in-depth inspection it was found necessary to:

- Replace all suspender ropes and their connections to the stiffening trusses;
- Make extensive structural repairs of corroded members in the approach viaduct structures; and,
- Study further the condition of the roadway slab.

Suspender Rope Replacement Program. The testing and inspection of the removed suspender ropes indicated that there was severe corrosion in areas that were inaccessible for maintenance directly above the bearing socket. A progressive general loss of the galvanized protective coating throughout the length of the ropes, both on the exterior and core wires, was noted. More important, the bearing connections to the stiffening trusses were extensively corroded, showing severe loss of metal section in plates and stiffener angles and complete loss of rivet heads in many areas. Due to the configuration of this detail, it was impossible to properly maintain this area or to inspect and discover the serious structural deficiencies while the ropes were in place.

Since the safety of the entire suspended structure was put into question as a result of the discovery of extreme corrosion in the suspender rope connection detail, immediate replacement of the ropes and their attachment details to the stiffening trusses was considered a necessity. Because of the severity of the condition, this replacement program was carried

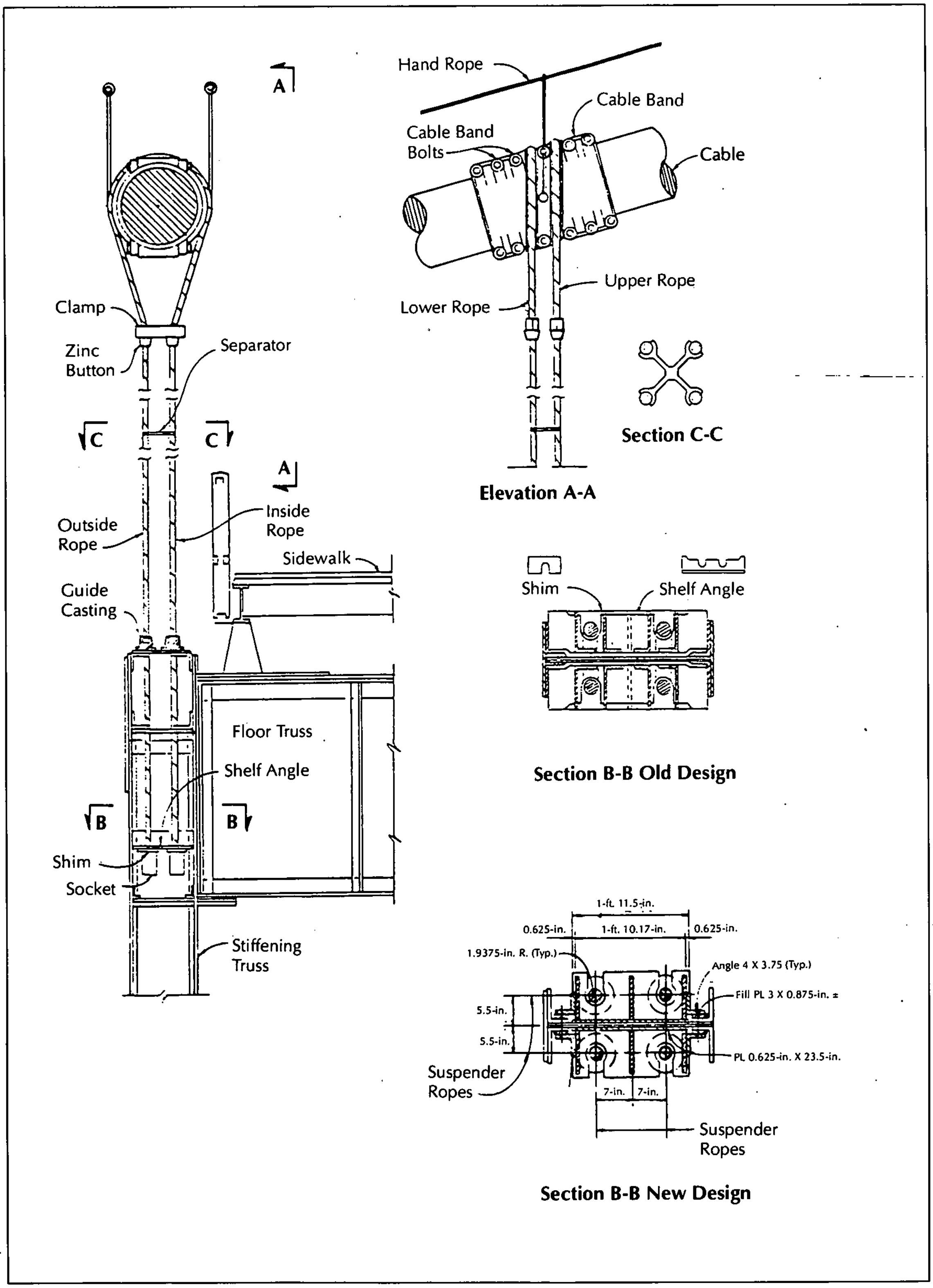

FIGURE 12. The suspender rope assembly for the Golden Gate Bridge.

out in two steps. The first provided for the replacement of every other suspender rope and connection to safeguard the structure; the second stage replaced the remaining ropes. The new bearing connection detail provides ample space for inspection and maintenance (see Figure 12), and the new suspender ropes have been provided with Class C galvanized coating (3 oz/ft^2) on the outer wires and Class B coating (2 oz/ft^2) on the inner wires to guarantee a 50-year life.

The contract documents required that the contractor provide temporary suspenders before removing the existing suspender ropes. Anticipated and permissible jacking forces for every suspender panel point were made available to the contractor as part of the contract documents.

The contractor elected to work with traveling platforms riding on the two main cables from which all temporary ropes and jacking equipment were suspended. This concept proved to be extremely successful, so much so that the entire replacement operation was hardly noticed by the commuting public. The only effect on traffic was that there were three short periods of approximately two hours at midnight when the bridge had to be closed to traffic to permit erecting and removing the traveling platforms.

Approach Repairs. The high level approach roadways are carried by long-span steel trusses and girders, supported primarily by trussed steel columns. Extensive corrosion was found during the in-depth inspection on both approach viaducts on anchor bolts, rivets, structural angles of chord members and diagonals, gusset plates, tie plates and similar members.

Additional anchor bolts were installed to secure the bases of all approach viaduct tower columns. Defective rivets were replaced with galvanized high-tensile bolts and individual corroded steel members are being repaired or replaced as part of a long-range maintenance program by bridge maintenance personnel. To facilitate access for inspection and maintenance operations, an inspection walk was installed for the full length of both approach viaducts in 1978.

Seismic Improvements. When the Golden Gate Bridge was designed, earthquake engineering was in its infancy. The design analysis made at that time assumed earthquake accelerating forces equivalent to five percent of gravity. Because of the exposed location of the bridge, its susceptibility to seismic disturbances has over the years received repeated attention. The most recent study was made based on basic research performed at the Earthquake Engineering Research Center of the University of California at Berkeley and published in their report, "The Effects of Seismic Disturbances on the Golden Gate Bridge."[6] The engineering analysis, utilizing most up-to-date knowledge gained from the 1971 San Fernando earthquake, revealed that the suspension bridge itself was originally designed with a sufficient safety factor to withstand the maximum earthquake forces expected in the Bay area, with no or only very limited and inconsequential local structural damage. However, the approach viaducts were designed to a much lesser standard and required local structural modifications to secure them against collapse during a major earthquake.

Consequently, earthquake restraining features were installed in 1981. Designed in accordance with the requirements of California Department of Transportation standards, these restraining features consisted of three basic types:

- Structural steel members, including rods, ropes and springs to prevent separation of the superstructure from the supporting steel towers (see Figure 13).
- Longitudinal and transverse steel bracing members of the towers themselves.
- Longitudinal and transverse concrete struts to tie together the individual tower column footings.

Adding these members greatly increased the stability of the high approach viaducts and their ability to resist earthquake motions.

Roadway Replacement. As recommended in the report of the 1969 inspection, a detailed investigation of the condition of the roadway slab and its supporting stringers was subsequently carried out over several years and concluded that an extensive rehabilitation with relatively brief life-time benefits or a complete

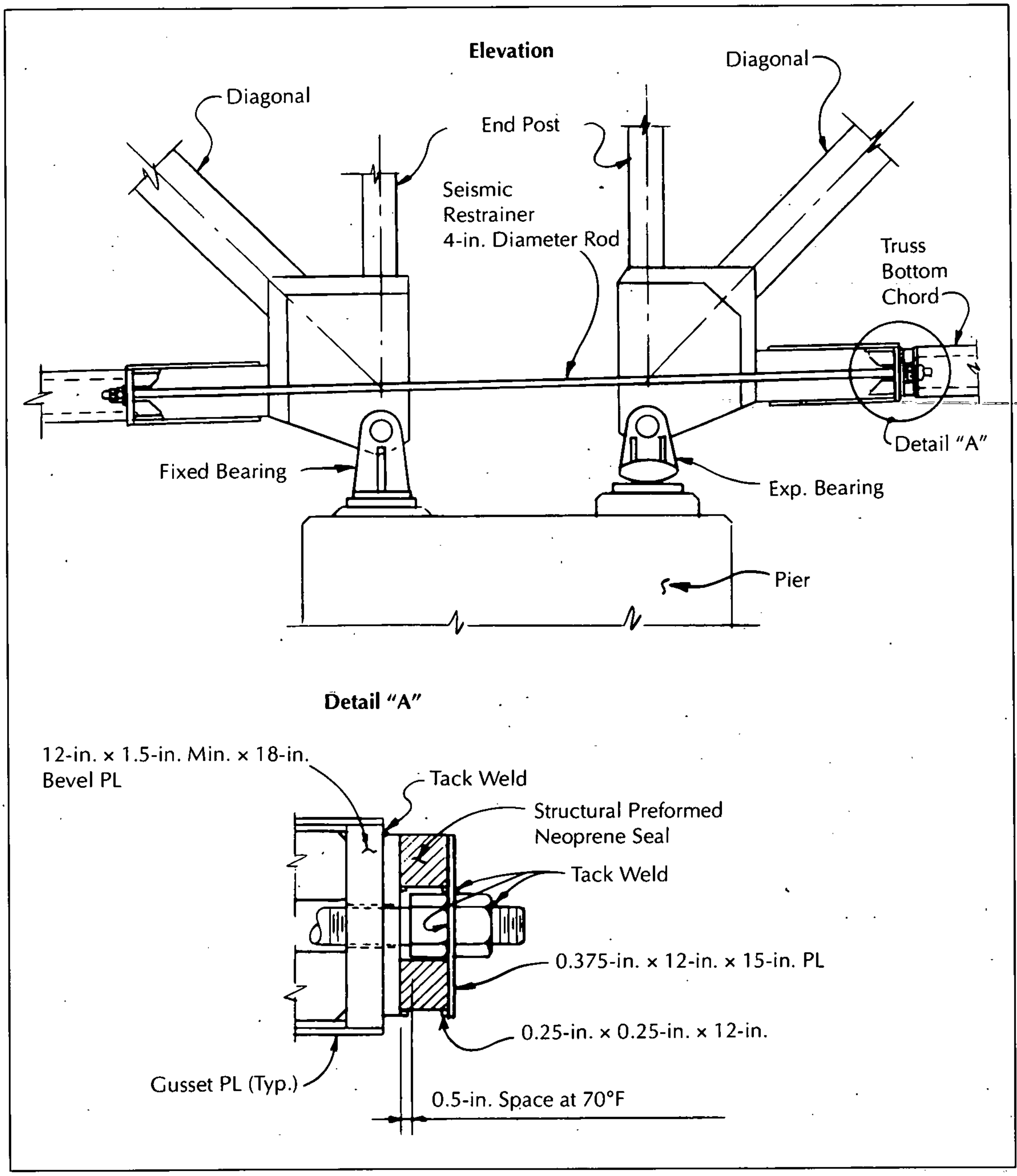

FIGURE 13. Seismic restrainer for the Golden Gate Bridge approach viaducts.

replacement of the roadway slab was necessary. The investigation revealed such defects as:

- A general wearing of the riding surface, in particular along the edges of transverse expansion joints.
- Widespread cracking of the concrete slab, consisting of transverse cracks along reinforcing bar trusses, longitudinal cracks along the distribution steel, random crazed pattern cracking, and shear cracks (on the Marin anchorage deck).
- Scaling and spalling of riding surface due to wearing and internal delaminations.

- Separation of the roadway slab from supporting stringers at expansion joints, probably caused by water leakage.
- Separation of the roadway slab from supporting stringers over intermediate floorbeams.
- Cracking and spalling of concrete haunches, opening up avenues of attack on slab and stringer top flanges.
- Excessive chloride ion contamination of the concrete in the areas of the reinforcing bar mats, in particular the bottom layer, resulting in rusting of reinforcing bars and in concrete spalls at the bottom of slab.

Most surprising and disturbing was the discovery of high chloride ion content in the area of the bottom reinforcement (see Figure 14). Normally, a higher chloride concentration can be expected in the area of the top reinforcing mat resulting from the use of chloride de-icing material. However, such materials are not used in the Golden Gate Bridge area. In this particular case, salt particles are apparently carried by the ever-present fog and deposited on the underside of the slab from where they seem to travel upward into the concrete slab.

An analysis indicated that replacing the deck slab rather than repairing it was by far the more economical solution. Consequently, a complete replacement of the roadway slab of the entire bridge, including approach viaducts, was authorized by the District's Board of Directors.

Since the bridge provides the only direct vehicular route between San Francisco and the counties to the north, deck replacement had to be designed to permit full use of all lanes during the daily rush hour periods, restricting all necessary construction operations to off-peak hours and at night. These traffic restrictions required that the replacement elements be modular, prefabricated, compatible with the existing deck at its interface, and immediately usable upon installation.

The new orthotropic deck (see Figure 15) is composed of a 0.625-inch plate stiffened with 11-inch deep, 0.375-inch thick longitudinal trapezoidal shaped ribs, all of ASTM A709 Grade 36T, Zone 2 impact structural steel. The typical deck module is between 14 feet 3 inches and 16 feet 9 inches in width and 50 feet in length, corresponding to the basic structural module of the bridge. The orthotropic deck is fitted with 0.5- by 12-inch deep subfloor-beams transversely at each end and at the midpoint. This design resulted in a 25-foot span between subfloorbeams. Welded steel plate pedestals of

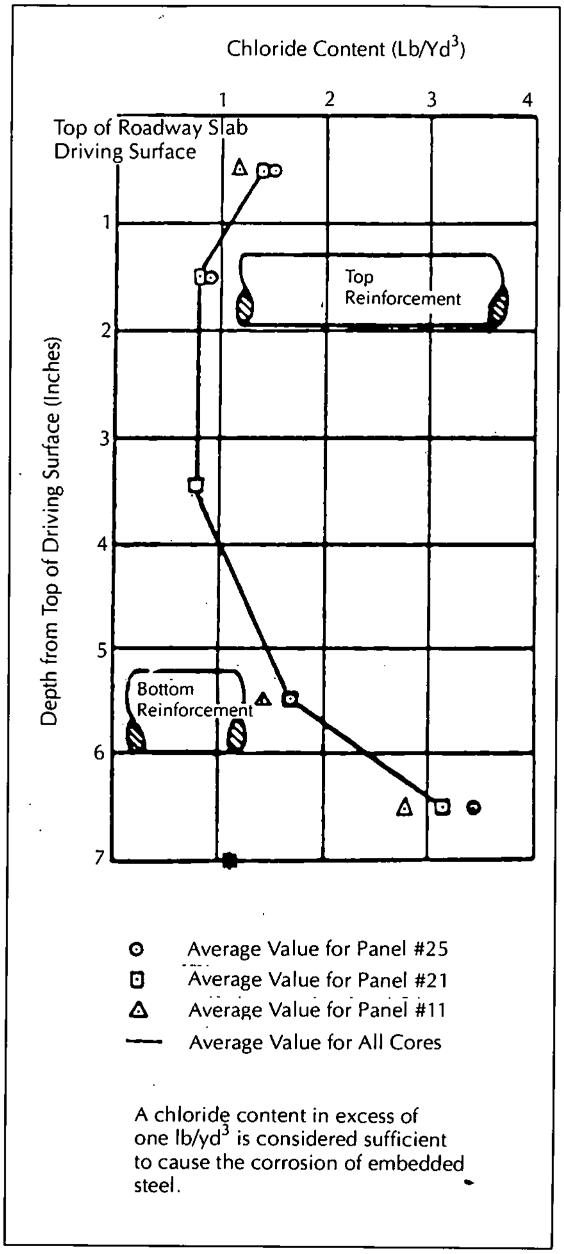

FIGURE 14. Roadway chloride contamination on the Golden Gate Bridge.

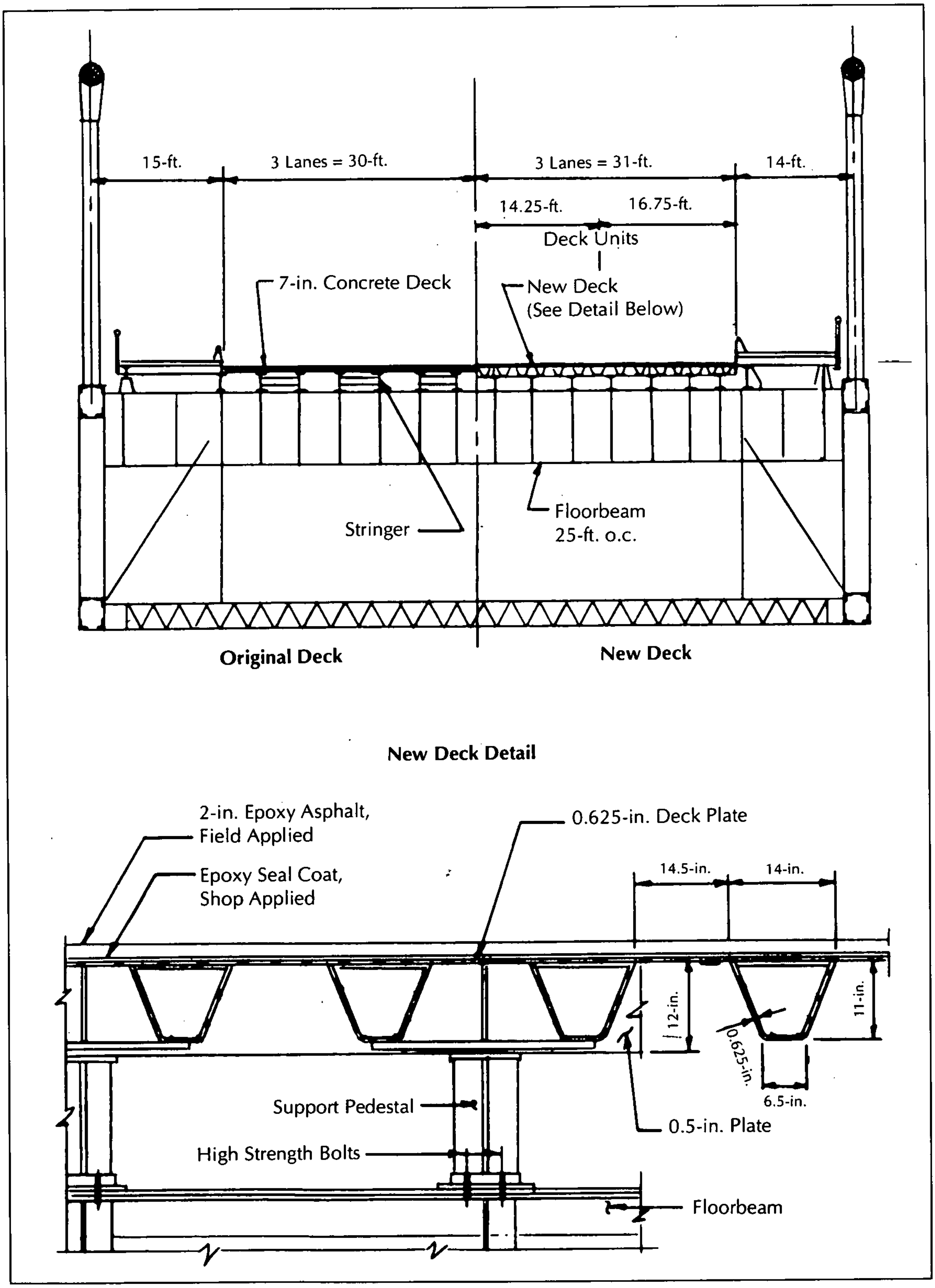

FIGURE 15. The orthotropic steel deck for the Golden Gate Bridge.

ASTM A709 Grade 50T steel connect the deck element to the main floorbeams of the bridge. Twenty-eight million pounds of structural steel were required for the fabrication of the new deck sections and other components of the project that included the suspended spans, and the San Francisco and Marin approach viaducts for a total area of new deck of 567,000 square feet.

Except for bolts in the field splices of transverse subfloorbeams and in the deck-to-support pedestals-to-floorbeam connections, the new deck consists of all-welded construction. The fabrication and installation of the 756 orthotropic deck panels required applying a wide range of welding techniques, from fully automatic full-penetration groove welds (as in the more than twelve miles of weld joining the deck plates in the shop and in the longitudinal splicing of adjoining deck units in the field) to simple manual fillet welds.

Since adequate fabrication facilities were not available in the San Francisco Bay area, the majority of the fabrication was performed in Utah, with other parts coming from Oklahoma and Texas. All fabricated components were shipped to a marshalling yard near the site, where the deck units were preassembled and paved with a temporary riding surface consisting of 0.5-inch crushed stone embedded in epoxy mastic. Field installation of the new deck included removing the existing concrete roadway and its supporting steel stringers; removing, rehabilitating and reinstalling sidewalks; and installing the new orthotropic steel deck. Reinstalling the sidewalks one foot outward from their original position permitted the widening of the roadway from the original width of 60 feet to 62 feet, thus providing 11-foot wide curb lanes for safer operation of trucks and buses.

Two high-speed 25-ton mobile hydraulic cranes were acquired specifically for this project. Traffic constraints required the contractor to perform all preliminary work necessary to remove the old concrete slab and install the new steel decks during the day from a 500 foot long scaffold platform suspended under the deck. The actual deck replacement operation could be performed only at night. Typically, one sidewalk section and two 20-ton deck sections, or a 50-foot long portion of one-half width of the bridge were replaced each night. Traffic interferences was kept to a minimum; the entire bridge was available daily to carry rush-hour traffic. Redecking was completed 401 working days after installing the first deck unit.

The final wearing surface, composed of a 2-inch thick course of epoxy-fortified asphalt concrete that was compatible with the temporary riding surface, was placed after orthotropic steel deck erection had been completed.

Because of its concrete beam and slab construction, the deck over the Marin anchorage was the only area on the bridge that was not replaced with the new orthotropic steel deck. Cracks in the concrete slab were repaired with epoxy injection prior to applying the epoxy-asphalt pavement.

Painting. Painting operations have continued throughout the last several years. In this process, all steelwork is being sandblasted to bare metal before receiving its new paint system of zinc-rich primer paint and vinyl top coat, retaining its landmark color of international orange. Completion of this re-painting process is finally in sight.

Other Deck Replacement Projects

Throgs Neck Bridge Approaches. The Bronx and Queens approach viaducts to the Throgs Neck Bridge in New York City carry two 38-foot wide three-lane roadways separated by a median divider. Rolled beam stringers spanning between relatively shallow floorbeams with long cantilevers support the 7.5-inch thick reinforced concrete deck. The floorbeam cantilever supports the heavily travelled exterior truck lane as well as part of the middle lane. The floorbeams are braced by two deep plate girders with simple spans ranging between 140 and 190 feet.

In the early 1970s, severe cracking and spalling was noted in the exterior lanes, which soon began spreading into the remaining part of the roadway. Inspection and design checks determined that this deterioration was caused by overstress in the concrete deck that resulted from the effects of differential deflections of longitudinal stringers elastically supported on the flexible floorbeam cantilevers. Such effects

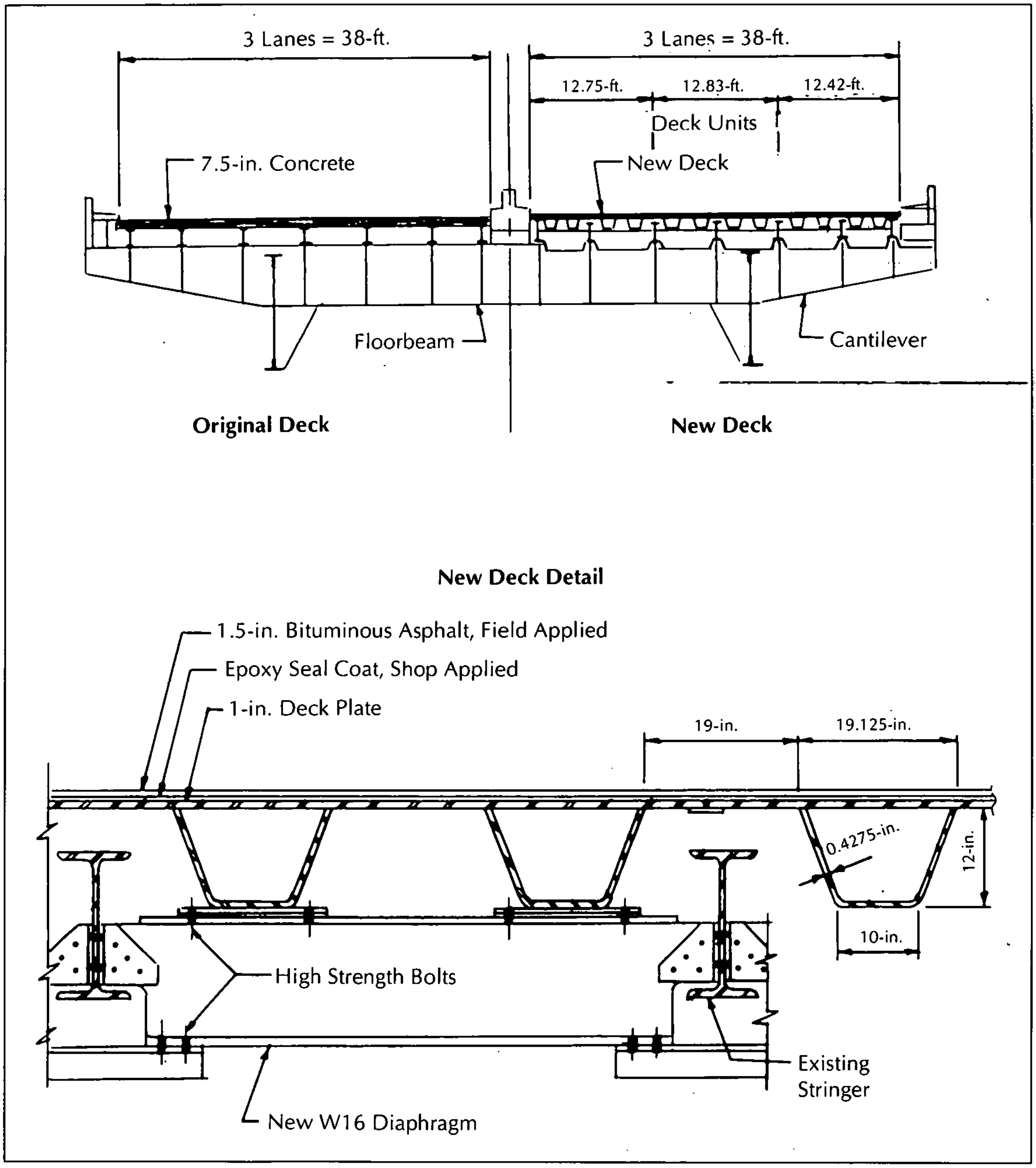

FIGURE 16. The orthotropic steel deck for the Throgs Neck Bridge.

are not recognized in the AASHTO design specifications that base concrete deck design on the assumption that stringers provide rigid support. Inadequate distribution reinforcement and chloride contamination bolstered the decision to replace the concrete decks.

The new orthotropic deck (see Figure 16) consists of a relatively heavy 1-inch thick plate that was required by the shape and spacing of the trapezoidal closed-cell ribs that had to be fitted between the longitudinal stringers. The ribs are 0.44 inch thick and 12 inches deep, and are supported by new W16 diaphragms that transfer the load to the existing stringers. Deck plate and ribs are of ASTM A709, Grade 36T steel with Zone 2 impact requirements. The new orthotropic deck is fully continuous within each span of the approach viaduct; longitudinal and transverse deck plate splices were field-welded and rib splices were bolted. The temporary riding surface consisted of a 0.125-inch thick shop-applied epoxy and grit seal coat that was topped in the field with a 1.5-inch thick bituminous asphalt course as the final surface.

Deck units were up to 52 feet long and between 12 and 13 feet wide. Erection proceeded along one roadway at a time from beginning to end, with traffic diverted during each night to one lane of the other roadway. Sections of existing concrete slab were removed and new deck panels installed for the full width of the roadway each night, with all lanes open in the morning. The total area of replaced deck in both approach viaducts was 492,000 square feet.

Benjamin Franklin Bridge. This suspension bridge across the Delaware River in Philadelphia has a 1,750-foot mainspan and was opened to traffic in 1927. It carries seven lanes of traffic with a daily volume of 100,000 vehicles, and two transit rail tracks. The original 6.5-inch thick reinforced concrete deck slab was progressively deteriorating as a result of heavy traffic and chloride contamination from deicing salts. Corrosion of the supporting steel stringers below the frequent open joints added to the problem. Replacement of the entire 600,000-square-foot roadway area with an orthotropic steel deck was decided in 1982.

An open-rib system was chosen for the new deck (see Figure 17). While somewhat heavier than a closed-rib system, it offered the advantages of simple splices for the continuous deck system, easier connection details to the existing floorbeam supports, and total accessibility of the deck underside for maintenance. The 0.625-inch thick deck plate is stiffened by specially rolled 12.5-inch deep bulb sections, all of ASTM A36 steel. The deck is directly supported on the existing floorbeams that permitted removing the corroded roadway stringers. The deck units were made fully continuous; field splices of deck plates and ribs were made with high-strength and interference body bolts. The elimination of all deck joints removed the maintenance problems that had added to the demise of the old concrete deck. The continuity in the suspended spans also permits the new deck to act as a fully participating component of the stiffening truss system of the bridge in carrying stresses, increasing the flexural and torsional rigidity of the bridge and improving its aerodynamic characteristics.

The base surfacing course of 1.25-inch thick epoxy asphalt was placed on the deck plate under controlled conditions in the shop. The final 1.25-inch thick bituminous asphalt course was placed in the field after completing the deck installation.

Erection of the new deck units proceeded in four construction phases. During each phase, five lanes were open to traffic during peak commuting hours and four lanes in off-peak hours. Work was performed during daytime hours between fixed traffic barriers.

Bronx-Whitestone Bridge Approach. Deck replacement with orthotropic steel plates is not restricted to suspension bridges and other large area projects. It has been successfully employed on small, simple span bridges where savings in weight were desired and in cases where closure to traffic could be tolerated only for a short period.

On the Bronx-Whitestone Bridge approach in Queens, a three-level structure carries the Whitestone Expressway on- and off-ramps over the Cross Island Parkway. Deterioration of the concrete deck of the uppermost level (on-ramp) became so severe in 1984 that complete deck replacement was considered the only acceptable solution.

Since the only detour available for the heavy truck and automobile traffic using this ramp led through local residential streets, the closure of this ramp bridge for the period normally required to place a new concrete slab was unacceptable. The solution was the construction of a pre-paved welded orthotropic steel deck of a design similar to that prepared at the same time for the Throgs Neck Bridge approaches. Replacement of the deck for this two-lane wide and 70-foot long bridge was performed over a weekend, with traffic diverted from 9 P.M. Friday night to 6 A.M. Monday morning.

Tobin Bridge

The Tobin Bridge in Boston was built with a reinforced concrete deck, although it was orig-

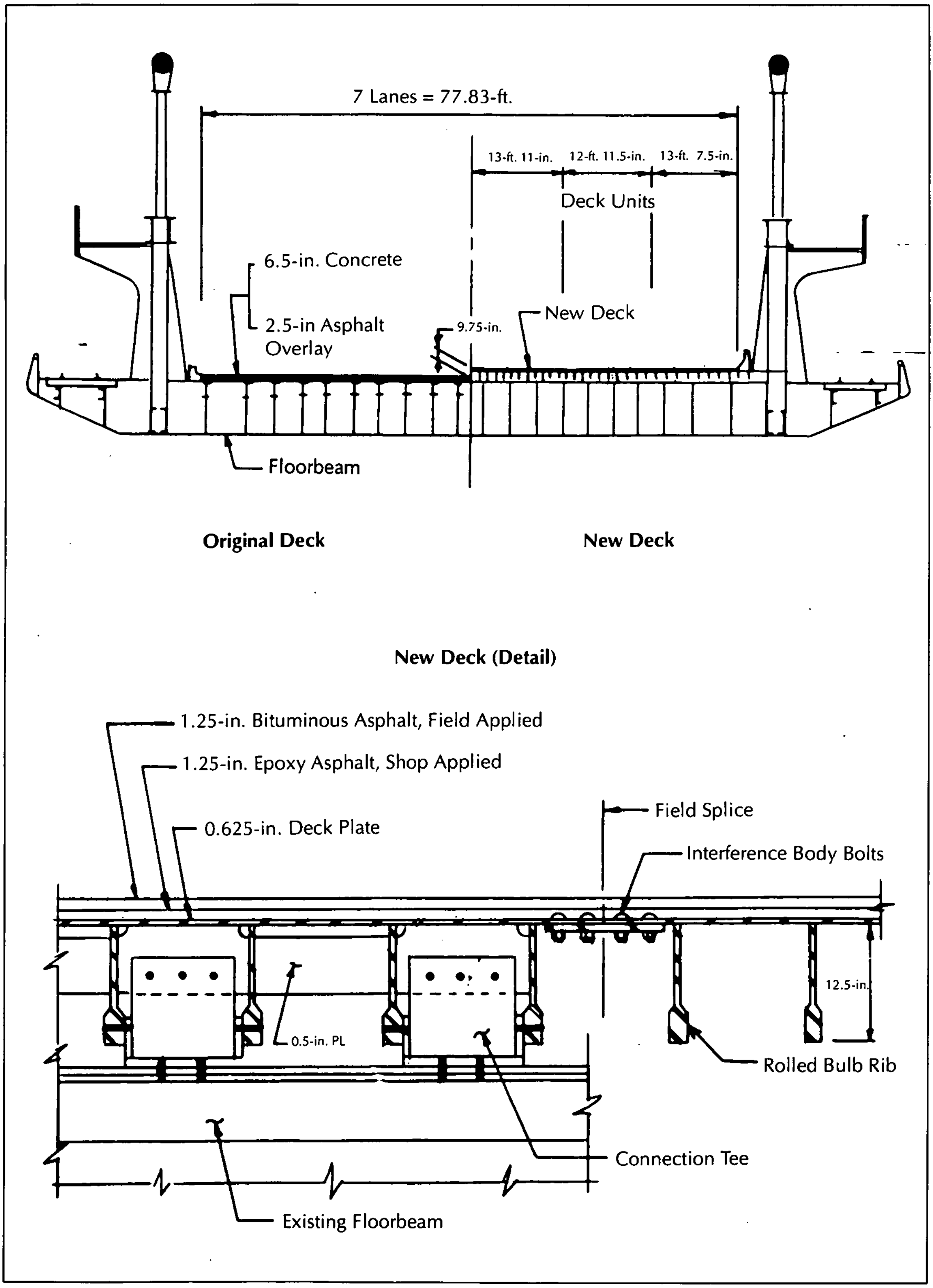

FIGURE 17. The orthotropic steel deck for the Benjamin Franklin Bridge.

inally designed for a steel grid system. Even though it was opened in 1950, by the late 1960s there was so much serious deterioration that the deck had to be replaced with a partially filled steel grid system. The replacement work was installed one lane at a time.

Some ten years after the new deck was installed, it again began to show indications of severe deterioration, both underdeck and on the surface, particularly along the lane lines. A detailed inspection determined that the predominant problem was the lane line deterioration that probably resulted from the difficulty in placing concrete in the narrow spaces of the grid floor adjacent to active traffic. In other areas of the deck, local deterioration most likely resulted from poor consolidation of the concrete and infiltration of surface water. In all cases, the visible damage consisted of break-up and potholing of the asphalt overlay, severe rusting and loss of the metal form pans used in placing the concrete fill, and local break-up of concrete fill. The latter was considered of primary importance because of the danger any metal object or loose concrete falling from the upper deck would pose to traffic on the lower deck.

A large number of rehabilitation and replacement options were investigated that included patch repairs, concrete replacement, grid replacement and even orthotropic plate deck replacement. Each option was reviewed on the basis of initial costs, operational impacts and life-cycle costs.

Since investigations and tests indicated that the steel grid itself and the weld connections to the subfloorbeams were in satisfactory condition, it was decided to remove the overlay pavement and all concrete fill, to make local repairs to the steel grid where necessary, and to place new concrete for the full depth of the grid floor.

This rehabilitation is being carried out in stages. The first stage, a portion of the upper deck on the Boston approach, has been completed. Removable wood forms were employed and silica fume concrete was used to provide impermeability. A 0.375-inch thick epoxy concrete overlay was placed over the concrete-filled grid floor. The second stage, consisting of work on the Boston approach upper deck that incorporates the same rehabilitation method, has yet to be completed.

Royal Gorge Bridge

Considered the highest bridge in the world, the Royal Gorge Bridge spans the gorge of the Arkansas River near Canon City, Colorado, at a height of 1,053 feet above the tracks of the Denver and Rio Grande Railroad that hugs the canyon walls only a few feet above the river. Built in 1929, the bridge has a main suspended span of 880 feet and a total length between abutments of 1,200 feet. Steel framed towers perched at the rim of the canyon support the two parallel wire cables, each about 9 inches in diameter and containing 2,100 No. 9 galvanized steel wires. The unstiffened, timber-decked floor is 18 feet wide. Wire rope wind cables anchored to the canyon wall provide the necessary wind stability.

The main cables were anchored in concrete-filled trenches cut into solid granite. Several years ago severe rusting began to appear at the stone abutments where the cables entered the trench. Careful excavation found that more than fifty years of burial in the ground had taken a severe toll. Water running inside the cables had been trapped in the crevices of the concrete and corroded the embedded cable wires causing a reduction of as much as 40 percent in the strength of the cable. Since the cables outside the anchorages were found to be in excellent condition, it was decided to replace all four cable anchorages and both ends of each cable, a feat never before attempted.

New anchorages consisting of rock anchors and structural steel members embedded in concrete were constructed on either side of each existing cable anchorage to receive the ends of new parallel wire strands. These strands were brought to a point approximately 100 feet from the anchorage where a new splay cable band had been installed on the existing cable. At this point, wires of the existing cable were individually spliced to a companion wire of the new strand extending up from the new anchorage. A total of 8,400 individual wire splices were thus made to connect the existing cables under proper tension to their new anchorage ends. The new cable anchorages are completely open to inspection and easily accessible for

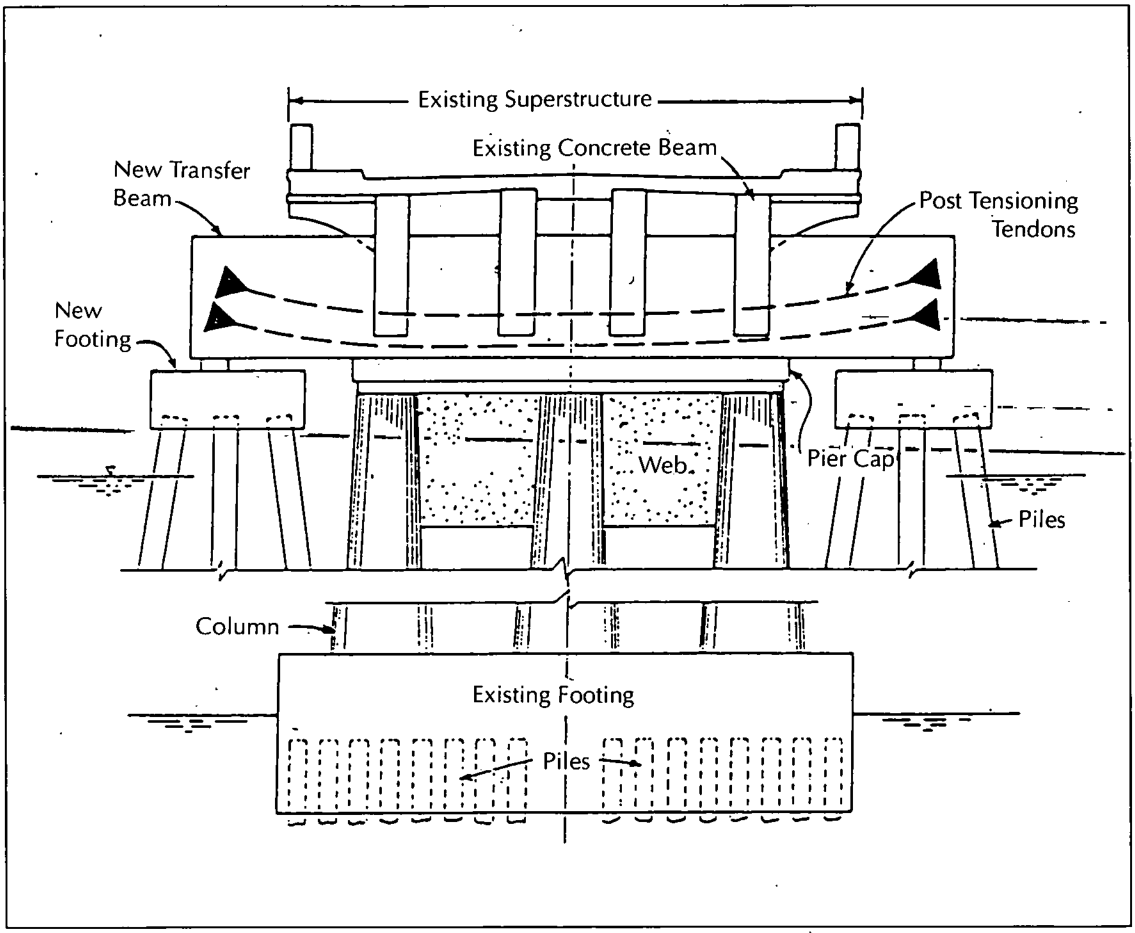

FIGURE 18. Foundation rehabilitation for the Ashley River Bridge.

maintenance.

The rehabilitation included replacing all suspenders that connect the bridge deck to the cables and completely reconstructing the wind cable system. Traffic was restricted on the bridge during all these operations.

Connecticut Bridge Rehabilitation Program

Spurred on by the tragic collapse of the Mianus River Bridge on the Connecticut Turnpike, the State of Connecticut embarked on a comprehensive Transportation Infrastructure Renewal Program that put the state into the forefront of bridge rehabilitation in the country. During a ten-year period that started in 1985, a total of 1,620 state bridges and 1,300 local bridges are scheduled for rehabilitation, reconstruction and improvement, at a total cost of $1.5 billion. Federal funds will pay for 80 percent of the costs on state and interstate highway bridges. The state will aid local governments in various ways in financing the costs for local bridge rehabilitation.

In the initial program, some 500 bridges were identified by state department of transportation engineers as being in poor to fair condition, with another 110 bridges expected to deteriorate to poor to fair condition each year of the program. Engineering contracts for the inspection and rehabilitation design for groups of bridges were initiated on an expedited basis; construction contracts followed the approval of consultant inspection reports and rehabilitation designs.

Work performed for this renewal program includes every imaginable item from improving foundations, repairing cracks and spalls in abutments, replacing bridge bearings, replacing rivets and corroded steel members, repairing cracks in concrete beams and bridge slabs, replacing bridge decks with new concrete slabs, grid floors and orthotropic steel decks to widening or replacing entire bridges.

Ashley River Bridge

While rehabilitating or replacing a bridge superstructure has become a familiar occurrence, the unusual replacement of a bridge foundation provides an excellent example of innovative engineering and imaginative design. The bridge crosses the Ashley River in Charleston, South Carolina, and connects the historic Charleston Peninsula with downtown and the state highway system. Completed in 1926, the bridge is 1,733 feet long and it carries 40,000 vehicles a day on its three lanes. The superstructure consists of a 202-foot long double-leaf bascule channel span, flanked by concrete T-beam approaches that have an average pier spacing of 76 feet. The approach piers consist of three-column bents with a concrete cap and partial depth diaphragms, resting on an unreinforced pile cap supported by precast concrete piles.

An underwater inspection in 1987 revealed extensive losses of column section, voids in the pile caps and major cracking. At one location, the tops of several piles were not in contact with the pile cap above. A preliminary analysis indicated that load and stability margins were severely reduced and the bridge had to be posted with load and speed restrictions.

Alternative studies of rehabilitation methods and total replacement were evaluated in the light of several considerations:

- Historic and architectural significance of the bridge;
- Speedy return to maximum traffic levels;
- Maintaining the reduced level of traffic during construction; and,
- Cost.

The solution that met all criteria favorably was the replacement of the deficient piers (see Figure 18).

Groups of new 24-inch square prestressed concrete piles were driven adjacent to each approach pier and capped with a new concrete footing. Cast-in-place concrete beams were then placed to span transversely between these new pile foundations to support the existing bridge superstructure. Specially designed falsework and concrete placement procedures had to be employed to support the weight of the freshly placed concrete without adding loads to the existing piers. Post-tensioning tendons and some normal reinforcing steel of the new support and load transfer beams were threaded through the existing T-beam webs. Shallow jacks placed between the top of the new footings and the bottom of the new support beams were used to transfer the superstructure load from the existing piers to the new footings prior to the grouting of the new permanent beam bearings.

A total of 18 piers were reconstructed at a cost of about $3.6 million. The entire operation from inspection and discovery of the foundation problem through design and construction was completed within one year.

Summary

The concept of regularly scheduled inspections has become well established throughout the country. Nevertheless, problems persist. Bridge failures, and the sudden unscheduled complete or partial closures of bridges such as New York City's Williamsburg and Manhattan Bridges, are all too frequent reminders of long years of lack of timely attention and "deferred" maintenance, of wasteful management and of utter disregard for the long-term needs and safety of the public.

Adherence to a long-range inspection program will permit the early identification and tracking of problems. It will enable the bridge operator — as was the case, for example, with the George Washington and the Golden Gate Bridges — to properly schedule necessary rehabilitation efforts with minimum impact on the community and maximum cost benefits. Fortunately, the importance of such programs is now generally recognized.

Bridge rehabilitation design requires familiarity with materials, equipment and construction procedures in addition to a detailed knowledge of the behavior of the structure. While there is a certain similarity in the problems befalling different bridges that permits typifying rehabilitation methods, details of application usually vary from structure to structure and demand the engineer's wide range of experience and full attention on an individual basis.

ACKNOWLEDGEMENT — *This article was originally presented to a meeting of the Boston Society of Civil Engineers Section/ASCE's Structural Group at the Massachusetts Insitute of Technology on November 21, 1989.*

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