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FINAL REPORT

EVALUATION OF CONVENTIONAL REPAIR TECHNIQUES FOR CONCRETE BRIDGES

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16. Abstract

This study presents the findings of a comprehensive assessment of conventional methods of condition evaluation and repair of concrete bridges. Case studies involving different techniques practiced in the US and Europe are presented in the report. The causes of deterioration of concrete are identified along with the methods of repair. The procedures for the recommended repair methods are outlined for the superstructure and substructure. The repair procedures include: patching, deck patching, crack injection, overlays, sealers, expansion joint repair, repair of prestressed concrete bridge girders using longitudinal external post tensioning and metal sleeve splice, bearings, scour, abutments, footings and pier rehabilitation.

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SUMMARY

The roads, bridges and other infrastructure in the U.S. are aging, confronting the travelling public and the engineering community with problems and concerns about maintenance and rehabilitation. The physical deterioration of the transportation infrastructure has received a great deal of attention in the literature.

This study presents the findings of a comprehensive assessment of conventional methods of condition evaluation and repair of concrete bridges. Case studies involving different techniques practiced in the US and Europe are presented in the report. The causes of deterioration of concrete are identified along with the methods of repair. The procedures for the recommended repair methods are outlined for the superstructure and substructure. The repair procedures include: patching, deck patching, crack injection, overlays, sealers, expansion joint repair, repair of prestressed concrete bridge girders using longitudinal external post tensioning and metal sleeve splice, bearings, scour, abutments, footings and pier rehabilitation.

Table of Conversion

To convert from	То	Multiply by	
Length			
Inch (in.)	Millimeter (mm)	25.4	
Foot (ft)	Meter (m)	0.3048	
Area			
Square inch (sq. in.)	Square millimeter (sq. mm)	645.2	
Square foot (sq. ft.)	Square meter (sq. m)	0.0926	
Volume			
Cubic inch (cu. in.)	Cubic meter (cu. m)	0.00001639	
Cubic foot (cu. ft.)	Cubic meter (cu. m)	0.02832	
Cubic yard (cu. yd.)	Cubic meter (cu. m)	0.7646	
Gallon (gal)	Liter	3.785	
Force			
Kip	Kilogram (kgf)	453.6	
Kip	Newton (N)	4448.0	
Pound (lb)	Newton (N)	4.448	
Pressure or Stress			
Kip/square inch (ksi)	Megapascal (Mpa)**	6.895	
Pound/square inch (psi)	Megapascal (Mpa)**	0.006895	
**a pascal equals one new	rton/square meter		
Mass			
Pound	Kilogram (kg)	0.4536	
Ton (short, 2000 lb)	Kilogram (kg)	907.2	
Mass (weight per length)			
Kip/linear foot (klf)	Kilogram/meter (kg/m)	0.001488	
Pound/linear foot (klf)	Kilogram/meter (kg/m)	1.488	
Pound/linear foot (klf)	Newton/meter (N/m)	14.593	

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CHAPTER 1

INTRODUCTION

1.1 Background

The roads, bridges and other infrastructure in the U.S. are aging, confronting the travelling public and the engineering community with problems and concerns about maintenance and rehabilitation. The physical deterioration of the transportation infrastructure has received a great deal of attention in the literature, the media, as well as by the legislative and executive bodies at the federal, state and local levels of government

Today, about half of the approximately 585,000 highway bridges in the United States are 45 years or older. The Federal Highway Administration estimates that about \$ 90 billion would be required to fully rehabilitate or reconstruct the bridges that are structurally deficient and those that are functionally obsolete. There is a clear need for increased research effort in the areas of bridge inspection, evaluation, and rehabilitation.

Bridge structures, like any other structure, deteriorate with time due to the inadequacy of design detailing, construction and quality of maintenance, overloading, environmental effects, abnormal floods and erosion. The maintenance of modern bridges has to take into account the damage caused by accidental or ground movement/subsidence. Maintenance needs to be done to preserve the intended load carrying capacity of the bridge and ensure safety of the public using it. Rehabilitation refers to restoring the bridge to the service level it originally was intended to have.

1.2 Objective

This study is focused on evaluation of conventional repair techniques for concrete bridges. The evaluation is illustrated with case studies involving different techniques practiced in the US, and Europe. The evaluation includes repair and rehabilitation techniques used by state departments of transportation in the United States

The causes of deterioration of concrete are identified along with the methods of repair. For the case studies, the causes of failure and deterioration are discussed and the method chosen for the repair is described and the performance evaluated. The procedures are provided for the most efficient and reliable repair methods and recommendations are presented in Volume 2.

1.3 Scope

Chapter 2 reviews the causes of deterioration of concrete and typical investigations of bridge decks. Chapter 3 presents the methods used for superstructure repair along with the case studies. This chapter deals with patching, deck repair, and bearings. Chapter 4 presents the repair methods used for substructure below the waterline, types of abutments and their rehabilitation, footings, jacketing of piles and piers. The summary of the repair techniques is presented in Chapter 5. The recommendations and procedures for the conventional repair methods are presented separately in a manual.

CHAPTER 2

CONCRETE DETERIORATION AND INVESTIGATION OF BRIDGE DECKS

2.1 Causes of deterioration

Concrete, being absorbent and permeable, is typically exposed to a variety of disintegrating agents. These aggressive agents are usually transported through the concrete by water through either capillary action or by pressure. The permeability of a prestressed concrete beam can result from such factors as, the air trapped in the concrete, water channels created by surface water, pores that were once occupied by water, voids caused by honeycombing, voids caused by segregation of the constituent materials, and small cracks near the ends of the beam caused during prestressing due to large tensile stresses around the anchorage zones.

While the permeability of the concrete allows disintegrating agents into the concrete, its ultimate durability is affected by weathering and exposure to reactive aggregates, chemical corrosion, and water containing sulphates, leaching and mechanical wear. Weathering includes freezing and thawing, alternating wetting and drying cycles, and temperature changes. The differential expansion and contraction caused by weathering often leads to cracking of the concrete.

Cracks can also be caused by the chemical reactions of several types of aggregate with cement high in alkali (more than 0.6 % alkali). The product of the reaction expands thus causing the concrete to crack. The high alkalinity (pH value of about 12.5 to 13.5) of Portland cement makes the concrete highly susceptible to reaction when exposed to an acidic medium.

Water containing sulfates from sodium, potassium, and magnesium affects the durability of the concrete in a number of ways. First, the product of the chemical reaction occurring between the sulfates and the hydrated calcium aluminate and hydrated lime present in the cement expands considerably thus causing the concrete to crack. Next, a purely physical deterioration of concrete is caused by crystallization of the sulfate salts, which accumulate following many wetting and drying cycles. The salt crystals create high tensile stresses, which can also cause the concrete to degrade.

Leaching, is yet another phenomenon that causes the concrete to degrade. Water containing calcium hydroxide that has leaked through the shear keys between beams, evaporates from the beam soffits, leaving behind aggressive icicle-shaped deposits of calcium hydroxide.

The steel in a prestressed concrete beam is also subject to degradation. Accelerated corrosion would take place, if the pH (alkalinity) is lowered. Typically, the steel is protected by the concrete surrounding it. The concrete, having a high alkalinity, acts as a barrier against corrosion causing agents.

2.2 Concrete evaluation

Thorough and logical evaluation of the current condition of the structure is the major step of any repair or rehabilitation. The determination in concrete structures is usually evident in certain form of visible distress such as cracking, leaching, spalling, scaling, stains, disintegration, settlement, or deflection. Generally, evaluation takes place as a result of certain visible sign of distress, causing structural and durability problems of poor functional performance, which, in turn, result in safety considerations.

A typical evaluation of concrete will include i) visual inspection, ii) review of

design and construction data maintenance and periodic inspection reports, iii) condition survey including deficiencies monitoring, joint survey, sampling and testing and structural analysis. Typical indicators of problems are i) cracking, ii) surface distress (spalling, surface disintegration and scaling), iii) water leakage, iv) movements (deflections, heaving and settlement, v) metal corrosion (rust staining, exposed post-tensioned cable strands and exposed reinforced bars).

- The U. S. Bureau of Reclamation (Bureau) identifies the following steps for standard concrete repairs in new concrete as well as old concrete damaged by long exposure to field conditions: i) determine the cause of damage, ii) evaluate the extent of damage, iii) determine the need to repair, iv) choose an appropriate repair system, v) prepare the old concrete, vi) apply the repair system, and vii) cure the repair properly.
- i) Cause of damage: It is essential to correctly determine the causes of the damage to the concrete. If this is not done properly, or if the determination is incorrect, the same cause will most likely lead to attack and deterioration of the repair. The resources spent for such repairs are, thus, totally lost and larger replacement repairs become necessary at much higher cost.
- ii) Extent of damage: The objective of this step is to determine the extent of damage in the structure.
- iii) Need to repair. Not all the damage to concrete requires repair. Repairs should be undertaken, only if they will result in longer or more economical service life, a safer structure, or necessary cosmetic improvements in the structure. This step also includes determination of when the structure can be taken out of service for repairs, an estimate of time for the repairs, and cost of the repairs.

The Bureau states that first three steps are the major components of a condition survey. Only after they have been properly performed, one should proceed with selection and installation of the repair materials.

- iv) Choice of an appropriate repair system: Upon completion of the first three steps, an appropriate repair system can be selected that takes into consideration the many factors essential to a successful repair.
- v) Preparation of the old concrete: The most common cause of repair failure is improper or inadequate preparation of the old concrete prior to application of the repair material. Even the best of repair materials would give poor service life, if bonded to weakened or deteriorated old concrete. It should be noted that each repair material has special preparation requirements.
- vi) Application of the repair system: Each standard and non-standard repair material has application procedures specific for that material. The procedures used with replacement of concrete depend on the concrete type. For example, the procedure for polymer concrete is quite different from that used for epoxy bonded concrete. It is essential that proper application techniques be identified.
- vii) Curing of the repair. The second most common cause of repair failures is improper or inadequate curing. Each repair material has specific curing requirements. As an example, replacement concrete benefits from long periods of water curing, while latex modified concrete requires 24 hours water curing followed by drying to allow formation of the latex film.

2.3 Bridge inspection (Raina, 1996)

2.3.1 General

The overall objective of the bridge maintenance management system is to identify the need for structural maintenance, rehabilitation and

replacement, and, provide guidelines and methodologies to enable local engineers to reach rational, cost-effective decisions regarding maintenance and rehabilitation for bridges and other highway structures.

For decision-makers the decision of whether to rehabilitate or to replace a deficient structure, and its subsequent justification, has not always been easy. One principal cause for this has been the approach of piecemeal synthesis in decision-making that has been oriented toward emphasizing certain advantages of an alternative and underestimating its disadvantages.

A 'systems' approach will result in a coordinated step-by-step analysis, which, when applied to the maintenance of bridges and other structures, will integrate essential elements of reliable information, well-defined criteria, clearly perceived constraints, and uniform evaluation of all the available alternatives. Further, this will allow for and encourage the use of experience, judgement, and analysis of the impact of certainty and possible future decisions, ensuring the optimal or near optimal use of public funds.

Maintaining highway bridges and keeping them in fit condition to provide safe and uninterrupted traffic flow, is the primary function of a bridge maintenance engineer. Protection of the investment in the structure-facility through well programmed repairs and preventive maintenance, is second only to the safety of traffic itself. To achieve the desired result requires constant alertness and thorough inspection procedures.

2.3.2 Categories of bridge inspection

All remedial and preventive maintenance or repair work, including replacement of components, should be planned in time, and economically, with minimum inconvenience to traffic. Original completion reports must be available for all

bridges, and these should form the basis for detailed periodic bridge inspections. The data thus collected should be properly evaluated from time to time to assess the need for remedial measures required to be undertaken. Broadly, the following three categories of bridge inspection need to be conducted to collect the performance data of bridges.

Routine inspection

These are broad general inspections, carried out quickly and frequently by highway maintenance engineers having reasonable practical knowledge of road structures, though not necessarily any specialized knowledge in design details or special construction problems of any particular bridge or expertise in special problems of bridge inspection. The purpose of this routine inspection is to report fairly obvious deficiencies, which could lead to accidents or future major repairs/maintenance problems. Such inspections should be carried out monthly.

Detailed inspection

This type of inspection can be of two categories, viz., general and major, defined by the 'frequency' and 'intensity' of inspection, respectively. 'General inspection' could be made at yearly intervals, and it should cover all elements of the structures against a prepared checklist. It would be mainly a visual inspection supplemented by standard instrument aids. A written report must be made of the conditions of the bridge and of its various parts.

The 'major inspection' should be more intensive and would require detailed examination of all elements, even requiring setting up of special access-facilities where required. Such inspection, depending upon the importance of the structure, could be spaced between 2 and 3 years, and even smaller intervals for sensitive designs, or for bridges in aggressive environments.

Special inspection

This could be undertaken to cover special circumstances such as occurrences of earthquakes, passage of high intensity loadings, unusual floods, etc. These inspections should be supplemented by testing as well as structural analysis, and hence the inspection team should have an experienced bridge design engineer available to them.

It is important that inspections are undertaken in those periods which offer the most critical evaluation of the performance of the structure. For example, items such as foundations, protective works, scour effects, flood levels, etc., should be inspected before, during and after the floods; bearing and joints should be inspected during temperature extremes; etc. The frequency of routine inspections could be determined by the importance of the structure, environmental conditions, and cost. The frequency indicated above may be considered as a guide. A comprehensive check-list of items related to the form, material, condition, and situation of the structure, should be drawn-up and followed by the inspecting team.

Besides being a qualified engineer, the inspection team leader should become familiar with design and construction 'features' or the bridge to be inspected, so that observations can be properly and accurately assessed for a meaningful report. His competency to recognize any structural distress or deficiency and assess its seriousness with complete recommendations for appropriate repairs, are important prerequisites for entrusting this assignment to him.

2.3.3 Inspection and investigative-structural-computations

This comprises of Activities I and II as described below:

Activity I

This shall include the following:

i) Detailed visual inspection of each element of the structure and its protection works from close range (not just looking for a mere overview using binoculars from distance).

Widths of structural cracks more than 0.3 mm and any signs of deterioration and distress should be recorded; in a structure, this will range from the usual non-load-induced cracks (caused by drying shrinkage, early removal of shutters, plastic cracks, lack of curing, etc.) to serious structural cracks, and manifestation of distress zones. In the protection works and channel configuration, it will range from nothing of concern to serious undermining/ scouring/ dislodgement and choking of waterway and diversion in water-course. All this has to be included in the report for the Activity 1, giving necessary indicative sketches.

- ii) If the observed signs and manifestations of deteriorations and distress are such as can be adequately taken care of by routine type of repair and restoration work, then this report shall also give detailed methodology of the repair-work along with its specifications, quantities and cost estimate, together with workman- like sketches and notes for execution. In addition, the report shall list out the details of the causes that lead to the observed deterioration.
- iii) If the observed manifestations of distress in the structure are so serious as would require a detailed structural investigation (computations, and possibly some tests) in order to enable a decision between 'repair' and 'part or complete demolition and replacement', then this report shall indicate so in detail (describing the likely causes of deterioration) and seek permission for taking up such work, which will then form Activity II.

The interim report shall propose various restorative measures to be taken to arrest furthering of the distress and maintaining the usability of the structure (for the time being) until the suggested investigative work is carried out and correct conclusions and recommendations drawn-up.

iv) If the observed manifestations of distress in the structure are clearly such as would require its outright demolition (and replacement), then the report shall clearly say so, giving explicit supporting reasons and the details of the likely causes that obviously lead to such distress.

Activity II

Upon receiving the necessary permission to take up this activity with respect to a particular structure, the designer shall then carry out all the necessary investigative structural computations (and tests, if needed), draw the necessary conclusions, detail out the relevant recommendations and submit all these in the report on Activity II.

For the work under this activity, use shall be made of the 'as-built' drawings and other relevant information (as available). All necessary structural investigative calculations shall be carried out to investigate the structural stability and estimate the material stresses at all critical sections under the operational conditions and possible load combinations. The current legal loading as well as the operating loading (if heavier) shall be used in the analysis. Appropriate conclusions shall then be drawn about the adequacy of the structure, complimenting the investigation with relevant tests, if found necessary. Based on this, the report for Activity II shall clearly and unequivocally offer the most appropriate and technically sound recommendations for restoration of the structure, with full details of repairs, their specifications, quantities, and cost estimate.

However, if restoration is found either technically not feasible and/or economically not viable, then the report shall suggest the most appropriate method for demolition with possibility of re-use of any part/parts of the structure, describing the necessary procedure and precautions.

2.3.4 Rating the condition of an element

In order to standardize the various condition-states of any bridge-element (so that uniformity of expression and understanding of the distress state prevails), it is necessary and convenient to rate each condition-state and give it a numerical designation. One of the convenient Numerical Rating Systems, ranging from 0 to 7 and N/A and U, that may be followed, is as follows.

Rating	Guideline definition
U	'Unknown' applies to components under inspection for which information is unavailable (such as footings below the ground line or the foundation piles under pilecaps). In certain cases these items can become exposed and be rated
N/A	'Not applicable' applies to elements called for on the inspection forms (because these elements exist on other bridges) which do not exist on the bridge under inspection. For instance, some bridge piers and abutments have bearing pedestals and others support the bearings directly on the abutment seat or pier cap beam.
7	'New' or 'Like-New' Condition, no sign of distress or deterioration. No repairs necessary.
6	'Good condition' no repairs necessary.

Rating	Guideline definition
5	'Functioning as Originally Designed' - insignificant deterioration or distress and does not reduce the capacity of the elements under inspection nor their ability to function. For example, a bridge expansion bearing, which is corroded but has not lost any effective strength and still permits the required movements. Minor repairs can be made to alleviate distress or eliminate deterioration.
4	'Minimum Adequacy' - immediate habilitation of affected elements required to maintain design loading capacity.
3	'Not Functioning as Originally Designed' - serious deterioration (and/or distress), sufficient to reduce the element's structural capacity and/or its ability to function as designed. When this rating applies to primary elements, the bridge must have the maximum design loading reduced accordingly. Immediate repairs must be made to return the structure to design capacity.
2	'Structurally Inadequate' - deterioration or distress so well advanced as to indicate the closing of the structure to all traffic pending immediate load rating analysis. This rating obviously applies to primary members only.
1	'Potentially Hazardous' - such a rating in primary members implies there is a danger of collapse under any further use of this structure and bridge should be closed to traffic immediately. When such rating applies to secondary elements, it can be the cause of vehicular or pedestrian accidents and should be corrected immediately.
0	'Dangerous' - bridge already closed, conditions beyond repair, imminent danger of collapse or already collapsed. Structure to be demolished.

Table 2.1 Numerical rating system

2.4 Patching of superstructure and substructure (Weyers, 1993)

Patching involves methods used to restore the structural integrity and appearance of deteriorating concrete bridge substructure and superstructure elements such as piers, pier caps, diaphragms, beams, and abutments. The depth of the patch may be shallow (to the level of reinforcing steel) or deep (a minimum of 0.75 in. (19 mm) below the first layer of reinforcing steel. Patching materials may be Portland Cement Concrete (PCC), quick set hydraulic mortar/concrete, or polymer mortar/concrete. In the case of deterioration caused by corrosion of reinforcing steel, patching material is normally limited to PCC.

2.4.1 Patching materials for concrete structures

Depending on the size, location, and the general function of bridge components, various materials are available for repair. The following influences selection of materials: a) compatibility of the material to the original concrete, b) environmental considerations, including aesthetics, c) cost effectiveness, d) expected service life, e) availability, and f) familiarity of the contractors with the material under consideration.

In repairing a concrete spall, the following requirements in the material selection should be satisfied:

- i) Properties of the repair material should be as close as possible to the existing concrete, particularly with respect to the coefficient of expansion and the modulus of elasticity.
- ii) Strength should be at least as high as that of the original concrete.
- iii) Repair material should have low shrinkage, low permeability, and a low water /cement ratio to prevent moisture and chloride penetration.
- iv) Repair material should adhere to the concrete substrate, either by applying a

rich cement mix or epoxy bonding compound to the prepared concrete surface before placing the new concrete.

v) Color and texture should match the original concrete as much as possible.

Selecting materials that meet all the necessary properties established by conditions and requirements is difficult. Most materials used for repairs use portland cement binders and well proportioned aggregates. Durability for these materials can be increased using special pozzolans (microsilica), polymers latex), or admixtures that reduce permeability. Most modified concretes and mortars can be easily used, if one has experience in how these materials behave during placement and while curing. The use of portland cement based repair materials requires special attention to shrinkage and curing. All repair materials used should have low shrinkage properties, and proper curing is critical in reducing early shrinkage and for future long-term performance.

In structural applications, it is important to understand the repair materials' response to loads. Two important properties for load sharing applications are elastic modulus and compressive creep. In understanding the material properties, it is important to understand the exposure and service conditions to which the selected materials will be subjected. For instance, it has been demonstrated that the addition of latex to modify cement based repair materials causes the flexure creep value to soar under high humidity conditions, but most of the reported material properties are evaluated under at low relative humidity and therefore, may appear acceptable.

The use of experimental materials or materials that contain unknown ingredients that could lead to unnecessary problems should be avoided. For instance, the use of materials containing gypsum results in uncontrolled expansion and extremely low durability when subject to moisture. Also, the high heat of hydration of high exothermal materials such as magnesium phosphate based

materials can cause thermal cooling stresses. Some materials are sensitive to the method of application. Latex modifiers have proven exceptional in bridge deck overlays, but, when used in some applications involving dry mix shotcrete, have resulted in interbond failure.

Polymer concretes and mortars are the other major class of materials used to repair concrete surfaces. Epoxies and acrylics blended with graded aggregates produce strong and chemically resistant materials. They can be used for thin application or thick applications, where the service exposure conditions do not cause dimensional incompatibility problems. Polymer materials have high thermal coefficients as compared to concrete. Except for thin surface coating systems, they should not be used in solar exposure situations.

The following are the most common repair materials: latex modified concrete and mortar, epoxy patching compounds, polyester resin, acrylic concrete and mortar, polymer-modified cement based materials, pozzolanic modified concrete, high alumina cement compounds, magnesium phosphates, molten sulfur, calcium sulfate based materials, non-shrink quick setting mortar cement based polymer concrete, and pneumatically applied mortar (shotcrete).

2.4.2 Cast-in-place portland cement concrete (PCC)

Description

PCC is used to backfill the prepared cavities of corrosion damage concrete members. The damaged concrete is normally removed to a depth of 0.75-in. (1.9 cm) below the first layer of reinforcing steel.

Disadvantages

Substructure and superstructure patches have a relatively short service life, because they do not address the cause of the deterioration mechanism and corrosion of the reinforcing steel, but merely the symptom. Consideration must be given in patching to the influence of the amount of concrete removal on the reduction of the structural capacity of the structure remaining in place after removal.

Estimated Service Life

The service life of substructure and superstructure PCC patches is somewhat dependent on the type of patch (shallow or deep) and whether the leaking deck joints are successfully repaired. The best estimate of service life of substructure and superstructure PCC patches is 5 to 10 years for elements exposed to either splash and spray or water runoff due to melting snow.

2.4.3 Shotcrete

Shotcrete (also called Gunite) is a pneumatically applied portland cement mortar/concrete used to patch substructure and superstructure elements. The damaged concrete is normally removed to a depth of 0.75 in (1.9 cm) below the first layer of reinforcing steel.

Disadvantages

Shotcrete repair methods do not address the cause of the corrosion deterioration but merely the symptoms. Thus the service life of the shotcrete repair is limited.

The removal of large quantities of concrete at one time may cause a reduction of structural capacity that needs to be considered. Quality of shotcrete repairs is highly variable and generally operator dependent.

Estimated service life

The best estimated service life for shotcrete repairs of substructure and superstructure elements exposed to either spray and splash or melt water runoff is 10 to 15 years.

2.4.3.1 Case study

Cuyahoga river bridge, Ohio - (Freeh, 1984)

This bridge was opened to traffic in 1955. The twin two-lane Ohio Turnpike bridges over the Cuyahoga River valley reach a height of 53 m above the valley. Each bridge has four 30 m spans and nine 76 m spans with 12 reinforced concrete piers. As the use of deicing salts increased during the 1960's, the deterioration also increased in the concrete portions of the bridges. In mid 1970's, efforts were made to divert the drainage and patch the piers but the deterioration continued. By 1980, about 40 % of the surface area of the piers were spalled or nearly spalled and thorough rehabilitation using shotcreting was proposed.

2.5 Field investigations of typical concrete bridge decks (Samples, et al 2000)

2.5.1 Initial visual inspection

Deicing salts on bridge decks was used beginning the late 1950's, and corrosion of the reinforcing steel related to the use of deicing salts emerged as a problem in the 1960's. Epoxy-coated reinforcement was first proposed as a solution to the problem of bridge deck deterioration due to reinforcing steel corrosion in the

early 1970's. Since epoxy coated bars were thought to significantly increase the service life of bridge decks, it was generally accepted that epoxy coated bars were cost-effective.

The Indiana Department of Transportation provided 114 bridge decks for the field evaluation. The bridge decks were constructed during the period 1972 to 1980. A requirement in the selection of the bridge decks was that no rehabilitation method was done besides patching.

The initial visual inspection included taking photographs and videos of the bridge deck, obtaining concrete cover measurements, recording crack patterns and rust stains, and measuring areas of spalling, scaling, and delaminating/debonding. The bridge decks were categorized by the corrosion protection method utilized in its construction.

- 46 bridge decks constructed with black reinforcing bar and Class C concrete with 38 mm (1.5 in) latex modified concrete overlay.
- 28 bridge decks constructed with epoxy-coated reinforcing bar and Class concrete.
- 8 bridge decks constructed with black reinforcing bar and Class C concrete.
- 4 bridge decks constructed with galvanized steel and Class C concrete.

Results of initial visual inspections

A survey of one lane and shoulder was conducted on all bridges. Preliminary analysis of the 114 bridge decks surveyed showed that 46 % had signs of distress. Signs of distress include spalling, areas of delaminating/debonding, and rust stains. Cover measurements were taken on 106 of the surveyed bridge decks. The average cover reading on 36 % of the decks was below the specified design cover. Table 2.2 shows a breakdown of the bridges by the types of

corrosion protection method and the percent showing any sign of distress, percent with significant distress (greater than 6 % of the surveyed area showing distress), and percent with severe distress (greater than 20 % of the surveyed area showing distress). Figure 2.1 and 2.2 show examples of distress observed on bridge decks containing uncoated reinforcement.

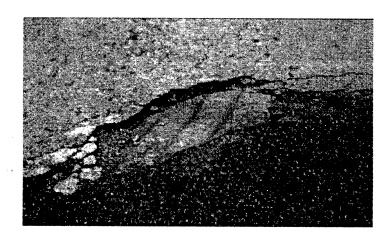


Figure .2.1 Spall with reinforcement exposed

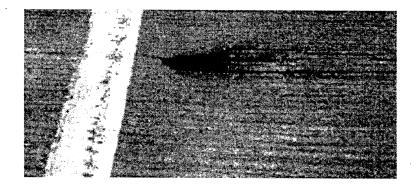


Figure 2.2 Rust stain in area of crack

Corrosion protection method	Percent of bridges surveyed showing any sign of distress	Percent of bridges with > 6 % area of distress (significant)	Percent of bridges with > 20 % area of distress (severe)	Years of construction
Black rebar and				
Class A concrete	71 % (20 of 28)	29 % (8 of 28)	14 % (4 of 28)	1972 to 1976
Black rebar and Class C concrete with 38 mm (1.5 in) LMC overlay	52 % (24 of 46)	15 % (7 of 46)	2 % (1 of 46)	1974 to 1980
Galvanized rebar with Class C concrete	50 % (2 of 4)	25 % (1 of 4)	0 % (0 of 4)	1976
Black rebar with Class C concrete	38 % (3 Of 8)	12.5 % (1 of 8)	12.5 % (1 of 8)	1973 to 1980
Epoxy-coated rebar with Class C concrete	11 % (3 of 28)	0 % (0 of 28)	0 % (0 of 28)	1976 to 1980

Table 2.2 Results of initial visual inspections

Summary

Field investigation of 114 bridge decks in Indiana indicated corrosion related distress in 46 % of the bridge decks surveyed. The initial field investigations also found 36 % of the bridge decks surveyed had an average cover reading below the specified design value. Epoxy-coated reinforcement combined with Class C concrete provided the most successful corrosion protection method, as only 11 % of the bridge decks in this category showed distress, and none of the bridge decks had any significant damage. This percentage was the lowest of all categories of corrosion protection methods. Uncoated reinforcement and a design cover of 25.4 mm (1.0 in) Class C concrete with 38.1 mm (1.5 in) latex modified overlay was the second least successful corrosion protection method.

In this category, 52 % of the bridge decks showed signs of distress. The range of age of construction for this method is comparable to that of epoxy-coated bridges. To date, the studies showed that epoxy-coated bars have performed satisfactorily in Indiana. However, damage of the epoxy coating may decrease the effectiveness of epoxy-coated reinforcement as a corrosion protection method. To increase the effectiveness of epoxy-coated reinforcement, sufficient cover of high quality concrete and increased thickness of epoxy-coating can be used.

CHAPTER 3

SUPERSTRUCTURE REPAIR

3.1 Introduction

3.1.1 Types of superstructure damages

The types of damages that occur in the superstructure are described in the following:

Wearing surfaces:

Asphalt: map cracking, edge cracks, reflection cracks, lane joint crack, corrugations, shrinkage cracks, slippage cracks, distortion, and disintegration.

Concrete: scaling, spalling, and cracks.

Deck slabs: scaling, cracking, and spalling.

Expansion joint: leakage, loosening of supports, and failure of sealant.

Primary members: deterioration, impact damage of stringers or girders, rusting of elements, and cracking.

Secondary members: deterioration due to spalling of concrete and corrosion of the reinforcement, impact damage of exterior diaphragms and struts.

Bearing: build up of debris, loss of protection system, corrosion and delamination, movement of bearing, and excess shear deformation

3.1.2 Philosophy of repairing and replacement

The choice to repair or replacement is obviously the most critical decision in the selection of the improvement scheme, and involves a cost effectiveness analysis and service life estimates for various scenarios. In some situations, it may be

possible to choose repair or replacement from a consideration of certain obvious criteria. In other cases, a systematic analysis of options, economic models, technical factors, and funding constraints may be necessary before the choice becomes obvious and acceptable to the agency. Within these various approaches, the following sequence is suggested.

- i) Identify relevant factors and objectives.
- ii) Judge how effectively the repair or replacement option meets the criteria of Step (i).
- iii) Analyze the costs of repair and replace alternatives.
- iv) Make a preliminary decision. This may be (a) repair, (b) replace, or (c) decision not obvious, and hence conduct further studies.

Step i) The factors most likely to influence the decision are: extent and level of deficiency, functional obsolescence of bridge, present and anticipated traffic volumes, period of traffic disruption, rating and remaining life of bridge if rehabilitated, local construction capability for the option selected, local considerations (public perceptions, institutional policies, available funding sources), essentiality of structure within the net work system, environmental and aesthetic considerations; and safety factors. This list is by no means complete, and may be expanded to include other criteria.

Step ii) The effectiveness of a choice (repair or replace) may be examined in terms of the factors considered in Step (i). Where this assessment is arbitrary or has a quantitative basis is decided by the engineer and the supervising agency. At this stage it is desirable to introduce some numerical index by which possible options can be compared. This index should also indicate how well a proposed option can meet the needs of the project. For example, if the decision is to repair the bridge, it is essential to check the ability of the repaired structure to satisfy the geometric criteria and the capacity rating.

Step iii) Cost data and unit prices for repair and construction items should be obtained from sources within the supervising agency, and should reflect the most reliable information available.

Stop iv) A repair or replacement option may at this point be obvious, and the decision justifiable. For example, a comparison of alternatives may be based on first cost (initial construction cost). If this cost for a repair option is, say, less than 30 percent of the replacement cost and the repaired bridge meets the operational requirements, repair should be the choice. If, on the other hand, the repair cost necessary to upgrade the bridge to current standards is 70 percent (or higher) of the replacement cost, replacement would probably be the choice. Whereas these ranges can be used as a guideline, they may also indicate the demarcation line between improvements.

3.1.3 General repair methods and requirements

Table 3.1, which matches the failure mode to possible repair procedures, is shown below. In general several options exist for each damage category. Thus the designer has to choose the appropriate procedure and compute the amount of restoration expected.

Patching
×
×
×
:
×
×

Table 3.1 Repair procedures for typical failure modes

3.2 Deck Repair

3.2.1 General

The deck is the element of the highway bridge structure that transmits traffic loads directly to the main superstructure system. Any deck must be evaluated based on its ability to perform this function properly. Since the deck is directly affected by traffic loads, it is most susceptible to traffic-related problems, such as the corrosive effects of deicing chemicals, ever increasing live loads, and impact loads that materially increase as the deck surface deteriorates.

3.2.1.1 Typical deck repairs

Depending on the depth of deterioration and corrosion, the following deck repair procedures are normally adopted.

i) Shallow repair

Where the depth of concrete deterioration is less than ¾ inch (19 mm), and reinforcing steel is not exposed, a shallow repair is needed (Figure 3.1). The defective concrete is saw cut ¾ inches deep into rectangular or square shapes, and removed either by a pneumatic hammer or hydrodemolition. The surface is cleaned thoroughly, and the repair material, generally a prepackaged nonshrink, quick-setting polymer-modified cementitious mortar, is applied and cured. Where the repaired deck slab is to be overlayed later, the contractor may be given the option to fill the shallow repair areas with overlay material at the time of overlay application.

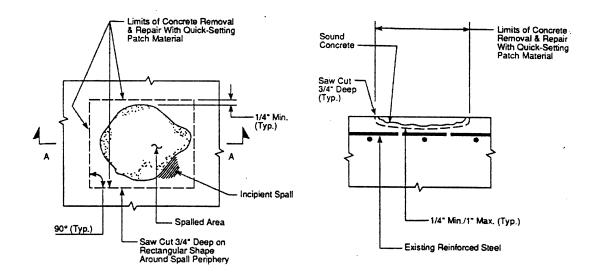


Figure 3.1 Shallow repair of deck slab (Brinckerhoff, 1992)

ii) Deep Repairs

Defective concrete deeper than the top mat of steel reinforcement requires a deep repair. The deteriorated concrete is removed as for a shallow repair, but the top reinforcing mat is completely exposed (by removing I inch of concrete below the top rebars) (Figure 3.2). Subsequently, these bars are thoroughly cleaned by sandblasting or hydrodemolition, and if there is section loss supplemental bars are added. Then the repair material is applied. Where large quantities are indicated, usually conventional concrete is used. After the concrete substrate is cleaned with high-pressure water or air jet, an epoxy bonding coat is applied to the surfaces, the reinforcing mat, and the bottom of the rebars. Before the coat hardens the concrete is placed, screeded, and cured. If a quick-set- ting mortar is used the epoxy bonding coat is normally applied only to the reinforcing bars, since the mortar has a good bond and will adhere to the concrete substrate.

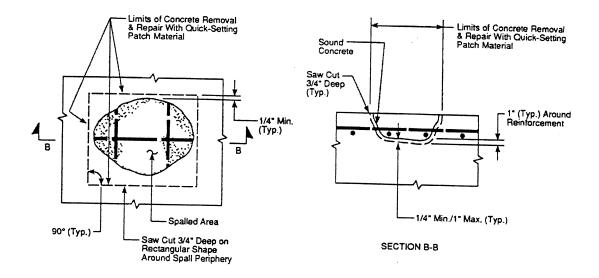


Figure 3.2 Deep repair of deck slab (Brinckerhoff, 1992)

In deep repairs, it is not prudent to place the repair material simultaneously with the overlay. If latex-modified concrete is used, the variation in the depth of repair would probably cause shrinkage cracks around the periphery. If low-slump dense concrete is used, it would be difficult to coat the concrete and rebar surfaces just prior to the placement of the overlay.

In bridge decks where spalls exceed 30 to 40% of the total deck area and where deck replacement is not warranted, a common treatment is to remove the contaminated upper deck concrete, curb to curb, below the top reinforcing steel; place class A concrete to one inch above the top steel and overlay it with latex-modified concrete; alternatively, build up to the final deck elevations with low slump dense concrete.

iii) Total Deck Replacement

Where the concrete deterioration extends below the top half of the deck slab, total deck replacement is indicated. As in other repairs, the deteriorated concrete is saw cut and removed. In preparation for this removal, traffic or waterways

below the bridge are protected from falling debris by proper shielding. After the concrete is removed, forms for the bottom surface of the deck slab are installed and suspended from above (Figure 3.3). The procedures then followed are the same as for deep repairs. When concrete has hardened, the wires supporting the forms are cut, and the forms are removed.

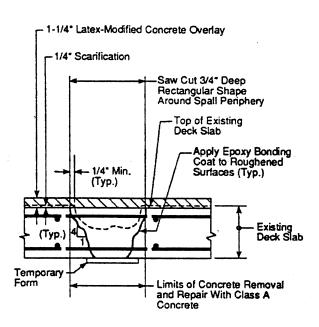


Figure 3.3 Total-depth repair of deck slab (Brinckerhoff, 1992)

3.2.1.2 Remaining Life of the Structure

Several considerations concerning the existing structure need to be evaluated before a decision can be made to either rehabilitate or replace the deck:

i) General Condition

The general condition of both superstructure elements must be assessed to determine whether the structure can be expected to perform for the extended period of time afforded by deck improvements. If these major elements are in reasonably good condition, or if normal maintenance can return them to a

reasonably good condition, deck improvements may be justified. If either the superstructure or the substructure is severely deteriorated, then deck rehabilitation or replacement may not be economically justifiable.

ii) Deck Joints

One of the important aspects of the general condition of the structure is the function of the deck joints. In many structural configurations, the joints are of primary importance to the long-term durability of the superstructure.

Most older designs utilized simple-span, noncomposite construction, with many joints, both sealed and unsealed. These structures have exhibited many joint-related problems through the years. Modern construction has attempted to limit the number of joints to reduce these problems. Maintenance records have shown that joints cause major maintenance problems, for the riding surface itself, for the supporting elements in the vicinity of the joints, and for the overall behavior of the superstructure system.

Deck rehabilitation or replacement can address some of the problems associated with deck joints. Joint replacement, alone, can be a form of rehabilitation, when problems can be traced directly to malfunctioning joints. In most instances, however, deck joints are a secondary consideration, with the deck itself being of prime concern.

Whenever deck rehabilitation or replacement is necessary, the joints need to be carefully considered. Some deck replacements have involved elimination of the joints entirely. By eliminating joints, many problems can be alleviated, including rough riding surfaces and damage to underdeck elements from water leakage. When joints are eliminated, however, the impacts on the behavior of the structure must be carefully investigated and addressed. The purpose of each joint must be established and understood, and the effects of its

elimination must be traced back to the original structural system.

For example, deflection joints - those joints that allow rotations of supporting elements, but no longitudinal movements - when locked, must carry live load forces they were not originally intended to carry. In addition, when expansion joints are eliminated, not only the live-load forces, but rotations and horizontal movements as well must be traced through the structure to their final destination, as well as an evaluation of all the elements affected along the way. The function of the bearings must also be fully understood in the process. Once the new deck has been designed to accommodate the new forces, rotations and movements, the superstructure elements must be investigated and upgraded, if necessary, for the new configuration.

iii) Original versus current live load requirements

Since decks in need of replacement are usually more than 20 years old, it is likely that the live-load criteria for the structure have changed since it was originally designed. One of the first elements to be investigated should be the current traffic conditions, and magnitude of loads as well as number of vehicles compared to those used in the original design. In addition, future live-load requirements should be evaluated, since the expected service life of the rehabilitated structure will allow it to function over an extended period. Historic traffic data, such as shown in Figure 3.4, can be of assistance in predicting future traffic demands. Toll facilities, such as the Marine Parkway Bridge usually have excellent records of patronage, which can be used to assess actual historical and potential future live loads, to determine the remaining fatigue life. In the absence of such records, a reasonable estimate must be developed through discussions with maintenance personnel or other parties familiar with the structure and its history as well as its possible future use.

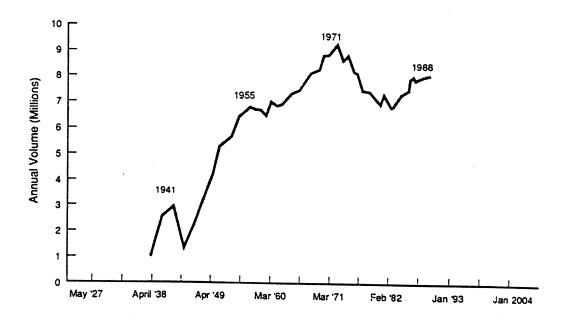


Figure 3.4 Traffic history at the Marine Parkway Bridge (New York City).

(Brinckerhoff, 1992)

iv) Configurational adequacy

In conjunction with the consideration of original design live load versus actual present live load, an investigation of the adequacy of the configuration of the existing deck should be made. It is common to find that safety standards have changed since the structure was originally designed. Before embarking on a rehabilitation program, the functional adequacy should be reviewed.

Some of the elements often found to be inadequate are lane widths, traffic barriers, shoulders, and side- walks. Many of these elements can be improved or even brought up to current standards during the rehabilitation effort, but occasionally the substandard elements will require extra measures to upgrade the system to current standards. When extra measures are necessary, the economic implications need to be addressed. Widening can rarely be accomplished without major superstructure strengthening even with the

currently available lightweight deck materials.

v) Remaining Fatigue Life

An important consideration with steel superstructures includes the magnitude of live-load stresses and the number of live-load stress cycles that the structure has experienced during its life. With the current understanding of stress ranges and the criteria that have been established as a result of investigations performed in the past 10 to 20 years, the remaining fatigue life of steel structures can be estimated reasonably accurately.

AASHTO has published Guide Specifications for Fatigue Design of Steel Bridges (1989) and Guide Specifications for Fatigue Evaluations of Existing Steel Bridges (1990). The former is used in the design of new structures while the latter is directed toward existing structures and has provisions for developing the remaining fatigue life of existing structures. The guide for existing structures presents an alternative methodology for determining fatigue life. The formulae presented are difficult and not straightforward and rely heavily on empirical factors and adjustments.

There are other design aids available, to assist in determining remaining fatigue life(AASHTO, 1974). The first step in determining the remaining fatigue life is the determination of the number of actual stress cycles that have occurred over the history of the structure. The methodology takes into account the number of stress cycles given the average daily truck traffic (ADTT) and known life (days) of the structure. This can be expressed as follows:

Where N = actual number of stress cycles
ADTT = average daily truck traffic

DL = life to date, in days

 α = member stress factor: 0.8 for transverse members and 0.7 for longitudinal members

C = a factor relating actual stress range to the gross vehicle weight distribution

Miner's linear fatigue damage equation provides a method of determining the remaining available cycles at a given stress range:

$$n_1 / N_1 + n_2 / N_2 = 1$$
(3.2)

where n_1 = the actual number of cycles that have occurred at the historical stress range, calculated above

 N_1 = the allowable number of cycles at the historical stress range (Figure 3.5)

 n_2 = the available number of cycles at the future stress range

 N_2 = the allowable number of cycles at the future stress range (Figure 3.5)

When the actual remaining cycles have been determined, design life formula (Equation 3.1) can then be used again to determine the remaining life, replacing historical data with expected future data.

For the deck replacement study of the Marine Parkway Bridge, mentioned earlier, the known information was the ADTT, which was 140 trucks per day, and the DL, which was 54 years, or 19,710 days. The resulting number of stress cycles to date was computed and found to be approximately 500,000 for the transverse floorbeams, which were considered to be the most critical elements for the structure. Future ADTT was estimated at 370 trucks per day, and an HS25 live loading was used for the future live load stress levels. The number of cycles to date was then substituted in Miner's equation, along with the allowable cycles at the historical HS20 stress levels (12 ksi) and the allowable

number of cycles at the proposed future HS25 stress levels (15 ksi) evaluated from Figure 3.5, resulting in a remaining fatigue life of 12 years. This conclusion implied that the floorbeams would need additional strengthening, if they were to be incorporated into the final structure and utilized at the HS25 live-load levels.

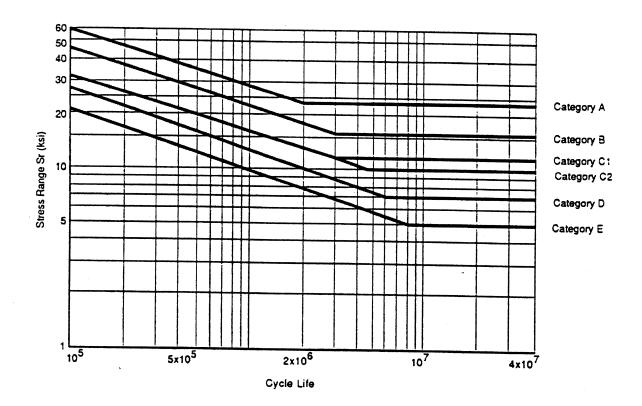


Figure 3.5 Number of cycles vs. stress range (AASHTO, 1974)

3.2.2 General bridge deck repair procedures

3.2.2.1 Crack injection

i) General

In reinforced concrete, cracks wider than about 0.3 to 0.4 mm should be sealed and filled by injection. Before deciding the most appropriate method/material for repairing/sealing a crack, a determination should be attempted on its cause and

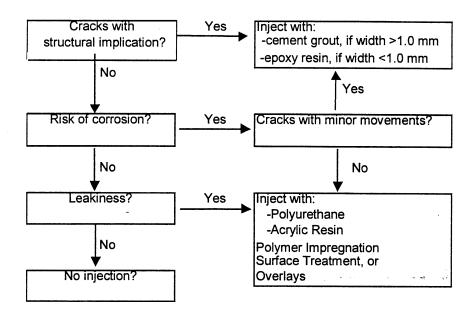
whether it is active or dormant. Whether the crack is active (i.e. propagates/breathes), may be determined by periodic observation.

Basically, a crack resulting from a rare load-application, and which has ceased to propagate, can be repaired (if it is wider than about 0.3 to 0.4 mm) by pressure injection with a suitable epoxy-formulation so that the integrity is restored and any adverse influence on the service life of the structure is eliminated or minimized.

In case of cracks which are the result of time dependent effects, such as shrinkage or settlement, the repair should be delayed as much as possible, compatible with the service efficiency of the structure, so that the effect of further deformation is minimized.

Dormant cracks (dead cracks), in excess of about 0.30 to 0.4 mm width, must be cleaned and then filled and sealed, by epoxy-injection for widths up to about one mm, and by fine cement grout for wider cracks. (Normal cement grout is easily possible for widths beyond 3 mm.) Live cracks (active cracks) must be periodically monitored for propagation and width-increase. Where their width exceeds about 0.3 to 0.4 mm, a 'chase' (V-groove) should be made along the crack, the groove and the cracks cleaned by a dry air-jet, and then filled to part of its depth by a flexible filler material (mastic, thermoplastic, etc.) to prevent ingress of moisture and other deleterious materials. After the crack has become dormant, the filler material can be removed and the crack cleaned and filled with a rigid (epoxy) filler. The chase can then be plastered with cement paste using a non-shrink additive.

ii) A Flow Chart for Decision-Making on Crack-Repair (Raina, 1996)



It must be noted that the corrosion-protection effect by covering a reinforcement bar with epoxy resin is not truly dependable. The resin might electrically isolate a corroding bar only where it is still intact (resin-coating some times gets punctured during handling and bar-tying operations). As a consequence, the corrosion propagation-rate may in fact even be higher at the puncture locations in the bar, giving rise to accelerated local corrosion.

With regard to load-induced cracks, it must be noted that injecting them will not strengthen the structure. The cracks will appear again, unless the loads are reduced.

iii) Materials used for filling (and hence full-depth sealing) the cracks

The material used for crack repair must be such as to penetrate easily into the crack and provide durable adhesion between the crack surfaces. The larger the modulus of elasticity of the material, the greater will be the obtainable adhesion strength. The material should be such as not to allow infiltration of water, and

should resist all physical and chemical attacks. Currently the following fluid-resins are used for crack-injection (together with hardeners):

- epoxy resin (EP)
- polyurethane resin (PUR)
- acrylic resin
- unsaturated polyester resin (UP)

The formulations of commercially available injection resins vary widely in their properties, and care must be exercised in making proper selection. Important properties of any injection resin are its resistance to moisture penetration and alkaline attack from the cement. Where tensile strength is a requirement, the tensile strength of the resin should approach that of the concrete as closely as possible. Therefore, a stiff and highly adhesive resin is desirable. These properties are available in epoxy or unsaturated polyester resins. After hardening of the injection material, the 'stiffness' of the crack will be dependent upon the elasticity of the resin.

The polyurethane or acrylic resin is recommended, where moisture resistance is a requirement. Some epoxy based low-viscous resins will penetrate to the crack-root even when the crack width at the surface is only about 0.2 mm. Comparable results can be obtained from unsaturated polyester and polyurethane resins. Acrylic resins are capable of sealing fine cracks because of their low viscosity. However, in all cases, this requirement can only be obtained with an appropriately long 'reaction-time'. Fast-reactive systems will only close the crack at its surface, which may not be desirable.

Although cement paste is relatively inexpensive, its use is limited to crack widths of approximately 2 mm or more because of its limited viscosity. However, finely ground cements allow injection of cracks with widths down to about 1 mm. Cement glues and mortars are of importance in such applications as injection of

voids, hollows, cavities, and honey-combing, and sealing of ducts, etc. For these applications the use of appropriate additives is recommended to reduce viscosity, shrinkage, and the tendency for settlement. Improvement of workability will be obtained, if the cement-suspension is formed by using high-speed mixers.

3.2.2.1.1 Sealing bridge deck cracks by gravity fill

Relatively wide, dormant cracks in bridge decks are effectively repaired through gravity fill polymers such as high-molecular-weight methacrylate and low viscosity epoxies. To fill these cracks, the deck is usually flooded with the polymer. The polymer is brushed into the cracks until they are filled. When deck surfaces do not have a rough texture, or saw cut grooves, aggregate is added to the polymer to provide adequate skid resistance.

Polymers used to repair cracks by gravity fill have a viscosity of less than 100 cp. High- molecular-weight methacrylates that have viscosity of less than 25 cp have been shown to be effective in repairing cracks with widths of 0.2 to 2.0 mm (0.008 to 0.08 in). A minimum crack width of 0.5 mm (0.02 in) is recommended for gravity fill epoxies that usually have a viscosity of about 100 cp.

3.2.2.1.2 Sealing concrete cracks by epoxy injection

Narrow cracks that are dormant (not moving) may be effectively sealed by epoxy injection. The procedure can be applied to both horizontal and vertical surfaces. Cracks as narrow 0.005 mm (0.02 in) can be sealed and bonded by an injection of epoxy. The procedure has potential to provide structural repair (increase in stiffness and strength) in addition to sealing the crack.

Figure 3.6 demonstrates the technique of epoxy injection. It requires the following:

- drilling holes along the crack,
- cleaning and drying the crack,
- installing ports,
- sealing the crack around and between the ports,
- injecting epoxy under pressure.

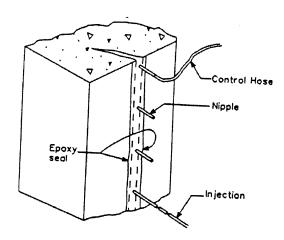


Figure 3.6 Repair of crack by epoxy injection (Babaei, 1996)

The injection progresses from port to port, normally starting at the lowest point, and continuing until epoxy is extruded from the next port. A handgun or pressure pot can be used, but various types of machines are available that assure the proper proportioning, mixing, and temperature of the two part epoxy and the proper injection pressure.

Epoxy pressure injection has gained widespread acceptance as a cost-effective method to bond together and seal cracked structural concrete members. States reporting its use for underwater repairs include North Carolina, Virginia, and Louisiana. The following precautions should be considered:

- organisms growing inside the crack, especially those found underwater, can

reduce the successful welding of cracks,

- corrosion debris can also reduce the effectiveness of pressure injection,
- Injection is labor-intensive. Time and patience are required for the successful injection project,
- as the temperature drops below 50° F, it becomes more difficult to pump the epoxies into the fine cracks,
- experience on the part of the diver in injection and the formulation of the epoxy are very important.

The cause of the crack is an important consideration in selecting a repair. For example, if the crack was caused by settlement that has since stabilized, the crack is dormant and epoxy injection is a viable method of repair. If the movement, or stress, that caused the crack is still present, the epoxy bond is not likely to be successful or the concrete will crack elsewhere to relieve the stress. If the crack is active, not dormant, it may be better to fill the crack with a flexible sealant and allow it to act as a joint (To determine if a crack is dormant, it can be measured at different times, a crack monitoring gage can be attached to the surface, or the surface of the crack can be filled with a grout and checked later).

It is best to schedule crack filling operations in the early spring when the crack is largest due to cold temperatures. This allows cracks to be in compression when the concrete expands; cracks filled during summer or early fall, when they are narrow, would cause the adhesive to be in tension during cooler periods.

3.2.2.1.3 Routing and sealing cracks

Both narrow and wide cracks that are dormant may be repaired by routing and sealing. This method involves enlarging crack along its exposed face and filling and sealing it with a suitable joint sealant (Figure 3.7). This is the simplest and most common technique for crack repair. It can be executed with relatively

untrained labor (compared to epoxy injection).

The routing operation consists of preparing a groove at the surface that is sufficiently large to receive the sealant. The groove is prepared by using a concrete saw or pneumatic tool. A minimum surface width of 6 mm (0.25 in) is desirable. Repairable narrow grooves are difficult. The surface of the routed joint should be cleaned with an air jet and permitted to dry before placing the sealant. The sealant is often an epoxy compound. Urethanes, which remain flexible through large temperature variations, have been used successfully. Rapid setting cementitious mortars may also be used.

3.2.2.1.4 Flexible sealing

Moving cracks can be repaired by flexible sealing (Figure 3.8). The technique consists of the following steps:

- routing the crack,
- cleaning and drying the crack and,
- filling the crack with a suitable field-molded flexible sealant.

As nearly as possible, the sealant reservoir (slot) formed by routing should comply with the requirement for width and shape factor of a joint having equivalent movement.

A bond breaker should be provided at the bottom of the slot to allow the sealant to change shape without a concentration of stress on the bottom (Figure 3.8). The bond breaker may be a polyethylene strip, or other material, which will not bond to the sealant during cure.

If the moving crack is not subjected to traffic impact, it is narrow and esthetics are not important, it may be sealed with a flexible surface seal (Figure 3.9). By

using a bond breaker over the crack, a flexible joint sealant may be troweled over the bond breaker providing an adequate bonding area.

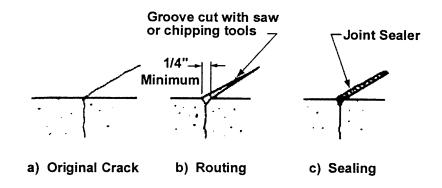


Figure 3.7 Repair of crack by routing (Babaei, 1996)

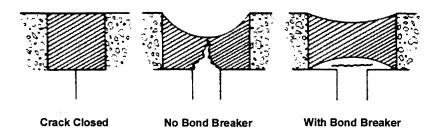


Figure 3.8 Effect of bond breaker in flexible sealing (Babaei, 1996)

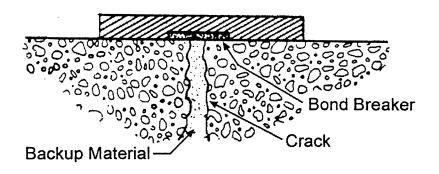


Figure 3.9 Repair of crack using flexible surface seal (Babaei, 1996)

3.2.2.2 Deck patching

Patching methods are used to replace localized areas of deteriorated concrete (spalls and delaminations). For decks, the depth of deterioration may include the top layer of reinforcing steel or both the top and bottom layers of reinforcing steel. If only the top reinforcing mat is corroding, a partial-depth repair would be used. For partial-depth deck repairs, the deteriorated concrete is removed to the depth required to provide a minimum of 0.75 in (19.05 mm) clearance below the top layer of reinforcing steel. Maximum depth of removal for a partial-depth repair should not exceed half the deck thickness. Corrosion of both the top and bottom layers of reinforcing steel requires full-depth repairs. For a full depth repair, the concrete within the delineated area for the entire deck thickness, normally 8 in (203.2 mm) is removed. Once all the unsound concrete has been removed, the cavity should be blasted clean to remove all loose material and provide a dust-free surface. Partial-depth deck patching materials include portland cement concrete, quick-set hydraulic mortar and concrete, and polymer mortar and concrete. Portland cement concretes are used for full-depth deck patches.

Deck patches have a relatively short service life, because they do not address the corrosion of the reinforcing steel, but address only the symptoms; spalling and delaminations. When concrete contaminated with chloride beyond the threshold level is left in place in the area surrounding the patches, the patches themselves often accelerate the rate of deterioration of the surrounding concrete. The patch concrete area acts as a large noncorroding site (cathodic area) adjacent to corroding sites and increases the rate of corrosion.

The most frequently used method of rapidly repairing a bridge deck involves removal of delaminated concrete, sandblasting the concrete surface, and filling the cavity with a high performance concrete. Sometimes cracks are also repaired and a rapid curing protective system is installed. This method has several advantages. The patching, crack repair, and application of the protective system can be done in stages. The patched area can usually be opened to traffic after two to four hours. Concrete removal costs are low, because very little concrete is removed, and the high cost of the patching materials is offset by the low volume of material required.

The disadvantage of the method is that spalling will continue, because all salt contaminated concrete is not removed, and thus corrosion is not stopped. Other disadvantages may be the following:

- All poor quality concrete is not removed.
- There is insufficient time to prepare the surface.
- The rapid setting materials are not properly consolidated or placed.
- The patches crack because of shrinkage.
- The repairs must be opened to traffic before sufficient strengths are developed.
- The repair materials are not similar to or compatible with the materials repaired.

3.2.2.2.1 Patching materials for bridge decks

Asphalt concrete

Transportation agencies have a responsibility to provide a deck-riding surface that is safe. Consequently, when decks spall, the cavity is usually filled with asphalt concrete until a more permanent repair can be made. In warm weather an asphalt concrete mixture that hardens as it cools (hot mix) is used to fill potholes. In cold weather a mixture that cures by evaporation of solvents (cold mix) is used. A proper repair includes removal of dust, debris, and unsound concrete from the cavity, application of a tack coat, and placement and compaction of the patching material. Asphalt patches should be used only as a temporary repair, and replaced with a hydraulic cement concrete patch as soon as practical.

High early strength hydraulic cement concrete

The most common method or permanent spall repair is patching with hydraulic cement concrete.

Polymer concrete

Patching with polymer concrete has been found to be effective when the thickness of the patches is less than 0.8 in (2 cm). The surface to be patched must be sound and dry. The polymer is troweled into place so that edges may be feathered. A prime coat may or may not be required.

Steel plate over concrete

Materials that develop strength slowly are usually easier to place, more compatible with the old concrete, and more economical than rapid curing materials. Patching with materials that do not attain a high early strength can be

done, if the patched area is covered with a steel plate that prevents wheel loads from damaging the concrete. The technique has been used by the New Hampshire Department of Transportation, The District of Columbia Department of Transportation and The Buffalo and Fort Erie Public Bridge Authority.

Disadvantages

Asphalt patches have a short service life, typically less than one year. Most hydraulic cement concrete patching materials shrink more than bridge decks concrete. Less than optimum cure is usually achieved when the hydraulic cement concrete patches are placed with lane closures of less than 56 hours. Curing time increases as temperature decreases. Special cements must be used at temperatures below 55° F (13° C), and patching at temperatures below 40° F (4° C) is not usually done with hydraulic cement concrete. Patches do not retard corrosion, when chloride contaminated concrete is left in place.

3.2.2.2.2 Case studies

i) Griffin Road over I 75, Florida.

This bridge is located in Broward County, Florida, and carries east and west bound S.R. 818 (Griffin Road) traffic over S.R. 93 (I-75). This structure has five spans with a total length of 404 feet, eight lanes with a curb to curb width of 130 feet and was designed for HS20-44 loading. The superstructure is constructed of precast prestressed concrete I beams with a cast-in-place reinforced concrete deck. The bridge was constructed in 1984.

The repair for this case study utilized a pressure grout patching. The deck of this bridge was constructed using prestressed concrete stay-in-place forms, which were placed on the beams with a thin felt pad to provide an even distribution of the load. The deck was then poured over the stay-in-place forms. The felt

pads were not removed, and deteriorated over time. This caused the stay-inplace forms to rotate and the deck to crack. To remedy this situation, the gap left by the deterioration of the felt pads was cleaned thoroughly and the remaining cavity pressure grouted as shown in Figure 3.1. This type of repair was done on several bridges in this area that used the same deck construction technique.

This repair has been in place for 10 years and has exhibited satisfactory performance. A visual inspection of bridge performed on 4/7/98 found the repair in good condition. In fact, the existence of the repair is barely discernible from the original construction. Figures 3.10 and 3.11 show the repaired area under the deck.

Pressure grouting the cavity in this case was an effective way to remedy a poorly constructed bridge feature. District 4 of the Florida Department of Transportation performed this repair and stated that in order for this repair to be effective, it is imperative that the area to be grouted be cleaned thoroughly. Also, all of the voids have to be filled during grouting process.

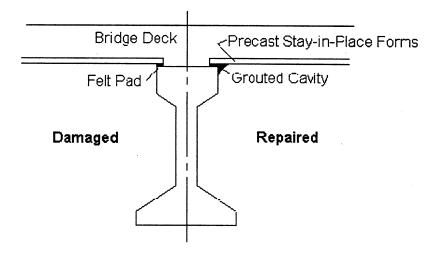


Figure 3.10 Griffin Road repair, Florida

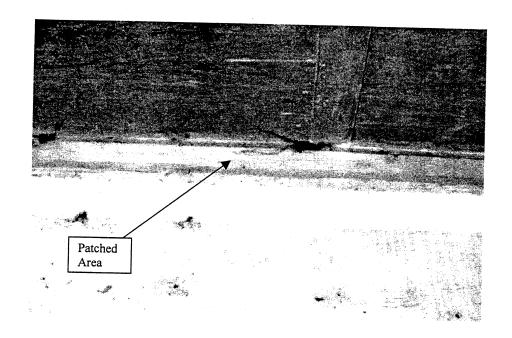


Figure 3.11 Griffin Road repair, Florida

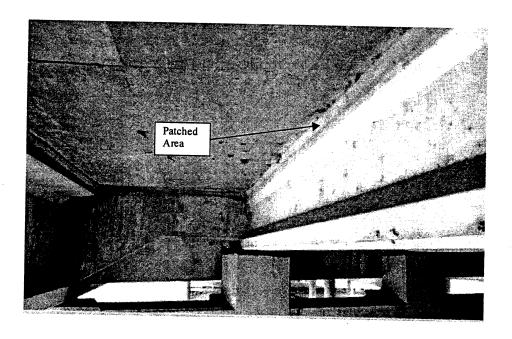


Figure 3.12 Griffin Road repair, Florida

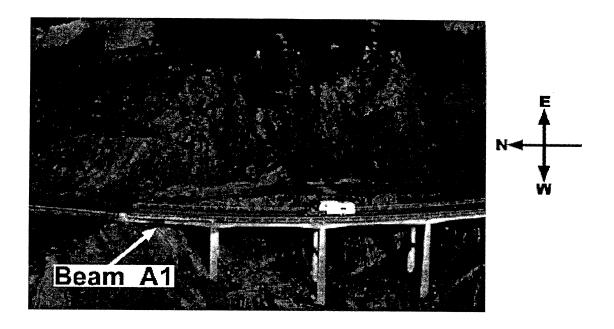


Figure 3.13 Old Point Viaduct (Covino et al, 1999)

ii) Rocky Point Viaduct, Port Oxford, Oregon (Covino et al, 1999)

Details

The Rocky Point Viaduct, located near Port Oxford, Ore., was replaced after only 40 years of service. Located on U.S. Route 101 southeast of Port Oxford, the Rocky Point Viaduct was constructed in 1955 at a coastal site 25 m (80 ft) from the Pacific Ocean.

The bridge had a T-beam structure with 5 spans and a total length of 114 m (374-ft) and a deck width of 10.6 m (34.8 ft). The beam investigated is marked in Figure 3.13 as beam A1. The north end of beam A1 rested on abutment No. 1 while the south end rested on bent No. 1. Beam A1 was at the extreme west edge of the viaduct and thus was exposed to the full impact of weather from the ocean.

Structural history

The inspection and maintenance history of the Rocky Point Viaduct are presented herein:

- 1955: viaduct constructed.
- July 1967: first report of maintenance problems with the bridge; steel rocker assemblies were badly corroded.
- May 1968: steel rockers were badly rusted; concrete beams were cracking and needed chipping and patching.
- January 1969: concrete was spalling along the outside of the west facing beam between bent No. 1 and bent No. 2; concrete was spalling up to the rebar, and rebar was badly rusted; maintenance personnel recommended that loose concrete be removed and patched and steel rocker assemblies sandblasted and painted.
- June 1969: concrete beams were cracking and needed chipping and repair; rebar in the columns was exposed; reinforcing steel rust was of 4 mm (0.16 in.) thick (equivalent to 0.8 mm (0.03 in. loss of metal); steel rocker assemblies were badly rusted.
- September 1969: first repairs were carried out, in which damaged concrete was removed from around the rebar. Steel rebar was sandblasted and some rebars primed with epoxy, whereas other rebars were coated with inorganic zinc coating. Patch concrete was cast around the rebar and the concrete surface sealed with linseed oil. Steel rocker assemblies were sandblasted and painted.
- July 1970: Rebars were exposed and rust streaks were visible on the columns. Small cracks were noticed on the outside of all beams together with transverse cracking on bridge deck. Concrete was spalling from overhang. Patching concrete was suggested for the columns and overhang.
- May 1976: Further concrete cracking and spalling and patch material falling

- out were reported along with substantial section loss from corroded rebar.
- June 1981: An unsatisfactory attempt was made to patch the falling off of the deck bottom due to salt contamination.
- February 1991: The substructure and superstructure were given condition rating 4 (Federal Highway Administration Bridge Inspection Handbook- poor condition, advanced loss section, deterioration, spalling) with an estimated life to be two years.
- 1994: Replacement of Rocky Point Viaduct began.
- 1995: The new Rocky Point Viaduct was opened for traffic.

Approximately 14.4 m (47 ft) of the 20 m (66 ft) beam was delivered in one piece to the U. S. Department of Energy, Albany Research Center (ARC) in Albany, Oregon for detailed study. The results of the study are presented in the following sections (Corina, et al. 1999).

Evaluation

The evaluation of the state of the corrosion of beam A1 was done by considering the beam condition survey, physical properties, half-cell potential survey, chloride distribution, surface air permeability, and a microbial survey.

Results and discussions

The presence of cracks, delaminations, and spalls for the east face of section 1 of the beam A1 are shown in Figure 3.14. These features are indicative of increased access of corrosive environments to the rebar and evidence of past and current corrosion damage. The overall condition of the beam was poor when delivered to ARC. This can be seen from the large delaminated and spalled areas of the beam and also the presence of cracks.

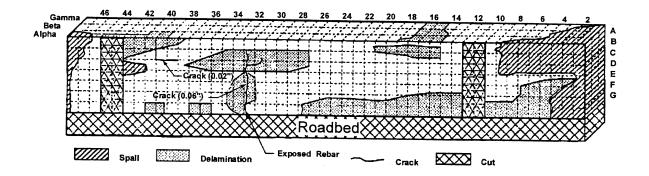


Figure 3.14 Schematic of section from beam A1, east face of the Rocky

Point Viaduct

Figure 3.15 shows the beam cross section and the patch concrete is seen at the top of the photo surrounding the large square rebar. Patch concrete was applied during 1969, repair operation after removing the original concrete from around the rebar. The remaining concrete in the beam is original concrete installed in the 1955 construction and differentiated from the patch concrete for the purposes of this photograph by wetting the concrete surface.

Conclusions

Chloride contamination of the Rocky Point Viaduct appears to be responsible for its failure. The use of coatings on the rebar after the repair in 1969 did not decrease the corrosion susceptibility of the rebar. The use of poor patch concrete in 1969 accelerated the failure of the Rocky Point Viaduct. The patch concrete was more porous and thus more susceptible to penetration of chlorides. More chlorides entered the patch concrete in 25 years than entered the original concrete in 40 years.



Figure 3.15 Cross-section of beam A1 showing the location of the reinforcing steel, the patch and the original concrete

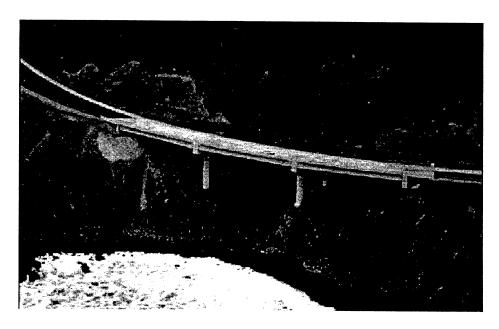


Figure 3.16 New Rocky Point Viaduct

iii) M6 Widening: Thelwall Viaduct (Mallet, 1994)

Having carried up to 134,000 vehicles a day over the Manchester Ship Canal, the 1.3 km viaduct would be retained for northbound traffic, when a new viaduct has been completed for southbound traffic. Corrosion of the deck slab reinforcement resulted in spalling and potholes in the carriageway. Cores revealed chloride concentrations up to 2.6% by weight of cement, which required the entire deck to be replaced.

3.2.2.3 Deck overlays (Weyers, 1993)

Description

Overlays are used to restore the deck-riding surface to as-built quality and increase effective cover over the reinforcing steel. Overlays include latex-modified concrete (LMC), low-slump dense concrete (LSDC), and hot-mix asphalt concrete with a preformed membrane (HMAM). The overlay has some influence on the service life of the repair, but the amount and degree of the chloride-contaminated concrete left in place remains the most important factor.

Advantage

The cost analysis shows that the latex-modified concrete overlay is more costly than the other overlay materials. However, this overlay increases the deck performance without reducing the load rating, since the thickness required is less than other overlay materials. Moreover, when the delay, inconvenience, safety, and costs of lane closures due to deck deterioration are factored in, overlays are not only warranted, but also mandated.

Disadvantages

LMC, LSDC, and HMAM overlays may increase the dead load and thus decrease the live load capacity of the bridge. The influence of the overlay on the live load capacity of the bridge must be evaluated before one of these overlay systems is specified. LMC, LSDC, and HMAM overlays should not be placed on decks where the existing concrete may be susceptible to alkali aggregate reactions (silica or carbonate) unless low-alkali cement is used or other preventive measures have been taken.

3.2.2.3.1 Microsilica concrete overlay

Used as a deck repair method, microsilica concrete (MSC) overlays restore the deteriorated riding surface to its original service condition. MSC is a low permeability concrete typically containing 7 to 12% microsilica by weight of the cement. Because of the extreme finesse of microsilica, a high range water reducer must be used to reduce the water and improve workability. The most common specified overlay thickness is 2 in (5 cm).

The overlay has some influence on the service life of the repair, but service life is largely controlled by the amount and contamination level of the chloride-contaminated concrete left in place.

Disadvantages

Overlays may increase the dead load and thus decrease the live load capacity of a bridge. Thus, before an MSC overlay is specified, the reduced live load capacity of the bridge must be compared with present and future needs. Since the primary factor that influences the service life of a repair MSC overlay is the area of sound but actively corroding chloride contaminated concrete left in place,

environmental chloride exposure conditions have little influence on the service life of the repair MSC overlay. The service life of a repair MSC overlay is estimated to be 22 to 26 years.

3.2.2.3.2 Rapid concrete treatment methods

Rapid treatment methods are used when lane closure for extended periods is not practical. Included in these methods are asphalt overlays on membranes, polymer overlays, and sealers. These methods restrict the infiltration of chloride ions into concrete that is not already contaminated with chloride. This consists of the removal of all deteriorated, delaminated, and chloride contaminated concrete, patching and applying a rapid protection method.

3.2.2.3.3 Polymer overlays (Weyers, 1993)

Polymer binders include acrylic, methacrylate, high-molecular-weight methacrylate, epoxy, epoxy-urethane, polyester styrene, polyurethane, and sulfur. Aggregates are usually silica sand or basalt. Multiple-layer, premixed, and slurry are the three basic types of polymer overlays. Polymer overlays usually maintain adequate skid resistance as long as aggregates remain bonded to the surface. Multiple-layer overlays are constructed by applying one or more layers of resin and aggregate to the deck surface. The average thickness of this overlay is 0.25 in (0.64 cm). Epoxy is the most widely used resin. Premixed overlays are usually 0.5 to 1.0 in thick. They are used to correct minor surface irregularities and make small improvements in surface drainage. styrene is the most commonly used binder and special alkali resistant polyester or a high-molecular-weight methacrylate is the most commonly used primer. Slurry overlays are usually about 0.38 in thick. Epoxy and methacrylate are the most frequently used binders.

Advantages

Polymer concrete overlays are effective in reducing the infiltration of chloride ions and water into the deck. Polymer overlays also increase the skid resistance of the surface. The increase in dead load is small compared to bituminous and hydraulic cement concrete overlays, because polymer overlays are thinner. With a 1.75 in thick average overlay, chloride ion corrosion should not become apparent for over 75 years, for a moderate chloride application rate.

Disadvantages

Polymer concrete overlays do not substantially improve the ride quality, or drainage, or increase the section modulus, because they are thin and follow the contours of the deck itself.

3.2.2.3.4 High-early strength hydraulic cement concrete overlays (Portland cement) (Weyers, 1993)

Hydraulic cement concrete overlays are placed on decks to reduce the infiltration of water and chloride ion and improve the ride quality and skid resistance. Overlays must also be placed to strengthen or improve the drainage of the deck. The overlays are usually placed with internal and surface vibration and struck off with a mechanical screed. The overlays usually have a minimum thickness of 1.5 in (3.8 cm) for concretes modified with 15 % latex by weight of cement and 2 in. (5 cm) for most other concretes, such as LSDC.

Some concretes, such as those containing 7 % to 10 % silica fume, or special blended cements like Pyrament have permeabilities similar to latex-modified concrete and should perform adequately with a thickness of 1.5 in (3.8 cm). High early strength hydraulic cement concrete mortars about 1 in (2.5 cm) thick have

been used as overlays, but these overlays tend to crack and do not provide much protection.

Advantages

These overlays reduce the infiltration of chloride ions and water. They also improve the ride quality and skid resistance of the deck. They may be used to strengthen or improve the drainage of the deck. Portland Cement concrete overlays are usually at least 1.25 in thick with a maximum thickness up to 2.0 in.

Disadvantages

Some high-early-strength Portland cement concrete mortars are as thin as 1.0 in, but do not provide much protection unless latex or silica fume is added to the mixture. Cracking is a common problem with this mortar. Very dense concretes, such as polymer concretes and polymer-modified concretes, or film-forming concretes should not be used in the patching before installing the deck overlay, because they interfere with the overlay bonding.

3.2.2.3.5 Asphalt concrete overlays (Weyers, 1993)

Transportation agencies have a responsibility to provide a deck riding surface that is safe. Consequently, when deck spalls, the cavity is usually filled with asphalt concrete until a more permanent repair can be made. In warm weather, an asphalt concrete, mixture that hardens as it cools (hot mix) is used to fill potholes. In cold weather, a mixture that causes the evaporation of solvents (cold mix) is used. A proper repair includes removal of dust, debris, and unsound concrete from the cavity, application of a tack coat, and placement and compaction of the patching material (U.S. Department of Transportation, 1980). Asphalt patches should be used only as a temporary repair, and they should be

replaced with a hydraulic cement concrete patch as soon as practical.

Advantages

These overlays provide a smooth riding and wearing surface. To improve skid resistance, a very thin surface treatment may be applied.

Disadvantages

The service life is very short and, therefore, asphalt concrete overlays should only be used for temporary rehabilitation. These overlays should be replaced with more permanent patches as soon as practical.

3.2.2.3.6 Evaluation of overlay performance

Methacrylate slurry overlays have the lowest permeability with age. Multiple-layer epoxy-urethane overlays, multiple-layer epoxy overlays, and premixed polyester overlays all have very small permeability, which increases with age. These overlays are constructed with flexible binders. Overlays constructed with brittle binders, like those used in multiple-layer polyester overlays and multiple-layer methacrylate overlays typically have much higher permeability with age. This is due to the cracking of the brittle binders. Table 3.2 shows the service life in years for treatments using different types of overlay.

Treatment	Avg.	Low	High
Asphalt Concrete Overlay on Membrane	11.8	4.5	20.0
Portland Cement Concrete Overlay	15.5	10.0	22.5
Polymer Overlay	12.7	6.0	25.0
Multiple-Layer Epoxy	15.0	10.0	25.0
Multiple-Layer Epoxy-Urethane		15.0	25.0
Premixed Polyester			25.0
Methacrylate Slurry	5.0	3.0	18.0
Multiple-Layer Polyester	10.0	10.0	10.0
Multiple-Layer Methacrylate	15.0	15.0	15.0

Table 3.2 Service life (years)

3.2.2.4 Deck Replacement (Brinckerhoff, 1995)

The prerequisite for deck replacement is superstructure and substructure systems that can be expected to be in service as long as the new deck. Deck replacement can cure a number of problems. The available replacement systems vary in weight between 50 and 120 pounds per square foot (including basic supporting elements of steel grids). The lighter weight systems, when used to replace heavier deck types, can improve dead-load stresses, allow larger than original live loads, and even introduce the possibility of roadway widening without complete bridge replacement.

The most common replacement options include

- Cast-in-place concrete.
- Precast concrete.
- Steel grid (open or filled).
- Orthotropic.

i) Cast-in-place concrete

Cast-in-place concrete appears to be the most popular redecking option. Lightweight concrete was used, for example, at the Braga Bridge in Massachusetts in 1988, and at the American Legion Bridge on the Capital Beltway between Maryland and Virginia in 1984. The Smith Street Bridge, over the Genesee River in Rochester, New York, was redecked in the late 1970s with a 9-inch normal weight cast-in-place slab. The Bear Mountain Bridge, over the Hudson River in New York, was redecked with cast-in-place concrete in 1970.

Most bridge owner/operators consider cast-in-place concrete decks their norm for new construction, and utilize this type of construction for deck replacement as well whenever possible. The popularity of cast-in- place concrete results from its comparative low cost, its well known construction methods, and its readily available materials. The longevity of concrete decks has improved through the years with the introduction of air entrainment, water-reducing additives, corrosion inhibiting agents, and silica fume additives, which have all helped to prevent water-borne salts from reaching the reinforcement. Epoxy-coated reinforcement also has helped to prevent corrosion after the salts have reached the reinforcement level.

Disadvantages of concrete construction include its relatively high unit weight, even with lightweight concrete, and the curing time required before traffic can be allowed on the finished roadway. The weight can become a drawback if the structural elements are not in good condition, if live loads have increased since the original structure was designed, or if the original deck was constructed of one of the lighter deck types. If traffic constraints require that the riding surface be placed back in service quickly, cast-in-place construction is at a disadvantage in comparison with other available redecking techniques.

ii) Precast concrete

Modular precast concrete involves placing precast panels and attaching them to the superstructure through blackouts filled with cast-in-place concrete. Shear studs or roughened concrete surfaces then provide the necessary horizontal shear connection. This type of construction has gained popularity in recent years. The New York State Thruway Authority has redecked a number of bridges with precast, prestressed deck panels, including an interchange bridge in Amsterdam New York, in 1974, and the Krum Kill Road Bridge in 1977. The Pennsylvania Turnpike Authority has also utilized precast panels. The deck of the Woodrow Wilson Memorial Bridge, crossing the Potomac River, partially in Maryland, Virginia, and Washington, DC, was widened and replaced with prestressed lightweight concrete in 1983.

The advantages of the precast construction include the excellent quality control, resulting in more durable, nearly crack-free, consistent concrete, flexibility of fabrication allowing the panels to be built in any weather, and speed of installation providing an almost immediate riding surface for traffic.

The disadvantages of this type of construction include its weight, which is about the same as the cast- in-place concrete. Its cost is slightly higher than the cast-in-place concrete. Besides, it has the added complexity of the details required for horizontal shear connection and the many cold joints in the final deck surface.

iii) Open grids

Open grids have found a number of applications across the country. This grid system is popular when dead loads and wind resistance must be reduced to an absolute minimum, as in movable structures. Normally open grids are not used in redecking applications, unless the existing deck was also open grid.

The advantages of the open grid are its minimal dead load, its low wind resistance, and its free-draining surface. It also offers an immediate riding surface as soon as the modules are placed. These advantages are offset by low skid resistance, poor riding quality because of its tendency to promote fish-tailing and tracking, and the lack of protection, allowing road chemicals to reach supporting elements easily.

iv) Filled grids

This deck is popular as a replacement surface because of its good riding surface combined with its relatively low weight. The states of Connecticut, Missouri, and West Virginia use this type of redecking commonly, generally when dead loads must be limited. West Virginia uses half-filled grid to replace its rural wood deck surfaces. Full-depth grids have been used on the Burlington Bristol Bridge in 1988, portions of the FDR Drive, Queensboro Bridge, Manhattan Bridge, and approaches to the George Washington Bridge, in New York, and the Crown Point Bridge over Lake Champlain. They were also used in the redecking of the Thousand Islands Bridge over the St. Lawrence River, and the Mid-Hudson Bridge over the Hudson River.

The filled grids are generally placed without concrete fill, attached to the supporting elements, and then filled, either half-depth or full-depth, with concrete in place. The grid functions as a reinforcement cage, preassembled and ready for placement of concrete. It also offers an immediate riding surface, in the event the roadway must be opened to traffic before concrete can be placed. Concrete fill is then either placed to the top of the grid or overfilled above the grid to form an integral wearing surface. When concrete is not overfilled, a separate wearing surface, either dense concrete or latex-modified concrete, can be placed after the original concrete fill cures. The integral or separate wearing surface is necessary to avoid cupping of the concrete within the grid squares, along with

the loss of skid resistance and rideability associated with riding directly on a steel grid surface. Also, grid growth has been reported where the deck has not been adequately protected and the steel elements have rusted and expanded.

v) Orthotropic systems

A properly designed orthotropic deck can provide an excellent riding surface, with square foot weights approaching those of the open grid systems. Orthotropic systems have been used to redeck a number of bridge decks in recent years, and their popularity as an initial decking system is growing. These decks were used as replacement decks at the Lion's Gate Bridge in Vancouver in 1975, the George Washington Bridge in New York in 1978, the Golden Gate Bridge in San Francisco in 1985, the Throgs Neck Bridge in New York in 1986, the Ben Franklin Bridge in Philadelphia in 1987, and the Champlain Bridge in Montreal, under construction in 1990. In all the above applications, the orthotropic decks were replacing original concrete decks.

The orthotropic deck, when used as a deck replacement system, usually replaces not only the deck, but its immediate supporting stringers, which allows the weight to be minimized. The stiffening ribs attached to the deck plate, span between floorbeams, or other supporting elements. Rib spans currently in use vary from 15 to 40 feet. When the orthotropic deck is placed above existing supporting elements, the impacts to profiles and vertical clearances must be addressed during the redecking design process. An advantage of an orthotropic scheme is that the modular construction can be accomplished at night to minimize traffic impacts. When this is done, the section is paved after all erection is complete.

Riding surfaces have caused some problems with orthotropic systems. More recent installations have used epoxy asphalts, rather than the earlier bituminous

asphalts. The epoxy asphalts have generally performed better than the bituminous, which were frequently subject to bond failures in the form of shoving and rutting of the pavement surface. Epoxy asphalts properly mixed and placed, combined with adequate deck plate thicknesses, can provide long-term, comfortable riding surfaces.

vi) Total Bridge Replacement

Bridge replacement must be considered an option when reviewing the condition of the existing structure. There are times when deck rehabilitation or replacement cannot resolve the problems associated with an existing structure:

- Primary superstructure and substructure elements are inadequate for even the lightest of replacement options.
- Major widening or reconfiguration of the structure is called for.
- Costs of rehabilitating existing primary superstructure and substructure elements approach the cost of total replacement.
- Current design standards place the structural configuration in the category of functionally obsolete.

If any of these conditions exist, they should be recognized during the deck rehabilitation and replacement design process, and alternatives should be explored. Total bridge replacement may include a complete new structure, either in the same location or at a new site, a twin structure to relieve load from the existing bridge, and/or rehabilitation or replacement of the existing structure after its twin is opened to traffic.

3.2.2.5 Deck: Sealers

Sealers are used for rapid deck protection, because they can be applied easily and quickly. They are also well suited for off-peak traffic period application

because, in general no problems arise when traffic is permitted on a deck, in which one span or lane is treated and one is not.

A sealer is a solvent or water based liquid applied to a deck surface. Only penetrating sealers, silanes and siloxanes (or combinations), are recommended for deck surfaces. Other sealer types have an inadequate depth of penetration and quickly wear when exposed to traffic abrasion.

Silane and siloxanes are normally applied to a prepared concrete surface at a rate of about 150 ft²/gal (3.7 m²/L). When silanes and siloxanes penetrate the prepared concrete surface, they react with the pore walls of the hardened, moist cement paste to create a nonwettable surface. This seal prevents liquid water from entering the concrete, but allows water vapor to enter and leave the concrete (SHRP 1993).

Advantages

Sealers reduce the infiltration of chloride ions and water. A number of methods of application are available, including application by spray, roller, brush, or squeegee. Some sealers penetrate the surface pores and capillaries of the concrete very well and leave a thin hydrophobic film 0 to 10 mils thick.

Disadvantages

Skid resistance is low unless placed on heavily textured surfaces. The deck surface must be shotblasted or sandblasted to remove all materials that may interfere with the bonding or curing of the sealer and to open the capillaries and pores so the sealer may be able to penetrate adequately. No deck moisture can be present prior to application of the sealer. This method should not be used until the concrete is cured sufficiently, usually at least 28 days, because the

moisture in the patch or gas produced by chemical reactions may interfere with the penetration or adhesion of the sealer.

3.2.2.6 Case studies

i) Repair of Fives Road Bridge (Borderie 1981)

The three span continuous post-tensioned concrete structure was built in 1956. A full investigation was initiated, when the cracking was observed in an inspection in 1966. This included leveling, acoustic and gamma-radiographic examination and making some openings for direct inspection of the tendons. More than half of the ducts inspected was incorrectly grouted, wires were corroded and some fractured. Short-term repair consisted of injecting the cracks, reinforcing the webs and splinting the joints. The log-term repair was completed in 1977, which involved the provision of additional longitudinal post-tensioning.

ii) Pancevo Bridge, Yugoslavia (Djurdjevic and Pakvor, 1989)

The 82.5 m continuous three span prestressed box-girder bridge was constructed in 1964, one of sixteen interconnected access structures forming part of the Danube crossing in Belgrade. Cracks in the vicinity of the supports and at midspan were injected with epoxy resin in 1976. Corrosion of the reinforcement and spalling in the bottom slab of the box was such that the slab required replacement and the prestressing tendons had to be replaced in 1984.

A further detailed inspection and analysis of the structural complex revealed cracking as shown in Figure 3.17(a). The rehabilitation included the carefully controlled pressure injection of cracks wider than 1 mm with low viscosity epoxy resin and supplementary prestressing, which ranged from 60 to 90% of the originally designed prestressing force according to the capacity of the section. A

typical arrangement of the supplementary prestressing is shown in Figure 3.12(b). Additional anchorage and deflectors, to provide for further 25% of additional tendons were provided to meet any future requirements. Traffic was completely stopped, while the crack injection work was in progress but, otherwise, traffic was maintained on one side of the bridge. Two of the structures needed temporary supports.

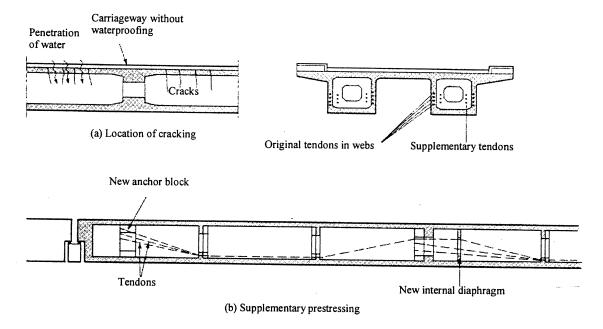


Figure 3.17 Pancevo Bridge

iii) Hanshin expressway, Japan (Mallet, 1994)

The structural behavior of deteriorated concrete is described with the preferred repair method using synthetic resin. Natural and accelerated exposure trials were conducted on concrete specimens treated with the following materials:

- epoxy resin coating,
- polymer cement paste lining,
- silane monomer impregnation, and
- silane oligomer impregnation.

Field repairs to the affected piers showed that both corrosion of reinforcement and expansion of concrete due to alkali-silica reaction were effectively controlled by filling cracks by pressure injection, followed by either an epoxy coating or silane impregnation.

iv) Dolphin Footbridge (Gledhill and Jones, 1981)

This four span prestressed concrete U-section footbridge over the M62 motorway was designed to carry some public utility services when it was hit by any item of plant on a low-loader, two 150 mm diameter steel sleeves provided the major restraint preventing its collapse onto the carriageway. The spans were supported alternately by dowelled and rubber bearings. One end of the impact was moved across the pier, which had tilted and failed at its root while the main span beam soffit and the web of a side span beam were cracked. The beams were repaired by epoxy grouting the cracks and prestressing the soffit and sidewalls with Macalloy bars. The pier was relieved of load, plumbed and its base secured by a prestressed and reinforced concrete collar.

v) A25 Woodbridge (Cogswell and Herbert, 1991)

This 21 m span reinforced concrete arch bridge was designed and constructed in 1912. It was treated extensively with sprayed concrete in the early 1960's. In 1988 a thorough investigation included a principal inspection, extensive testing and assessment of load capacity. Numerous areas of spalled concrete were found throughout the deck, due mainly to inadequate cover. There was no evidence of chloride attack and the strengthening work included casting a new reinforced concrete slab on top of the existing deck as shown in Figure 3.18.

vi) Passaic River Bridge of the New Jersey Turnpike (Mallet, 1994)

Passaic River Bridge was constructed in 1951. About 10 percent of the slabs had to be repaired after 8 years of the construction, and an average of about 1 percent more have been repaired every year by different methods. The most recent repair method used was the latex-modified concrete overlay. This method has performed very well since 1991.

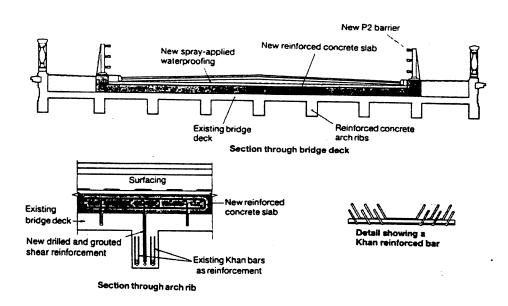


Figure 3.18 Woodbridge, Guildford

vii) Rip Van Winkle Bridge (Moreau, 1993)

The Rip Van Winkle Bridge is a 5,000-foot combination of thru- and deck-truss toll structure over the Hudson River, thirty miles south of Albany, New York. The New York State Bridge Authority built the bridge in the early thirties. The bridge was designed to carry concrete trucks, which resulted in a very solid, and potentially long-lasting structure. However, in the mid-1980's, the bridge was in danger of becoming unusable. A latex-modified concrete overlay was used to repair the deck of the bridge. Even with the overlay, the condition of the deck continued to deteriorate. In 1989 the New York State Bridge Authority was

asked to study a possible deck widening and replacement project. The result of that wide ranging study was a recommendation to replace the deck with a new cast-in-place concrete deck, and widen the deck so that the new roadway would extend the full width available in the thru-truss area.

viii) Overboda Bridge over Dalalven, Sweden (Paulsson and Sifwerbrand, 1998)

The Overboda Bridge is a part of national road 76 and has average daily traffic (ADT) of 7,600 vehicles. Ten percent of the ADT is heavy vehicles, mainly loaded timber trucks, at high speed. The bridge is 12 m wide and has two lanes. Completed in 1942, it is a zero hinged concrete arched bridge. The concrete slab is reinforced in two directions and had a thickness of 170 to 230 mm in 1942, including an original bonded concrete overlay. The bridge has four main spans and two approach spans.

In 1984, after 42 years in service, the end beams were suffering from scaling due to freeze-thaw action and weathering. The old concrete did not have an adequate air-void system. The increased use of deicing agents accelerated the deterioration. The top of the bonded concrete overlay placed in 1942 was delaminated, but the concrete at the reinforcement was sound. The concrete bridge deck was subjected to water jetting to remove damaged concrete and a bonded overlay of steel fiber reinforced concrete (SFRC) replaced the removed concrete and the asphalt wear in course. Observation from the water jetting indicated that the concrete had good strength. The water-jetted surface was carefully cleaned both after the water jetting and prior to overlay placement.

In 1995, the overall impression from Overboda Bridge was that the overlay had performed very well. The compressive strength has not degenerated. No contamination was found at the interface in any cores. Bond testing indicated a good bond between the overlay and the existing concrete. The failure took

place mainly in the cement paste below the large aggregates and reinforcing steel, and in the aggregate in the old concrete. Moreover, the freeze-thaw resistance of the overlay was tested, and the tests showed that a steel fiber reinforced concrete (SFRC) had very good freeze-thaw resistance after 56 and 112 freeze-thaw cycles. The repaired deck is shown in Figures 3.19 and 3.20.

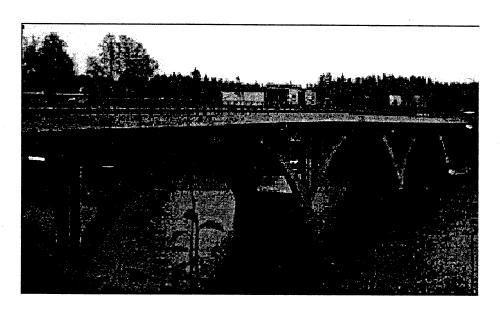


Figure 3.19 Overboda Bridge prior to the repair in 1986

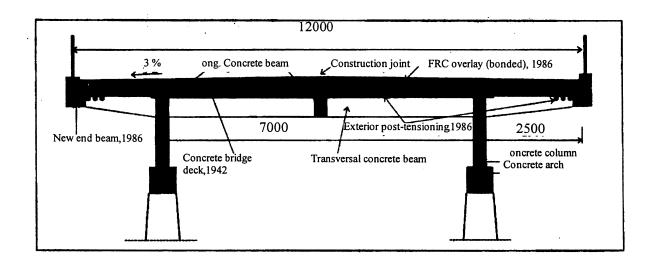


Figure 3.20 Cross-section of Overboda Bridge after repair in 1986

ix) 60-year-old Canada-U.S. crossing (Pickle, 1999)

Completed in the 1930's, the landmark Thousand Island Bridge system spans the St. Lawrence River from Ivy Lea in Ontario to Collins Landing in New York (Figure 3.21). The region has been a major summer vacation area of northern New York State and eastern Ontario. The bridge system was built not as a tourist attraction, but as a crossing for traffic over the St. Lawrence River. It provides a direct link between Highway 401, a major expressway from Windsor, Ontario to Quebec City, Quebec, and Interstate I-81 in New York State. At the time of its opening, the annual number of vehicle crossings was approximately 150,000. By 1995, that number exceeded 2 million. A daily traffic of 1000 trucks and 4000 passenger cars cross the Thousand Island Bridge. The peak traffic demand during the summer period can be as great as three times that of the winter.

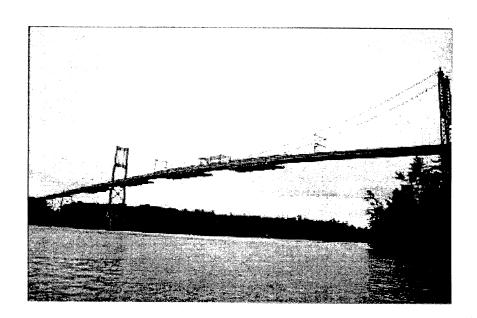


Figure 3.21 Suspension spans with work platforms and portal frame gantry (Pickle, 1999)

The Canadian crossing consists of four distinctive bridge structures. Spanning the North Channel from the Canadian mainland to Georgina Island is the

signature structure of the crossing, a 412 m (1350 ft) long suspension span with a vertical shipping clearance of 36 m (120 ft) above the St. Lawrence River. From Georgina Island to Constance Island is a 106 m (348 ft) long rib arch structure, and from Constance Island to the south abutment on Hill Island is a 183 m (600 ft) long warren truss structure. Viaduct spans form the north and south approaches as well as the span for the islands between the suspension, arch, and warren-truss structures. The total length from abutment to abutment is 1014 m (3330 ft).

Over the life of the structures, numerous improvements have been made to accommodate the increases in traffic volumes and truckloads. After the 1940 collapse of the Tacoma Narrows suspension bridge in Washington State, a system of cable stays and torsion framing was installed on the suspension spans. Stiffening plates have been added to a number of the original structural members to meet increased capacity demands. There have been many localized repair projects to replace cracked clip angles and loose rivets. The Thousand Islands Bridge Authority carries out an extensive annual maintenance program that includes cleaning, painting, and bearing lubrication.

Inspection

In 1994, a detailed structural inspection of the Canadian Crossing was completed for the St. Lawrence Seaway Authority, the bridge owners, and the Thousand Islands Bridge Authority. The field inspectors found the substructure, abutments, and piers of all of the structures to be in good condition.

The deck systems of the suspension spans and the Warren-truss spans were found to be in poor condition. On the warren-truss spans, the deck was cracked and separated from the stringers. A number of floor beams had become warped along the axis of the web. The corrosion of the main bars of the steel-grid deck

panels led to the longitudinal expansion of the concrete deck. There was also severe spalling of the concrete deck surface as well as cupping of the concrete in the deck panel cells. The deck system of the suspension spans was found to be in a similar condition as that of the Warren-truss spans.

This inspection made it apparent that the decks of the suspension and Warrentruss spans were nearing the end of their service life and the total replacement would be necessary by the year 2000. The recommended deck replacement would address the structural concerns as well as provide for the continued widening of the roadway.

The deck system of the viaduct and arch spans was in good condition with some deterioration of the structural steel. The concrete in the deck exhibited areas of random spalling and delamination. The recommendation was made to replace the deck on the viaduct and arch spans concurrently with the replacement of the deck on the warren-truss and suspension spans.

Canadian crossing rehabilitation

The original deck on the Thousand Islands Bridge was a system of steel grid panels with the grid cells filled with concrete. The main bars of the grid panels were aligned to transfer loads to the first level of support of the substructure. On the suspension spans and Warren-truss spans, longitudinal stringers provided the support and on the viaduct and arch spans the deck panels were supported on lateral crossbeams. The cells of the grid panels were filled with concrete to minimize ice accretion in the winter.

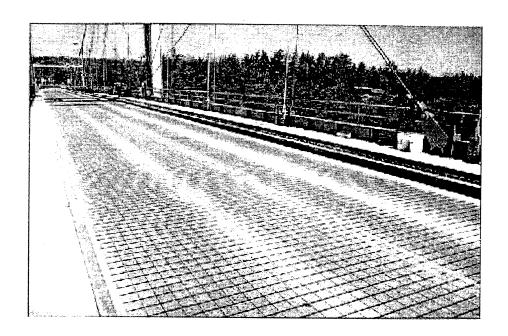


Figure 3.22 Original deck surface condition on the suspension spans (Pickle, 1999)

In 1995, a detailed design for the necessary rehabilitation work was completed. Designers considered a number of different deck system types. In the final analysis, the existing system of concrete-filled steel-grid deck panels, as originally designed in 1938, proved that it was still the most appropriate system for use in the structures.

On the suspension and Warren-truss spans, the original planned thickness of the deck was 4.25 in. (108 mm). For the rehabilitation project, the new steel grid deck panels were specified to have 108 mm high main bars with cross bars 16 mm (0.63 in.) in height, positioned through the main bars to create cells 102 mm (4 in.) wide and 150 mm (6 in.) long in the top of the panel. A secondary bar was positioned in the bottom of the panel for reinforcement of the concrete.

For greater resistance, the designers specified permeability. Microsilica concrete was specified with air entrainment 1 to 3 percent in the steel grid deck panels of the suspension and Warren-truss spans. Tests indicated that the presence of

silica fume in concrete can reduce chloride ion permeability by as much as 80% over a normal concrete mix with a type 10 Portland cement (Type 10 normal portland cement as provided in Canadian Standards Association (CSA) Standard A5, generally equivalent to ASTM Type 1 cement).

The low-permeability, high-density characteristics of the microsilica concrete provided two benefits for the new steel grid deck panels. Low permeability meant resistance to capillary movement of moisture through the concrete, thereby reducing the potential for corrosion of the steel bars of the panels and the supporting stringers and floor beams. The higher density of the concrete provided for improved wear resistance of the surface to minimize the potential of cupping of the concrete in the grid cells. The microsilica concrete was struck off and finished flush with the top of the bars of the panel.

On the viaduct and arch spans, the original deck was an 8 in. (200 mm) deep reinforced concrete slab, supported on crossbeams typically spaced at 6'-5" (1.96 m) on center. For rehabilitation of these spans, a 132 mm (5.2 in) deep steel grid panel was specified. The main bars were to be oriented parallel to the centerline of the structures with 16 mm (0.63 in) cross bars installed to create cells 150 mm (6 in) wide and 100 mm (4 in) long, with a secondary bar in the bottom of the panel for reinforcement of the concrete.

A 50 mm (2 in) concrete overfill of the deck panels was proposed to provide a total depth of 182 mm (7.2 in) for the new deck. This overfill would provide cover on the steel bars of the grid panels and a wearing surface for traffic. An AASHTO Class C air-entrained concrete was selected for the viaduct and arch spans. The specified air entrainment was 4 to 6 percent.

The rehabilitation of the deck of the Canadian Crossing would serve two purposes. The primary one was to address the deteriorated conditions of the

suspension and Warren-truss spans. In addition, removal of the old deck would allow for the widening of the roadway portion of the deck to a safer width for traffic of 7.3 m (24 ft). To create this widened deck, the new panels for the east and west side of the deck were manufactured to a width of 4.0 m (13 ft).

Conclusion

When designed and constructed in the late 1930's, the Thousand Island Bridges represented the state-of-the-art in bridge design. Over the life of these structures, a number of improvements have been made to upgrade the spans to meet user demands. In 1994, deck sections on the Canadian Crossing, which had reached the end of their life cycle, were replaced. In 1997, new construction materials and methods were used for rehabilitating the structure using 1930's style bridge deck construction.

3.2.2.7 Detailed investigation of existing decks (Samples, et al 2000)

3.2.2.7.1 Field investigation

The detailed survey of six bridge decks in Indiana included cores taken to extract reinforcing bars and examine concrete, cores taken for chloride analysis and half-cell potentials on bridges with uncoated reinforcement. Corrosion of the epoxy-coated reinforcement was discovered during the detailed bridge inspection in areas of cracking and shallow cover. The following items were included in the inspections:

- Three 102 mm (4in.) cores taken to extract and examine reinforcing bars and concrete in good and bad areas;
- Ten 25 or 38 mm (1 or 1.5 in.) cores taken to be ground for chloride analysis;
- Half-cell potentials on bridge decks with uncoated reinforcement;
- Cover readings; and

Delaminations.

Specifically, powder samples were analyzed to determine the level of chlorides present and cores examined to assess concrete quality and reinforcing bar deterioration. A mapping of the half-cell potentials, delaminations, and cracking patterns was performed to identify deteriorated portions of the deck. The three 102 mm (4 in.) cores taken to examine the reinforcing bar quality were taken in three different areas of the bridge deck. When possible, one core was taken form each representative area: sound concrete, delaminated concrete, and cracked concrete. The smaller cores taken for chloride analysis were spaced evenly over the surface of the bridge deck. Table 3.3 shows the specifics of the bridge structures chosen for the detailed survey.

3.2.2.7.2 Results of detailed bridge deck survey

One lane and shoulder of each of the six bridge decks listed in Table 3.3 were investigated in the detailed survey. The bridges chosen for the detailed survey were selected based on the level of distress observed in the initial visual survey. For each of the five main corrosion protection methods, the bridge deck with the highest level of distress was chosen for the detailed survey. No signs of distress were found on two of the chosen bridge decks. None of the bridges surveyed with epoxy-coated reinforcement with Class C concrete and uncoated bottom mat of reinforcement showed any signs of distress.

The bridge structure chosen for this category was selected based on the close proximity to other structures chosen for the detailed survey. Also, originally it was thought that one bridge structure with epoxy-coated reinforcement and Class C concrete had 38 mm (1.5 in.) of latex-modified concrete overlay. This bridge showed no signs of distress but was chosen for the detailed survey as a sixth method of corrosion protection. During the detailed survey it was

discovered through the cores removed from the bridge deck that no overlay was present. Therefore, this bridge deck provides a duplicate in the category of epoxy-coated reinforcement with Class C concrete. Table 3.3 provides some of the results of the detailed survey. Figure 3.23 shows the results from the chloride analysis. The discussions of each bridge deck are presented in the following:

Structure #37-47-5980

This bridge is located at State Road 37 Northbound over US Highway 50 in the southwest region of Indiana. The bridge structure is a four-span continuous composite steel beam with a total length of approximately 91 m (300 ft) built in 1973. The estimated average daily traffic on this bridge is 5100 vehicles. The bridge deck has uncoated reinforcement and a design cover of 50 mm (2 in.) of Class A concrete as a corrosion protection method.

Class A concrete (maximum water-cementitious materials ratio [w/cm] of 0.532) is a more permeable concrete mixture that was used in bridge deck construction in Indiana until 1976. As shown in Table 3.4, the concrete cover survey found an average cover of only 44.45 mm (1.75 in.) with a standard deviation of 12.4 mm (0.49 in.). The chloride concentration profiles in Figure 3.23 show that the average chloride level at the depth of the top mat of reinforcement was in excess of the estimated level required for initiation of corrosion (Scannell, et al 1996). The level of chloride, however, is less than in four of the other bridge decks all constructed in later years.

Bridge structure #	Corrosion Protection method	Type of structure	Region of Indiana	Year built	Design cover
37-47-5980	Black reinforcing bar with Class A concrete	Continuous composite steel beam	Southwest	1973	50.8 mm (2.0 in.)
32-18-2182	Black reinforcing bar with Class C concrete	Continuous prestressed concrete I-beam	East central	1975	63.5 mm (2.5 in.)
6-50-5187	Black reinforcing bar with Class C concrete with 38j mm (1.5 in.) latex-modified overlay	Continuous prestressed concrete I-beam	North central	1980	63.5 mm (2.5 in.)
6-50-6624	Epoxy-coated reinforcing bar with Class C concrete	Hinged composite steel girder and beam	North central	1980	63.5 mm (2.5 in.)
331-50-6608	Epoxy-coated reinforcing bar with Class C concrete	Continuous welded girder	North central	1976	63.5 mm (2.5 in.)
6-50-6577	Epoxy- coated reinforcing bar with Class C concrete and uncoated bottom mat of reinforcement	Reinforced concrete slab	North central	1980	63.5 mm (2.5 in.)

Table 3.3 Bridge structures chosen for detailed survey

The location of this bridge structure explains this difference. The southwest region of Indiana is milder in climate and the bridge deck would be expected to receive smaller amounts of deicing salts, thus explaining the lower chloride concentrations. The field investigations found 32% of the surveyed area of the bridge deck exhibiting signs of corrosion distress. The majority of the distress on this bridge deck was found along the shoulder of the roadway. The shoulder area of the bridge deck also had the larger number of half-cell potential readings which indicates corrosion. Two of the three 102 mm (4 in.) cores taken through the reinforcement confirmed corrosion of the reinforcement.

Structure #32-18-2182

Located at Tillitson Avenue over State Road 32, the structure is a five-span continuous prestressed concrete I-beam bridge with a total length of approximately 96 m (315 ft) built in 1975. The estimated average daily

traffic on this bridge is 13,200 vehicles. The bridge deck was constructed with uncoated reinforcement and a design cover of 63.5 mm (2.5 in.) of Class C concrete as a corrosion protection method. The concrete cover survey found an average cover of 67.56 mm (2.66 in.) with a standard deviation of 15.5 mm (0.61 in.), which exceeds the design requirements. Class C concrete is the concrete mixture currently specified for bridge deck constructions in Indiana.

The chloride concentration profiles in Figure 3.23 show that the average chloride level at the depth of the top mat of reinforcement was the highest of all six bridges surveyed and in excess of the estimated level required for initiation of corrosion. The field investigations found 71% of the surveyed bridge deck area to be exhibiting signs of corrosion distress. The majority of the area of distress was found in the first two spans. The bridge deck also had extensive cracking and asphalt patching over the majority of the deck surface. The half-cell potential readings suggested corrosion activity in two bands. Delaminations were also found in these areas. Half-cell potential readings could not be taken in areas of patching due to the interference of the asphalt patching material. Two of the three 102 mm (4 in.) cores taken through the reinforcement confirmed corrosion of both the transverse and longitudinal reinforcement.

Figure 3.24 shows the core taken in an area of delamination. The delamination was severe in this area of the bridge deck and the hollow area underneath is evident in the picture. Corrosion damage was also observed on the prestressed concrete I-beams of the substructure.

Bridge structure #	Corrosion protection method	Year built	Percentage distress of surveyed area	Average cover	Number of 102 mm (4 in.) cores showing corrosion
37-47-5980	Black reinforcing bar with Class A concrete	1973	32	44.45 mm (1.75 in.)	2
32-18-2182	Black reinforcing bar with Class C concrete	1975	71	67.56 mm (2.66 in.)	2
6-50-5187	Black reinforcing bar with Class C concrete with 38 mm (1.5 in.) latex-modified overlay	1980	31	65.53 mm (2.58 in.)	1
6-50-6624	Epoxy-coated reinforcing bar with Class C concrete	1980	0.68	54.61 mm (2.15 in.)	1
331-50- 6608	Epoxy-coated reinforcing bar with Class C concrete	1976	0	55.63 mm (2.19 in.)	1
6-50-6577	Epoxy-coated reinforcing bar with Class C concrete and uncoated bottom mat of reinforcement	1980	0	74.42 mm (2.93 in.)	0

Table 3.4 Results of detailed survey

Chloride Concentrations vs. Depth

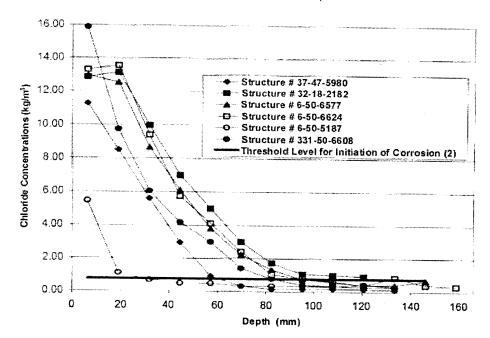


Figure 3.23 Chloride concentration profiles (Samples, et al 2000)



Figure 3.24 Delamination present in the bridge deck (structure # 32-18-2182) [Samples, et al 2000]

Structure #6-50-5187

Located at US Highway 6 over the Yellow River, this structure is a three-span continuous prestressed concrete I-beam bridge with a total length of approximately 42.67 m (140 ft) built in 1980. The estimated average daily traffic for this bridge is 7700 vehicles. The bridge deck has uncoated reinforcement and design cover of 25.4 mm (1.0 in.) of Class C concrete and 38.1 mm (1.5 in.) of latex-modified overlay as a corrosion protection method. The concrete cover survey found an average cover of 65.53 mm (2.58 in.) with a standard deviation of 6.6 mm (0.26 in.), which exceeds the design requirements.

The field investigation found 31% of the surveyed bridge deck area exhibited signs of corrosion distress, which was concentrated in one main area. Corrosion activity was also indicated by half-cell potential measurements in the same distress areas as found by the visual inspection and chain drag. One of the three 102 mm (4 in.) cores taken through the reinforcement confirmed corrosion of the reinforcement. The chloride concentration profiles in Figure 3.23 show that the average chloride level at the depth of the top mat of reinforcement was the lowest of all six bridges surveyed and slightly below the estimated level required for initiation of corrosion. One explanation for the high level of corrosion distress with a low chloride concentration may be related to moisture. The latex-modified overlay may trap moisture within the deck causing accelerated corrosion of the reinforcement. No data was collected during this investigation to verify this explanation.

Structure #6-50-6624

This bridge, located at US Highway 6 over State Road 331, is a three-span hinged composite steel beam with a total length of approximately 49 m (160 ft) built in 1980. The estimated average daily traffic for this bridge is 8200 vehicles.

The bridge deck has epoxy-coated reinforcement and a design cover of 63.5 mm (2.5 in.) of Class C concrete. The concrete cover survey found an average cover of 54.61 mm (2.15 in.) with a standard deviation of 8.9 mm (0.35 in.), which is lower than the design requirements. The chloride concentration profiles in Figure 3.23 show that the average chloride level at the depth of the top mat of reinforcement exceeds the estimated threshold level for initiation of corrosion.



Figure 3.25 Delaminated area with spalls and exposed reinforcement (structure # 6-50-6624)

The field investigations found only 0.7% of the bridge deck exhibiting signs of corrosion distress. This area of distress was localized to one region of the bridge deck. A close up of the area of distress with reinforcement exposed is shown in Figure 3.25. The rest of the bridge deck surface showed no signs of corrosion distress. The epoxy coating was removed from the bar in this area and extensive corrosion was evident. The field investigation found this region of

distress to have less than 1 in. of cover in the localized area surrounding the spalls and delamination. The 102 mm (4 in.) core taken through the reinforcement confirmed corrosion of the reinforcement and delamination in this region (Fig. 3.26).

The bar closest to the surface in this area showed extensive underfilm corrosion. The reinforcing bar located in the perpendicular direction under the top bar showed little sign of corrosion; however, the coating was debonded and easy to remove. The average epoxy coating thickness found on the bars removed from the cores was 197.6 μ m (7.78 mils) with a standard deviation of 20.1 μ m (0.79 mils).

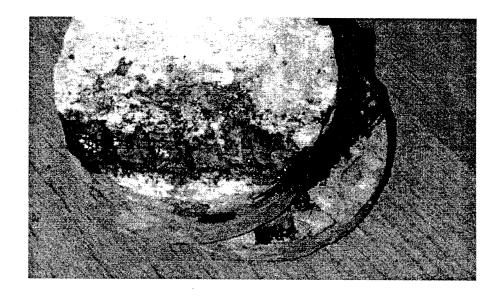


Figure 3.26 Core removed from delamination area of deck

Structure #331-50-6608

Located at State Road 331 over US 30, this bridge structure is a two-span continuous welded girder with a total length of approximately 69 m (225 ft) built in 1976. The estimated average daily traffic for this bridge is 2500 vehicles. The bridge deck has epoxy-coated reinforcement and a design cover of 63.5 mm (2.5

in.) of Class C concrete. The concrete cover survey found an average cover of 55.63 mm (2.19 in.) with a standard deviation of 7.4 mm (0.29 in.), which is lower than the design requirements. The chloride concentration profiles in Figure 3.23 show that the average chloride level at the depth of the top mat of reinforcement exceeds the estimated threshold level for initiation of corrosion. The visual field investigations found no evidence of corrosion distress; however, one of the 102 mm (4 in.) cores taken through the reinforcement at a cracked location showed signs of corrosion. Rust stains were evident at the rib locations of the reinforcement.

Figure 3.27 shows the reinforcement removed from the cracked region. The coating was cracked with evidence of corrosion. Also, the coating was easy to remove and underfilm corrosion was observed (Figure 3.28). No other signs of corrosion were found on any of the other removed bars. The coating of these bars was well-adhered and difficult to remove. The average epoxy-coating thickness found on the bars removed from the cores was 204.2 μ m (8.04 mils) with a standard deviation of 15.5 μ m (0.61 mils).



Figure 3.27 Alternate side of reinforcing bar removed from cracked area (structure # 331-50-6608)

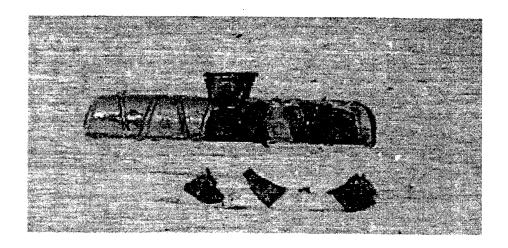


Figure 3.28 Underfilm corrosion found after removing coating (Structure # 331-50-6608)

Structure #6-50-6577

The bridge structure type is a reinforced concrete slab, located at US Highway 6 over Stock Ditch, with a total length of approximately 27 m (90 ft) built in 1980. The estimated average daily traffic for this bridge is 9500 vehicles. The bridge deck has epoxy-coated top mat reinforcement and an uncoated bottom mat reinforcement with a design cover of 63.5 mm (2.5 in.) of Class C concrete. The concrete cover survey found an average cover of 74.42 mm (2.93 in.) with a standard deviation of 13.5 mm (0.53 in.), which is greater than the design requirements.

The chloride concentration profiles in Figure 3.23 show that the average chloride level at the depth of the top mat of reinforcement exceeds the estimated threshold level for initiation of corrosion. The field investigations found no evidence of corrosion distress. None of the 102 mm (4 in.) cores taken through the reinforcement showed any signs of corrosion. Corrosion of the uncoated reinforcement in the bottom mat, however, was found along the edge of the underside of the bridge deck. The open guardrail design used in this bridge

deck promotes this damage. The average epoxy-coating thickness found on the bars removed from the cores was 205.2 μ m (8.08 mils) with standard deviation of 21.8 μ m (0.86 mils).

3.2.2.8 Summary and conclusions

The detailed field investigation of six bridge decks in Indiana found corrosion of epoxy-coated reinforcement in areas of cracking and insufficient concrete cover, in which the concentration of chlorides had exceeded commonly accepted threshold values. In the area of cracking in Bridge Structure #331-50-6608 where corrosion of the epoxy-coated reinforcement was found, no delaminations were present to indicate any sign of distress. A total of nine cores were removed from bridge decks containing epoxy-coated reinforcement and corrosion was discovered in two of these cores.

The coating on the reinforcement from these two cores was debonded, easy to remove, and underfilm corrosion was observed. The coating on the reinforcement was also debonded in one additional core, but no corrosion of the reinforcement was observed. Cracking and insufficient concrete cover may decrease the effectiveness of epoxy-coated reinforcement as a corrosion protection method when damage to the epoxy coating is present. Corrosion of the epoxy-coated reinforcement was discovered during the detailed bridge inspection in areas of cracking and shallow cover. Over the past 23 years, however, the use of epoxy-coated reinforcement with 2.5 in. (63.5 mm) of good quality concrete as a corrosion protection method has clearly outperformed all other methods used in Indiana.

The detailed field investigation also discovered the lowest level of chloride concentrations in Bridge Structure #6-50-5187, which contained a latex-modified overlay. The average chloride concentration at the level of reinforcement was

0.5 kg/m³ (0.8 lb/yd³), which is below the threshold level of 0.77 kg/m³ (1.29 lb/yd³) for initiation of corrosion (Samples, et al 2000). The level of distress, however, in the form of delaminations, spalling, and patching was 31% of the surveyed area.

3.3 Deck joint

Deck joints are among the smaller elements of a bridge structure, but when they fail to function properly, joints can create problems out of proportion to their size. For example, when they leak, the underlying structural elements deteriorate, or when they are unable to move, bridge elements may experience over stress or even damage.

The potential for developing serious structural problems from joint problems comes from their role in accommodating necessary structural movements of many types. In a bridge structure a number of external and internal factors, such as variations in the ambient temperature, imposition of live loads, earthquakes, shrinkage, relaxation, camber growth, and settlement, can initiate movement of the superstructure relative to the substructure. Depending on the initiating factor, this movement may be longitudinal, transverse, or rotational about a longitudinal or transverse axis.

The majority of existing bridges utilize deck joints to accommodate movements at the superstructure level without sustaining any damage to the bridge or approaches. Some newer bridges, on the other hand, utilize continuous (jointless) deck slabs and rigid connections between superstructure and substructure. These bridges accommodate the superstructure movements at substructure level by relying on the flexibility of piles, deformation of slender substructure elements, or hinged columns.

There are ten types of deck joints that are often used in the expansion joint. A typical deck joint rehabilitation (or replacement) procedure starts with the removal of the deteriorated or damaged joint, and the debris and dirt from the deck joint opening. Then the deck substrate is repaired to ensure a firm support and anchorage of the joint system. The repair ends with the replacement of the joint components. More detailed descriptions of the rehabilitation procedures for several joint types are presented in the following:

3.3.1 Open joints

Open joints are mostly encountered in old short span bridges, where the movements to be accommodated are typically smaller than those in longer span bridges. They are constructed by forming a gap between a deck slab and an adjoining deck slab, abutment or approach slab.

During bridge rehabilitation, open joints are usually replaced with watertight joints or modified to add joint armoring and a joint drainage system, which may include a (metal or elastomeric) trough, a collection (silting) basin, and downspout pipes. Armoring provides protection against live load impact, and the joint drainage system protects surrounding bridge components from the harmful effects of infiltrating water and chemicals.

If an open joint is being replaced in kind, the practical range for the width of the joint is from 1/2 to 2 inches (13 mm to 51 mm). The smallest joint opening allows enough gaps for movements of a short span bridge without causing any structural damage, whereas the largest opening still provides for a riding surface with acceptable smoothness. Changing the direction of an open joint or extending the joint into a barrier curb during rehabilitation work can be done simply by placing the concrete formwork in the desired direction and location. Although the initial construction cost for an open joint is relatively low, open joints

are prone to the intrusion of deicing salts and water, creating costly repairs on surrounding bridge components in the long run. Therefore, open joints are seldom used in new bridge structures, and are often worth replacing by other types during rehabilitation of existing structures.

Deteriorated deck slab concrete along the deck joint opening is removed to expose sound concrete. The exposed reinforcement would then be cleaned and epoxy coated followed by reconstructing deck slab corners using quick-setting patch materials or high early strength concrete. Alternatively, deck slab is removed to half of its depth along the entire length of the joint. The exposed reinforcement is then cleaned and epoxy coated and the deck slab reconstructed by installing angles for protection of slab corners. As a minimum measure, debris is removed from joint opening and drainage trough. If a trough system does not exist, one is installed to prevent intrusion of water, debris, and deicing salts to the parts of the bridge below the deck level. (Figure 3.29)

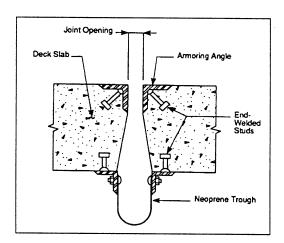


Figure 3.29 Open joint (Parsons, 1992)

3.3.2 Filled joints

Filled joints are inexpensive and easy to install and maintain. As a result, they have enjoyed widespread use in bridges. They are frequently

encountered in existing bridges, and continue to be used in new bridges and in the deck joint rehabilitation projects. A filled joint is an open joint filled and sealed with a flexible and compressible material. The joint sealers, depending on the manner they are applied, are classified as fieldformed, useful for movements up to 1 inch (25 mm), or preformed, useful up to 4 inches (102 mm).

Fieldformed (Figure 3.30) joint sealers are commonly composed of hot applied thermoplastics or chemically cured thermosetting sealants. Application of a fieldformed joint sealer requires a preformed backup material and waterstop beneath it to control the sealer depth and shape, and at the same time provide support to the sealer. The fieldformed sealers usually have limited service life because of poor installation conditions and workmanship that are often encountered in the field.

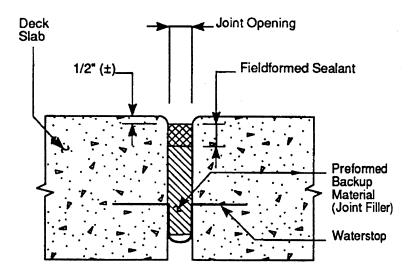


Figure 3.30 Filled joint with fieldformed sealant (Parsons, 1992)

Preformed sealers (Figure 3.31) are somewhat newer and hence have a shorter record of proven service than field-formed sealers. An important advantage they

offer is quick installation time and less interruption to traffic. Most commonly used types of preformed joint sealers for deck joint rehabilitation projects are extruded shapes made of elastomeric material. To install them, the contact surfaces between the sealer and the joint opening are cleaned and primed with adhesive. The preformed sealer is then inserted into the gap. With this type of joint, the installation is achieved without compression (unlike compression seals), but a firm contact between the joint filler and the sides of the joint is maintained during the curing process by inflating the joint sealer under air pressure.

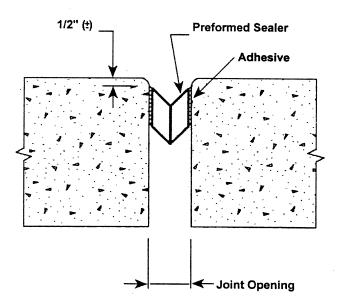


Figure 3.31 Filled joint with preformed sealer (Parsons, 1992)

Advantage

During joint rehabilitation work, it is relatively easy with either type of sealer to change the direction of a filled joint or to extend the joint into a barrier or curb. The best result is obtained when the sealer is extended into the curb at the level of the deck slab.

3.3.3 Sliding plate joints

Sliding plate joints are quite frequently encountered in existing medium span bridges, and they continue to be used in new bridges and in the rehabilitation of existing deck joints. Most of the existing sliding plate joints were constructed without any joint drainage system, since the joint itself was considered to restrict the amount of infiltrating water to a minimum. The joint does not, however, completely eliminate the intrusion of water. Therefore, if an existing sliding plate deck joint is rehabilitated or a sliding plate deck joint provided as a replacement, a trough system is often needed beneath this type of joint for long term protection of the surrounding bridge components.

The sliding plate joint (Figure 3.32) features a steel plate spanning an open joint and embedded in adjoining deck slabs. It can also be arranged to bear on the steel structure (made up of angles or other shapes that are embedded in the deck slab at each end). At one end, the sliding plate is secured by bolts or weldment to the steel substrate, and at the other end, it rests freely on the steel element on the adjacent deck slab. Superstructure movements are accommodated by the sliding action of the plate at the free end. Sliding plate joints can accommodate up to 4 inches of total movement. Direction changes in the joint that may be required during deck joint rehabilitation can easily be achieved by welding the steel plates and shapes at such location. Recent designs have incorporated springs to reduce movement in the plate.

Spalled deck slab corners are repaired along the length of the joint by following the procedure provided for open joints. The corroded, bent, jammed, or cracked plates and anchorages are removed and replaced. Also, debris accumulation from the joint and drainage trough could be removed. Where a trough does not exist or the existing one has deteriorated, a new trough (Figure 3.32) is installed. Sliding plates are fastened at one end and free to move at the other, providing a

simple way to cover moderate-sized gaps.

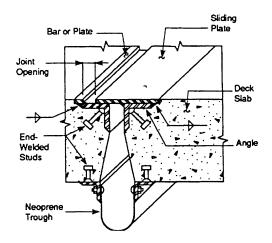


Figure 3.32 Sliding plate joint (Parsons, 1992)

3.3.4 Finger plate joints

Finger plate joints have been successfully used in medium and long span bridges for some time. They continue to be a popular option for new medium and long span bridges or in deck joint rehabilitation projects, as they are able to accommodate relatively large movements.

Finger plate joints (Figure 3.33) are made up of two loosely interlocking pieces of steel plates that cantilever into the deck joint opening. The cantilevered portion of each plate is made up of rows of finger-shaped protrusions that fit into the rows of grooves in the opposing plate. The finger plates are anchored into the deck slab or directly attached to the underlying superstructure steel.

Whenever the bridge spans undergo a movement, the finger plates move back and forth into the opposite grooves and accommodate this movement. Properly design finger plates remain overlapped at all times to secure a continuous riding surface and can accommodate total movements from 4 to 24 inches (102 to 610 mm). Performance and service life of a finger plate joint can be enhanced by

limiting the size of openings on finger plates to permit safe operation of narrow-tired vehicles or motorcycles, aligning the fingers in the direction of the longitudinal bridge axis, and adopting fatigue-resistant details. Special details can be developed, if bicycles will cross the joint.

When a finger plate joint is considered as a replacement alternative for an existing joint, it is important to keep in mind that achieving sharp direction changes in the joint is difficult, because of heavy steel construction and the small tolerance between opposing finger plates. Small angle changes, however, can be made by welding the plates as required. In the curb areas, normally a sliding plate joint is installed and connected to the finger plate joint. A finger plate joint system can also be used along the face of a sidewalk curb or barrier curb, if the rotation at the end of the span is not significant.

In most existing finger plate joints, the water and debris passing through the finger joint are collected and carried away by a trough system similar to the one described for open joints. If no through systems exist, or the existing one requires replacement, accumulation of debris and eventual clogging of the trough can be prevented by placing the new trough to a steep slope. A transverse slope of 1 inch per foot (8.3 mm per 100 mm) is adequate to promote a self-cleaning action by the flow of water in the trough. The trough is usually neoprene sheet, while older bridges could have used copper.

The repair procedures are the same as for sliding plate joints. The relatively long projections of finger plates can accommodate total movements as large as several feet. Finger plate joints are permitting water to leak through and deteriorate surrounding bridge elements.

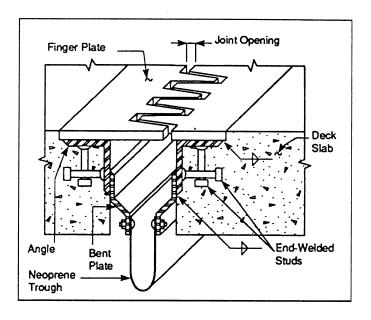


Figure 3.33 Finger plate joint (Parsons, 1992)

3.3.5 Sawtooth (serrated) plate joints

Another frequently encountered joint type in existing medium span bridges is the sawtooth plate joint. This type of joint is made up of two steel plates cut in a serrated fashion (Figure 3.34). The projections in one plate fit into the recesses of the other plate. The construction and function of this type deck joint are very similar to those of the finger plate joint. The only difference is the size of the projections (they are shorter length and wider in width) in comparison with those of the finger joints.

Sawtooth joints are still used in new bridges and considered as an alternative in deck joint replacements, where total movements in the range of 3 inches (76 mm) need to be provided. The direction changes in the joint layout and barrier curb applications can be easily achieved by welding the steel plates. As with sliding plate joints and finger plate joints, providing a trough system with a sawtooth joint benefits overall protection of bridge components.

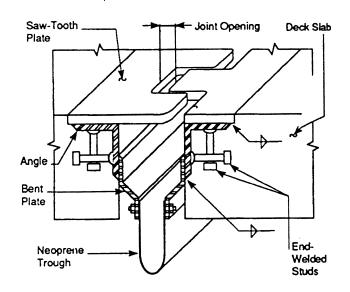


Figure 3.34 Sawtooth plate joint (Parsons, 1992)

3.3.6 Compression seal joints

Compression seals are made of either preformed closed-cell plastic or (more commonly) hollow extruded neoprene shapes (Figure 3.35). The seals are generally installed by squeezing and inserting the seal into a preformed joint opening. Properly sized seals remain in compression under all anticipated deck joint movements. To improve the water tightness of the joint, the contact surfaces between the gap and seal are coated with a high-solid urethane adhesive prior to the insertion of the compression seal into the joint opening.

The number of successful armored neoprene compression seal applications in the past decades (National Cooperative Highway Research Program, Synthesis 141, 1989) have made this type of seal probably the most popular one. A large variety of choices in movement ranges, watertightness, relative ease of installation, and cost effectiveness have all contributed to the success of neoprene compression seals. They are often considered to be an alternative, when an existing joint needs replacement or upgrading. Closed-cell plastic

compression seals, on the other hand, have had their greatest use in buildings, where movement requirements and environmental exposure are limited, rather than in bridges.

Performance and useful life of a neoprene compression seal in a new or rehabilitated joint depend primarily on the quality of the installation and the correct choice of the seal size and seal material. Compression seals that are manufactured from ozone-sensitive neoprene compositions have been known to lose their elasticity and harden after several years of service. The watertightness of such a hardened joint will fail, when bridge spans contract at low temperature and cause the hardened seal to pull away from the sides of the joint until the high solid-urethane adhesive debonds.

Along with adherence to proper installation, sizing, and material selection guidelines, the useful life of a neoprene compression seal joint can be improved by armoring the deck slab corners with steel angles or shapes. Neoprene compression seals are available in a variety of configurations and movement ratings. The largest size seal can provide for a total movement of 4 inches (102 mm).

When used in deck joint rehabilitation, the small angle changes in the direction of neoprene compression seal joints can be accommodated by bending the seal. However, applications that involve sharp angles or extensions into barrier curbs require splicing of the seal by heat bonding or cyanoacrylate adhesive. If the joint is skewed so that the seal will need to accommodate transverse as well as longitudinal movement, special care must be taken in the design.

Spalled deck slab edges are repaired following the procedure given for open joints. The corroded, broken, bent, and cracked portions of joint armoring and anchorages would be removed for replacement. Additionally the debris

accumulation from the seal would also be removed. If the splice location in the existing seal were leaking, the particular region would be cleaned and sealed with adhesive (Figure 3.35). The compression seal joint is designed to remain always in compression; these seals have become quite popular, particularly with neoprene as the seal material.

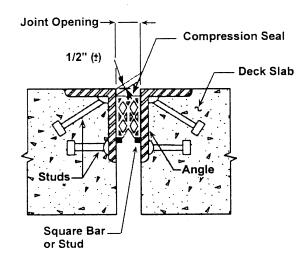


Figure 3.35 Compression seal joint (Parsons, 1992)

3.3.7 Strip seal joints

Although strip seal joints were introduced for bridge use later than compression seals, they have established a successful performance record comparable with that of neoprene compression seals. They continue to be a popular choice in deck joint replacement projects. Because of the locking nature of the seal, the strip seal system performs better than a compression seal at locations in which transverse slab movements are anticipated and provides a superior seal against water leakage.

Strip seal (Figure 3.36), as the name implies is a strip of specially shaped elastomeric material that spans a deck joint opening. The seal is mechanically locked into a pair of rolled or extruded metal shapes that are in turn anchored to

the edges of the deck slabs. These metal shapes serve two important functions: they provide end restraint to the seal so that the strip seal can function in tension or compression depending on the direction of the deck slab movement and they protect the edge of the deck slab against live load impact.

seals are available in a number of configurations and a wide range of movement ratings. The largest size strip seal can provide up to 5 inches (127 mm) of total movements. However, most design authorities limit the total movement to 4 inches (102 mm). During the course of deck joint rehabilitation and replacement, the necessary direction changes in the strip seal deck joints can be achieved by welding the steel plates and vulcanizing the elastomeric seal material. Certain types of strip seals can accommodate 30° to 35° turns in the joint configuration without a splice. The repair procedures are the same as those for compression seal joints. Strip seals combine elastomeric material with metal supports for a locking seal, often favored at locations where differential movements are anticipated.

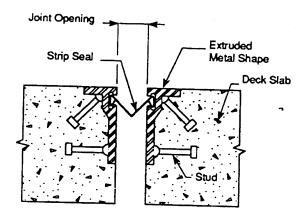


Figure 3.36 Strip seal joint (Parsons, 1992)

Advantage

Because of the locking nature of the seal, the strip seal joint performs better than

a compression seal at locations in which transverse slab movements are anticipated and provide a superior seal against water leakage.

3.3.8 Sheet seal joints

Sheet seal (Figure 3.37) is a sheet of fiber reinforced elastomeric membrane with a center corrugation that links a deck opening. At both ends, the seal is held down and anchored into the corners of deck slabs by means of metal, elastomeric or combination hold-down bar (retainer bars), and anchor bolts. Similar to the strip seal, the sheet seal functions either in tension or compression, and the deformation of the center corrugation accommodates the deck slab movements. Sheet seals represent one of the possible choices for deck joint replacements in existing medium span bridges, and are available in a variety of shapes, configurations and sizes. A maximum of 4 inches (102 mm) of total movement is easily obtainable in a sheet seal. Spalled deck slab corners along the length of the existing joint are repaired following the procedure Damaged hold-down bars and anchorages are described for open joints. removed for replacement and the debris accumulation cleaned from the seal (Figure 3.37). These elastomeric seals can accommodate changes and skews in joint configuration, but depend on close contact and may loosen under repeated live loads.

Advantage and Disadvantage

One important advantage of the sheet seal over the compression seal is the ability to accommodate directional changes and skews in the joint configuration often without any need for a splice in the seal. Sheet seals do not allow water to leak through and deteriorate the surrounding bridge element. Failure of anchorage systems, under repetitive live-load impact, has been a frequently encountered problem with the sheet seals.

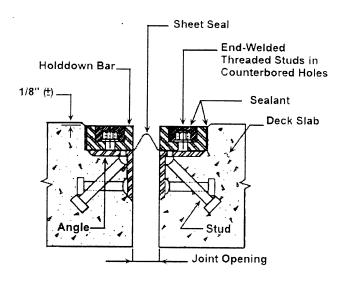


Figure 3.37 Sheet seal joint (Parsons, 1992)

3.3.9 Plank seal joints

Plank seals (Figure 3.38) are molded neoprene sections of varying widths. The seal spans the deck opening and is bolted down to the deck slab at each end. Although the performance record of plank seals has not been as satisfactory to those of compression or strip seals, they continue to be an alternative for the replacement of existing joints in medium and long span bridges.

A typical cross section displays a number of grooves placed alternately on each face of the neoprene plank, and metal plates spanning between the grooves. These metal plates are internally or externally bonded to the seal, and reinforce the seal against vertical deformation under live load. Additionally, the checkered metal plates that are placed on the roadway face of the seal improve the skid resistance and protect the seal against snowplow damage, or simple wear and tear. A plank seal accommodates the deck slab movements by the closing and opening motions of the grooves in the plank surface. This type of joint system, depending on the width of the plank and the number of grooves, can allow total movements ranging from 1.5 inches to 13 inches (38 to 430 mm).

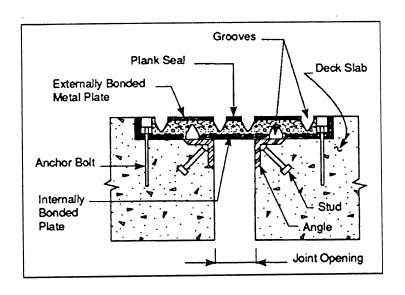


Figure 3.38 Plank seal joint (Parsons, 1992)

Plank seal are usually manufactured in 5 to 6 foot long (1.53 m to 1.83 m) segments and joined together by means of butt joints, or tongue and groove connections to form a leakage free surface. Vertical and horizontal variations in the joint configuration are achieved by welding the (bonded) metal plates and bonding the neoprene seals at such interfaces. The proper functioning and watertightness of plank seal joints depend on the quality of workmanship involved in their installation and the performance of the seal anchorage under repetitive live load impact. Leakage at joints between segments, loose anchorage, excessive noise, and snowplow damages (National Cooperative Highway Research Program, Synthesis 141, 1989) have been the problems commonly reported with the existing plank seal joints. The repair procedures are the same as those for the sheet seal joints. These molded neoprene sections can accommodate joint movements to 13 inches, but seal anchorage may loosen under prolonged service.

3.3.10 Modular joints

Modular joints represent the state-of-the-art approach to accommodating the

complex movements in long span or curved bridges. Although the number of past applications are not as numerous as single compression or strip seals, the success rate of modular joints, especially the ones with steel components, appears to be very favorable (Kazakavich, 1987; National Cooperative Highway Research Program, Synthesis 141, 1989). Therefore, when an existing joint with a large movement needs replacement or upgrading, the modular joint is usually the recommended choice.

The modular joint system is composed of three main components: sealers, separator beams (for sealers), and support bars (for separator beams). Sealers and separator beams form a watertight surface at the level of the riding surface and accommodate static and dynamic deformations by virtue of the deformation in the sealers. Sealers can be of compression, strip, or sheet seal type. Separator beams are often extruded or rolled metals shapes, and provide for the joining of seals in series. The separator beams are supported on support bars at frequent intervals.

The support bars span the joint opening, and the ends of the support bars fit into a compressible (spring-like) restraint system. This system is composed of two polyurethane or elastomeric blocks. One block (upper bar or spring) rests on top of the support bar, the second block (bearing) fits under the support bar, and both blocks are, in turn, attached to the deck substrate. The deformation in the spring and bearing blocks combined with the sliding action taking place (because of the presence of low-friction surface) between these blocks and support bar can accommodate all possible deformations that sealers normally undergo. The presence of spring and bearing block system also helps to eliminate noisy metal-to-metal contact and dampens the live load impact.

The proper functioning of the modular joint depends on maintaining equal deformations among the sealers. This control is achieved by providing support

bars at set intervals. Each support location may have multiple or single support bars. A multiple system provides an individual support bar to each separator beam and introduces compressible blocks between these support bars to equalize the distances between separator beams (multiple support bar control type, Figure 3.39). Alternately each location may provide only a single support bar for all separator beams, with a system of intermediate short support bars and compressible blocks placed under the separator beams to maintain equal spacing between separator beams (single support bar control type, Figure 3.40) (Koster, 1989). In general, the multiple support type gives greater redundancy in case of failure of a bar and can better accommodate rotations and differential settlements than the single support type.

The modular joint system, because of its refined mechanical performance, can accommodate the complex movements of long span bridges as well as those of horizontally curved bridges. The modular system available in the market today can provide total movements in the range of 4 feet (1.22 m), and also permit nonparallel horizontal movement, differential settlement, rotation, and high shearing movements. During deck joint replacement, the necessary changes in the modular joint configuration are achieved by forming welded joints in the separator beams, and by bending or splicing the seals. Although barrier curb applications can be achieved in the same manner, it is more practical to extend the joint into the barrier curb at deck level and provide a sliding plate joint system along the face of the barrier curb.

The watertightness and adequate performance of a modular joint depend on good workmanship during the installation and on maintaining an equal distance between sealers and providing for fatigue-resistant metal components and connections. Based on the field performance record of six different modular joint systems (Kazakavich, 1987), the main points of concern with this joints appear to be the noise under live load impact, water leakage at seal splice locations, debris

accumulation in seals, and snow plow damage.

During deck joint replacement, the necessary changes in the modular joint configuration are achieved by forming welded joints in the separator beams, and by bending or splicing the seals. Spalled deck slab corners along the length of the existing joint are repaired following the procedure described for open joints. Damaged seals and separator beams and other accessible hardware are removed for replacement. The debris accumulation from the seals is removed (Figures 3.39 and 3.40). Modular joints provide state-of-the-art joints for joint movements up to 4 feet. This type of joint features an individual support bar at each separator bar. Various sealant materials can be combined with beams and support bars.

Disadvantage

Based on the field performance record of six different modular joint systems, the main points of concern with these joints appear to be the noise under live-load impact, water leakage at seal splice locations, debris accumulation in seals, and snowplow damage.

3.3.11 Case studies

i) Camsley Lane Viaduct, Cheshire (Ambrose,1985 and Thompson,1989)

The six reinforced concrete spans were constructed in 1963 to carry the M6 over the A56 and a railway. After 20 years, the reinforced concrete substructure showed widespread staining, cracking, delamination and spalling as a result of chloride deicing salt leaking through the expansion joints. Asphaltic plug joints were installed and the waterproofing and drainage of the deck were improved before carrying out the repairs to the concrete. Carbonation depths were only 3

to 5 mm, but chloride concentrations up to 5% by weight of cement were found near the surface and between 1 and 2% at depths between 100 and 150 mm. During the winter of 1984-85, with appropriate jacking support, concrete was removed to at least 20 mm behind the reinforcing bars to expose at least 50 mm of uncorroded bar. After grit blasting, replacing bars of less than 90% nominal size, coating steel with an inhibiting primer and saturating the concrete, repair material was placed. A flowable concrete was cast against formwork in lifts up to 1.5 m and a repair mortar was applied in layers for areas less than 0.1 m².

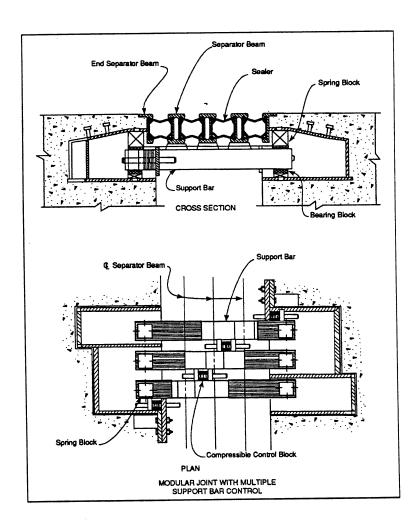


Figure 3.39 Modular joint with multiple support bar control (Parsons, 1992)

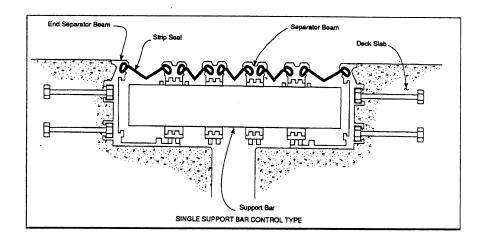


Figure 3.40 Modular joint with single support bar control (Parsons, 1992)

As chloride contamination was still present, the effectiveness of the repairs was monitored by TRRL. Thompson (1989) found from surveys that after three years new corrosion sites had developed and that corrosion continued in some repaired areas. He concluded that half-cell potential mapping alone was inadequate. But, the half-cell potential mapping, in conjunction with chloride concentrations, would lead to criteria with less risk of corrosion.

ii) Garden State Parkway Bridge (Parsons, 1992)

The three-span Garden State Parkway Bridge is located in New Jersey, over local creeks. The existing obsolete armored open joints are to be replaced with more watertight joints-armored compression seals. Although the existing open deck joints were paved to achieve watertight joints during an earlier deck overlay work, cracking and deterioration in the overlay allowed the joints to continue to leak and promote deterioration at the pier ends of the steel girders and bearings. The planned joint replacement process involves removing the existing overlay and the portions of deck slab over the end diaphragms, then installing a new joint assembly, consisting of compression seal and armoring angles, and connecting it to the existing end diaphragms using new steel plates and shapes. Finally, placing the concrete around the joint assembly completes the slab

and joint reconstruction. The resulting joint gives superior water tightness with minimum slab removal. (Figure 3.41)

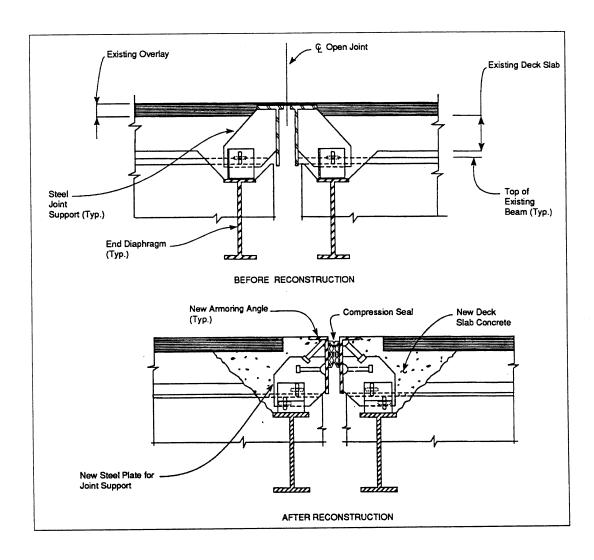


Figure 3.41 Replacement of open joint by armored compression seal for joints on the Golden State Parkway Bridge (Parsons, 1992)

iii) Route 71 bridge (Parsons, 1992)

The bridge is located on route 71 over the Shark River in New Jersey. The existing finger plate joints lacked a trough system and were permitting water to leak through and deteriorate surrounding bridge elements. To prevent further

substructure deterioration and improve joint performance, the existing finger plate joints at the abutments and piers were replaced by sheet seals with steel hold-down blocks. First, portions of deck slab above the end diaphragms were removed, including the reinforced concrete encasement around the end diaphragms. The new joint substrate was formed by installing steel angles and placing concrete around them. Then the sheet seals were installed and joint restoration completed. In line with these replacements, the filled joint between the abutment backwall and approach slab was removed and replaced (Figure 3.42).

iv) Cherry Hill and Littleton Road bridge (Parsons, 1992)

The bridge is located over route 80 in northern New Jersey's Morris County. All deck joints on this 400-foot long four-span bridge were of compression seal joint type. After 20 years of service, these seals were aged, brittle, and leaking. This was caused by spalling of the deck slab edge and loss of elasticity and debonding of the seal. Joint replacement was performed concurrently with deck rehabilitation and overlay. After removal of the existing compression seals and the surrounding portions of the deck slab, new compression seals with armoring angles were installed at elevations even with the new deck overlay. The deck slab around the new deck joint was repaired with quick setting patch material before placement of the new deck overlay (Figure 3.43).

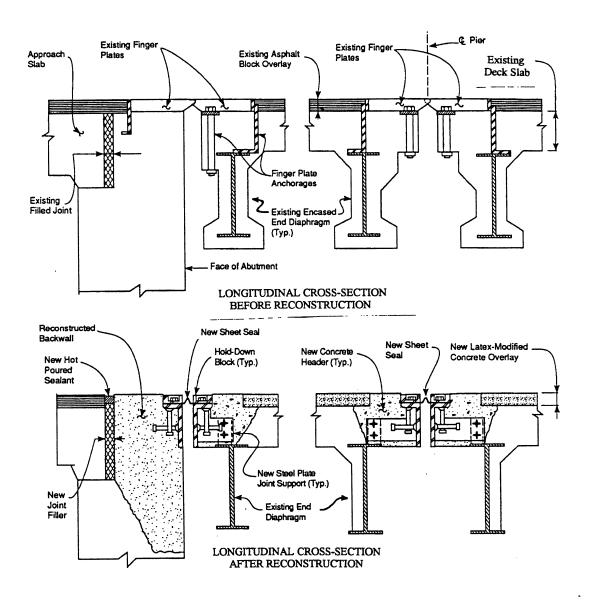


Figure 3.42 Replacement of finger plate joints by sheet seals on route 71 Bridge (Parsons, 1992)

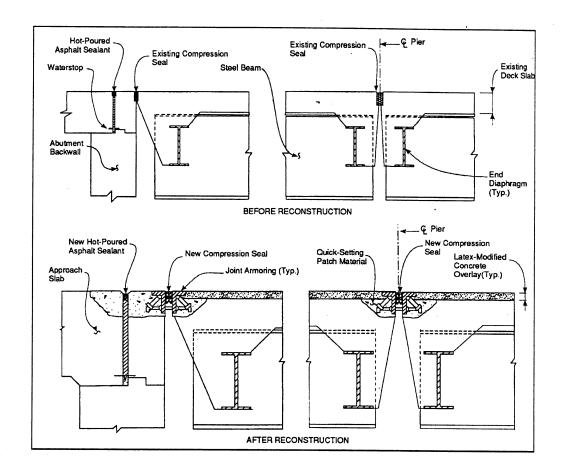


Figure 3.43 Replacement of compression seals in kind on Cherry Hill and Littleton Road Bridge (Parsons, 1992)

v) Rehabilitation of an Elevated Roadway Bridge (Vaysburd, 1989)

The rehabilitation project carried out between September 1988 and July 1989 on the concrete bridge at Baltimore/Washington International Airport is presented by Vaysburd (1989). Built in 1977, the bridge superstructure is supported by 152 concrete columns and two abutments on spread footings. The 1720 ft (524 m) long concrete structure consists of 73 spans in continuous units three or four spans long, with cantilevers at the end of each unit and a suspended slab from 12 ft (3.7 m) to 28.8 ft (8.8 m) long between cantilevers (Figure 3.44). An expansion joint is provided at each end of the suspended span and at each abutment.

The width of the bridge varies from 24 to 44 ft (7.3 to 13.4 m) of clear roadway with a parapet and a safety walk on one side of the bridge and a sidewalk on the other. The bridge deck is a 20 in. (51 cm) thick cast-in-place reinforced concrete slab. The positive moment regions in the deck slab in each 24 ft (7.3 m) clear roadway span contain 20 equally spaced 10 in. (25 cm) diameter voids. The specified concrete strength for the columns and deck slab was 4500 psi (31 Mpa).

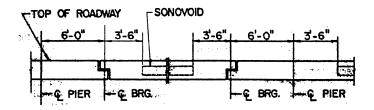


Figure 3.44 Typical suspended span longitudinal section. (Vaysburd, 1989)

The bridge was visually inspected for spalls, cracks, rust stains, discoloration, corrosion of reinforcement, and other indications of concrete deterioration. The surface areas were also inspected by sounding (hammer tapping) to locate delaminated areas. The inspection revealed the following:

- The top of the slab had some minor spalls along the expansion joint angles;
- Cracking, light at most locations and moderate at others, existed throughout the deck surface. Transverse cracking was predominant, although some longitudinal and random cracking was observed.
- Severe deterioration was found on the underside of the deck slab at the expansion joints. Leakage through the joints had caused extensive corrosion of the reinforcement, spalling, and delamination of the concrete

cover (Figure 3.44). Concrete spalls at the cantilever supports were about 4 in. (10 cm) deep at several locations and the main reinforcing bars were heavily corroded.

- The concrete sidewalk at several locations had severe random cracking with crack widths varying from fine to ½ in. (6 mm).
- Of the 30 expansion joints, 26 had preformed neoprene elastomeric compression seals. Four joints had been repaired in 1986 by replacing the existing compression seals. All of the remaining joints were showing evidence of leaks (some very severe).

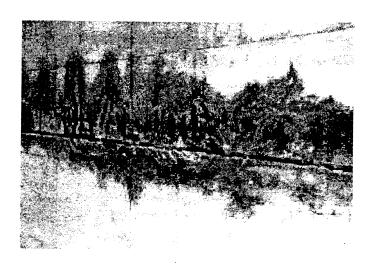


Figure 3.45 Spalling of concrete along the expansion joint (Vaysburd, 1989)

A concrete sampling and testing program was carried out to supplement visual observations and determine the potential durability of the concrete and the expected service life of the structure. This program included testing cores and samples for compressive strength, chloride ion content, depth of carbonation, pH value, and for performing petrographic examinations. The most severe areas of concrete deterioration were adjacent to expansion joints on the underside of the

deck slab. Cores taken near the expansion joints (Figure 3.46) were visually examined and tested for compressive strength. A total of 15 cores were taken from various locations throughout the bridge.

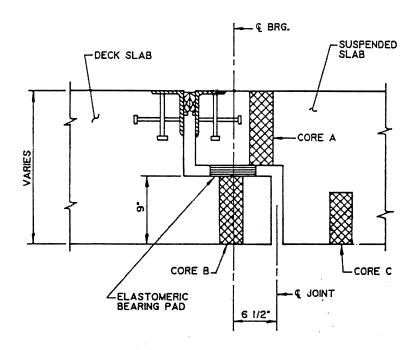


Figure 3.46 Core location sections (Vaysburd, 1989)

The bottom portions of the cores (inside the joint) contained deteriorated concrete from 1/2 in. (12.5 mm) to 3 in. (76 mm) thick. Some of the cores contained delaminations and cracks, and the reinforcements in two cores had corroded to the extent that the bond between concrete and steel was severely affected. Average compressive strength of the two cores tested was 7590 psi (52 Mpa).

Measurements of depth of carbonation using a phenolphthalein indicator showed that color changes occurred at an average depth of 5/16-in. (8 mm). Tests for pH value indicated an alkalinity of the concrete of about 11. Sampling and testing for total chloride content was carried out in accordance with AASHTO T-260. The shallow samples showed levels of chlorides that ranged from 0.3 to 7.4 lb/yd³

 $(0.18 \text{ to } 4.4 \text{ kg/m}^3)$ of concrete, and the deep samples ranged from 0 to 3.2 lb/yd 3 (0 to 1.9 kg/m 3).

The results of the petrographic examination indicated that the concrete had been affected by an alkali-silica reaction that generated fine cracks in the concrete. The orientation of the cracks was both parallel and normal to the exterior concrete surface. The cracks were shallow at some locations and penetrated to a depth of about 1 in. (2.5 cm) at other locations.

Repair of spalls along the expansion joints

Pumped concrete was selected as the repair material, since the most important factor in selecting methods and materials for repair is the compatibility of existing concrete and the new materials. Prior to the repair operation, temporary supports for the suspended slabs were installed. The concrete surface was prepared by removing the deteriorated concrete to sound concrete, but not less than to a depth of 1 ½ in. (3.8 cm) behind the main reinforcement. At several locations, the depth of deteriorated concrete was about 4-1/2 in. (11.5 cm) into the cantilever support of a suspended slab and the cantilever support was completely replaced with new concrete. The chipping was done using hammers with a maximum weight of 30 lb (14 kg). Specifications recommended removing concrete until the coarse aggregate particles were being broken rather than simply removed from the cement matrix. The concrete surface and exposed reinforcing steel were then sandblasted and finally cleaned of loose material with compressed air. An anchoring system was installed using self-drilling anchors for 3/8-in. diameter hooked bolts spaced in a 12 x 12 in. (30.5 x 30.5 cm) grid (Figure 3.36). Just prior to pumping the repair concrete, the existing surface was dampened with water. The pumped concrete was an air-entrained portland cement concrete mixture with a minimum compressive strength of 5000 psi (31Mpa). Mix proportions for the pumped concrete are given in Table 3.5.

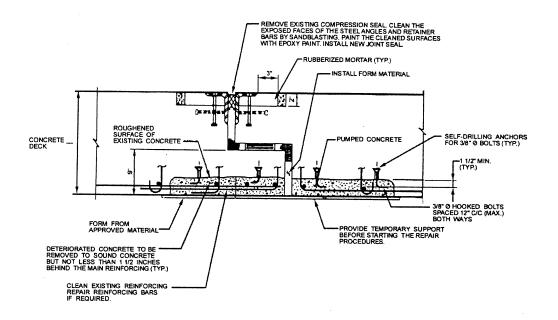


Figure 3.47 Expansion joint repair detail (Vaysburd, 1989)

Material	Mix proportions in lb/yd ³	Mix proportions in kg/m ³
Portland cement Type I	750	445
Fine aggregate	1400	830
Coarse aggregate	1650	980
(AASHTO M43 SIZE 7)		
Water	31 gals.	155
Water reducing	112 ozs.	3 L
admixture	9.5 ozs	0.6 L
Air entraining admixture	6.5 ± 1.5 %	6.5±1.5%
Air content		
Total	4066	2414
Slump	8 ± 1 in	20 ± 2.5 cm
Minimum 28-day		
compressive strength	5000 psi	35 MPa

Table 3.5 Concrete mix for pumped concrete

Repair of cracks

In order to repair cracks wider than 1/8 in. (3 mm) and cracks with spalled edges in the horizontal surfaces of the sidewalks, a low viscosity epoxy that has a workable life of approximately 45 minutes at 75° F and feeds into the crack by gravity until it hardens, was used. The exposed face of the crack was V-grooved ¾ in. (19 mm) deep and ½ in. (12 mm) wide, cleaned and sealed with epoxy. The remaining cracks in the deck slab, abutments, parapets, and retaining walls were repaired using injection of a two-part epoxy composed of a resin and hardener.

Repair of expansion joints

The repair of expansion joints included installation of elastomeric concrete nosing, epoxy coating of the steel angles, and installation of new seals. The elastomeric concrete used was a pre-packaged material consisting of three components: a resin, a hardener, and a mineral filler material. The existing concrete was removed along the expansion joints, the surfaces receiving the new material were cleaned, and a self-leveling elastomeric concrete mix was placed. The old seal was removed from the joint and the top and inside portions of the steel angles were blast cleaned to a white metal finish immediately before the application of the bonding agent and the installation of the new seal material. After installation of the new seal, epoxy paint was applied to the exposed surfaces of the expansion joint angles.

The final step in the project was the application of a penetrating sealer to the bridge deck, sidewalks, parapets, and abutments to minimize moisture and chloride penetration into the concrete.

vi) Structure X6, Egerkingen, Switzerland (Mallet, 1994)

Built in 1966, the two 50° skew slab decks carry four lanes southbound and five lanes northbound of Highway N2 over a Cantonal road. The slabs suffered damage due to ingress of salt water to their adjoining edges and at their supports due to leaking expansion joints. The bearings and abutment sill beams were also badly corroded. It was decided to make a clear opening longitudinally between the two slabs, to remove defective concrete from the support regions and make the deck slabs monolithic with the abutment walls. Rehabilitation was carried out in 1989-90, keeping two lanes of the N2 open in each direction throughout, except for two closures of one night each when traffic was diverted to the opposite carriageway. Traffic on the Cantonal road was controlled by traffic light and a height limit of 4.3 m was applied.

3.4 Repair of reinforced concrete beam

3.4.1 General (White, 1992)

Beams or girders are normally braced by to provide stiffening and in some cases the diaphragms may have been formed as part of the beam or girder when they were cast in place.

Loading of the simply supported beam or girder will cause bending and produce a compression stress in the top side and a corresponding tension in the bottom. The magnitude of the bending stress reaches its maximum near the center of the span and over the intermediate support of continuous spans. Maximum shearing stress is produced near the supports or points of bearing upon the substructure. Bearing points on cantilevered girders must be examined as a unit for the evidence of the shear at the support point and possible bending and compression stress.

The significant areas in reinforced concrete beams and girders are basically the center of the span, the bearing points, and flanges. Small vertical hairline cracks along the bottom of a reinforced concrete beam or girder due to bending moment usually will be many in number and spaced over some length. These hairline cracks are usual in reinforced concrete beams but not in prestressed concrete beams.

Surface cracks or spalling can permit moisture to enter. Rusting may result along the bars. These areas can become critical and therefore should be inspected carefully. The three most important deterioration conditions to look for during inspection of concrete beams and girders are scaling, spalling, and cracking.

Scaling usually indicates a slight reduction in cross section, and while not critical in itself, it may be significant, if it appears in the area of a support or bearing. Spalling, to a greater degree than scaling, causes reduction in cross section, particularly at the bearing points. Where spalling exposes reinforcing bars, corrosion or rust with a resulting loss of steel section is likely; therefore, the strength of the beam or girder may be reduced. The severity of this condition is of concern along the entire length of the beam, but most particularly at maximum flexure locations, as well as at the end bearing point.

The entire beam or girder should be inspected for cracks. Although hairline flexure cracks can be expected in a reinforced concrete beam, shear cracks, sometimes referred to as diagonal tension cracks, are not expected and should be investigated at length. The direction, size, and extent of cracks or cracking should be recorded and described completely.

The vertical and transverse alignment of the beam should be checked for abnormal deflection, particularly when live loads are on the bridge. Deflection usually indicates the start of beam failure in resisting normal design loads. It may

also indicate failure elsewhere in the bridge, which shifts or transfers more of the load on the bridge to the sagging beams, i.e., overloading due to failure elsewhere. The amount and location of the sag should be reported.

The vertical and longitudinal alignment of the beam should be checked for overturning, buckling, or canting. Canting indicates that the upper portion of beam, being in compression, has a tendency to buckle. The effect of this lateral movement produces a rotational movement in the beam called canting.

Figures 3.48-3.50 show several examples of concrete beams. The beam shown in Figure 3.48 is in fair condition. Several horizontal cracks are visible, as well as stains indicating that water may be getting to the reinforcement. The cracks or deterioration are not severe enough to lower the safe load capacity, and hence the fair rating. The deterioration, cracks, and spalling are significant in Figure 3.49, resulting in a poor rating. The reinforcement has been exposed to water and air, which will promote rapid rusting. The cracking and reinforcement deterioration would reduce the capacity rating, and call for a poor condition rating. The concrete and reinforcement deterioration is severe for the beam shown in Figure 3.50. The steel reinforcing has large section loss; therefore, the condition of this beam is rated critical and requires immediate attention in terms of temporary repairs, replacement or rehabilitation.

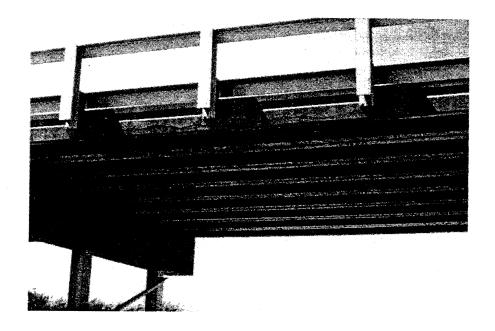


Figure 3.48 Concrete beam rated fair because of minor horizontal cracking (White, 1992)

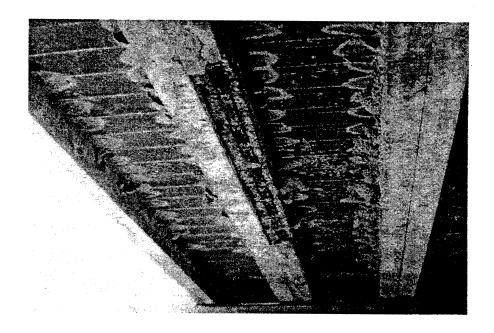


Figure 3.49 Cracking and spalling reduces condition rating poor (White, 1992)



Figure 3.50 Severe cracking, spalling, deteriorating of concrete and loss of steel section give beam a critical rating (White, 1992)

3.4.2 Improvement of the strength of reinforced concrete beam (Klaiber, 1987)

3.4.2.1 Addition of steel cover plate

One method of increasing flexural capacity of a reinforced concrete beam is to attach steel cover plates or other steel shapes to the beams tension face. The plates or shapes are normally attached by bolting, keying, or doweling to develop continuity between the old beam and new material. If the beam is also inadequate in shear, combinations of straps and cover plates may be added to improve both shear and flexural capacity. Because a large percentage of the load in most concrete structures is dead load, for cover plating to be most effective, the structure should be jacked prior to cover plating to relieve the member of dead-load stresses. The addition of steel cover plates may also require the addition of concrete to the compression face of the member.

A successful method of strengthening reinforced concrete beams involves the attachment of a steel channel to the stem of a beam (Figure 3.51). Test were performed on a section using steel channels and it was found to be an effective method of strengthening (Taylor, 1976). An advantage to this method is that rolled channels are available in a variety of sizes, require little additional preparation prior to attachment, and provide a ready formwork for the addition of grouting. The channels can also be easily reinforced with welded cover plates, if additional strength is required. Prefabricated channels are an effective substitute, when rolled sections of the required size are not available. The reinforced concrete beams are first prepared by removing dirt and other foreign material, exposing the coarse aggregate (grit biasing is recommended), and removing any final debris and dust with compressed air or vacuuming. Stirrups and longitudinal steel are then located and marked and bolt holes are drilled through the reinforced concrete beam. It should be noted that the bolts are placed above the longitudinal steel so that the stirrups can carry shear forces transmitted by the channels. If additional sheer capacity is required, external stirrups should also be installed (Cestilli-Guidi, 1983). It is also recommended that an epoxy resin grout be used between the bolts and concrete. The epoxy resin grout provides greater penetration in the bolt holes, thereby reducing slippage and improving the strength of the composite action.

Bolting may be an expensive and time consuming method, because holes usually have to be drilled through the old concrete. Bolting is effective, however, in providing composite action between the old and new material.

The placement of longitudinal reinforcement in combination with a concrete sleeve or concrete cover is another method for increasing the member's flexural capacity. This method is shown in Figure 3.52(a) and (b) and outlined in an article on strengthening by Westerberg,(1980). Warner (1981) presents a similar

method shown in Figure 3.52(c).

Developing a bond between the old and new material is critical for developing full continuity. Careful cleaning and preparation of the old concrete and the application of a suitable epoxy-resin primer prior to adding new concrete should provide adequate bonding. Stirrups should also be added to provide additional shear reinforcement and support the added longitudinal bars.

Design and analysis procedure

The design of steel cover plates for concrete members is dependent on the amount of continuity assumed to exist between the old and new material. If one assumes that full continuity can be achieved and that strains vary linearly throughout the depth of the beam, calculations are basically straightforward. Much of the load in concrete structures is dead load, and jacking of the deck during cover plating will greatly reduce the amount of new steel required. It should also be pointed out that additional steel could lead to an overreinforced section. This could be compensated for by additional concrete or reinforcing steel in the compression zone.

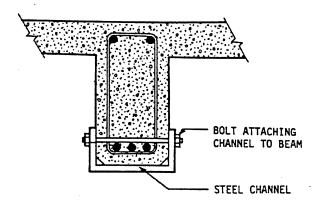


Figure 3.51 Addition of a steel channel to an existing reinforced concrete beam (Klaiber, 1987)

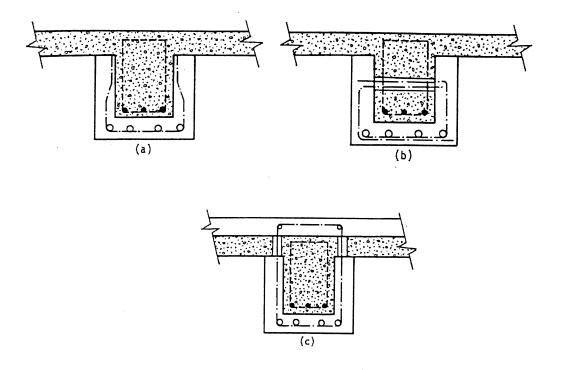


Figure 3.52 Techniques for increasing the flexural capacity of reinforced concrete beams with reinforced concrete sleeves (Klaiber, 1987)

3.4.2.2 Epoxy injection and rebar insertion

The Kansas Department of Transportation has developed and successfully used a method for repairing reinforced concrete girder bridges. The procedure used by the Kansas DOT on bridges with shear cracks in the main longitudinal girders not only prevented further shear cracking but also significantly increased the shear strength of the repaired girders.

The method involves locating and sealing all of the girder cracks with silicone rubber, marking the girder center line on the deck, locating the transverse deck reinforcement, vacuum drilling 45-deg holes that avoid the deck reinforcement, pumping the holes and cracks full of epoxy, and inserting reinforcing bars into the epoxy-filled holes. A typical detail is shown in Figure 3.53.

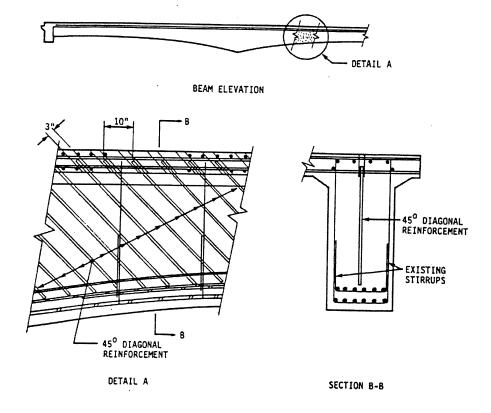


Figure 3.53 Kansas DOT shear strengthening procedure (Klaiber, 1987)

An advantage of using the epoxy repair and rebar insertion method is wide application to a variety of bridges. Although the Kansas DOT reported using this strengthening method on two girder, continuous, reinforced concrete bridges, this method can be a practical solution on most types of reinforced concrete girder bridges that require additional shear strength. In addition, the Kansas DOT was able to train and utilize its own maintenance forces in applying this method, and only minor traffic restrictions were noted during the construction phase.

However, the essential equipment requirements needed for this strengthening method may limit its usefulness. Prior to drilling, the transverse deck steel must be located. The drilling unit and vacuum pump required must be able to drill quickly straight holes to a controlled depth and keep the holes clean and free of dust. Failure to keep the holes clean may result in the following occurring:

- The epoxy pumped into the hole will not bond to the girder concrete.
- ii) the epoxy will not penetrate into small shear cracks (if they are present) because they will be filled with drilling dust.
- iii) if the dust is not extracted as drilled, the drill will likely be irrevocably locked into the hole. In addition, the epoxy injection pump must be of positive displacement and able to deliver a certified volume ratio of hardener to resin in the temperature and pressure range needed to perform the injection.

3.5 Repair of prestressed concrete bridge girders

The widespread use of precast, prestressed concrete girders in bridge construction began in the mid-1950's in Canada. Since then, they have been used extensively for bridges with spans up to 40 meters (130 feet). Historically, girder damage has been primarily a result of either high load impact or concrete deterioration and prestressing steel corrosion in areas of leakage between girders. The loss of prestressing in a bridge girder results in a reduction in structural capacity. If unattended, this typically necessitates traffic lane closures or load restrictions on a bridge, thus reducing the level of service for which that bridge was originally intended. A reduction in service ultimately results in inconvenience to the general public and the users of the transportation network. Repairs must be completed quickly, effectively and at a reasonable cost.

3.5.1 Longitudinal external post tensioning

Longitudinal external post tensioning is applied to damaged concrete girders. On either side of the damaged area, in the sound sections of the beam, symmetrical jacking corbels are built and anchored to the bottom flange. Post-tensioning tendons (270 K strands, Grade 160 rod) are passed through the corbels and anchored against bearing plates. After preloading the beam, the

concrete is repaired and allowed to harden; when strong enough, the preloading is removed and the exterior post-tensioning of the beam is applied, simultaneously at both corbels. To protect the bars or strands, they are placed in plastic conduits and pressure grouted.

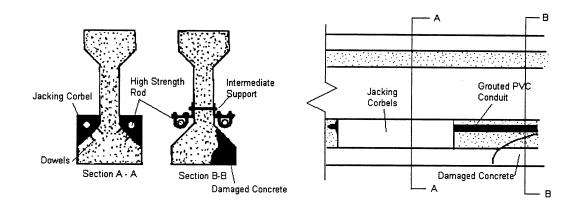


Figure 3.54 Longitudinal external post tensioning Prestressing Steel and Concrete Repair (Xanthakos, 1995)

Prestressed concrete girders that have suffered damage associated with severed or corroded prestressing strands require strength restoration. Strength lost may be regained by external posttensioning, internal splicing, and with the use of a metal sleeve splice.

Splices used for these repairs have the capacity to restore the strength of one to $ten \frac{1}{2}$ - in diameter severed strands. Generally, there are no restrictions with regard to the number of strands that can be spliced. Strands broken on one side of a girder only may give rise to a combination of torsional and transverse flexural stresses induced by the resulting eccentricity, and this condition requires a complex analysis.

A more practical way to assess these stresses is suggested by Shanafelt and

Horn (1985):

- i) Measure the lateral curvature (sweep) of the bottom flange. If this sweep is within the standard tolerance for prestressed girders, the lateral stresses may be ignored.
- ii) If the sweep exceeds the allowable deviation, calculate the torsional and flexural stresses induced and include these values in estimating the splice capacity.
- iii) Consider jacking the bottom flange into allowable alignment and hold in place with an additional diaphragm.
- iv) If neither procedure is practical, replace the girder.

Splice types described in this section are for standard AASHTO I-beams, but details can be modified for use with other beam sections. Nearly all splices extend above the bottom flange of the girder. Where intermediate diaphragms obstruct the installation, enough concrete can usually be removed or cored out from the diaphragm to allow passage of the splice. Any portion of the metal sleeve splice not used for strength can be slotted to fit around the diaphragm.

Concrete for cast-in-place splices should have a minimum strength $f_c = 5000$ lb/in², and should be well compacted in posttensioning jacking areas. The repair procedure for all types is essentially similar. Broken ends of strands should be recut and the new end should be used to compute the development length Broken strands should be tied in place. After removing loose concrete, epoxy grout should be applied to the damaged area, and the concrete repair should be completed as previously. Two of the splice types described in this section require a roughened interface of beams and corbels. These surfaces should be cleaned and roughened to a minimum depth of $\frac{1}{4}$ in, and all loose particles removed. Holes in webs and flanges should be located to miss strands and reinforcement bars.

3.5.1.1 Load capacity analysis

i) Posttensioning with splice type A. (Xanthakos, 1995)

Type A splice details are shown in Figures 3.38 and 3.39, and the use of two posttensioned 1-in diameter ASTM A722-75 smooth Grade 160 rods is illustrated to restore the loss of prestress due to four severed ½ -in 270ksi strands in AASHTO girder Type IV.

The splice requires 4-ft long jacking corbels located outside the damaged area. The corbels should be located so that hole can be drilled without interfering with harped or draped strands. The high-strength rods between corbels are protected by one ½ -in inside diameter rigid plastic conduit, which is pressure-grouted after posttensioning. The yield point of A722-75 bars corresponds to a 0.7 percent strain and 0.2 percent offset. The corbel length of 4 ft is determined using a minimum tie spacing of 7.5 in. The posttensioning is applies to the AASHTO beam in the preloaded condition.

Splice capacity analysis.

Given data:

Span length = 85 ft.

Girder spacing = 7 ft.

 $f_c' = 5000 \text{ lb/in}^2$ (prestressed concrete girder)

 $f''_c = 3500 \text{ lb/in}^2 \text{ (cast-in-place concrete)}$

Non-prestressed $E_c = 3.58 \times 10^6 \text{ lb/in}^2$

Prestressed $E_c = 4.29 \times 10^6 \text{ lb/in}^2$, n = 3.58/4.29 = 0.8

Section properties for the girder:

 $A = 789 \text{ in}^2$

 $I = 260,700 \text{ in}^4$

$$S_b = 10,540 \text{ in}^3$$

Section properties of girder and slab:

 $A = 1257 \text{ in}^2 \text{ (composite section)}$

 $I = 573,000 \text{ in}^4$

 $S_b = 15,570 \text{ in}^3$

The live load moment (one lane) of 1255 ft-kips is obtained directly from AASHTO tables. For a live-load distribution S/5.5 = 7/5.5 = 1.36, the live-load moment plus impact per girder is M_{LL+1} = 1058 ft-kips (HS 20 loading). The stress f_b (composite section) from live load is

$$f_b = \frac{1,058x10^3 x12}{15,570} = 815 \text{ lb/in}^2 \text{ (bottom fiber of girder)}$$

Assume four severed strands in the bottom layer (Figure 3.55). Using a prestress loss of $45,000 \text{ lb/in}^2$ (about 17 percent) and initial prestress of 70 percent of the strand capacity, the working prestress of the four severed strands is [0.7(270) - 45] (0.153)(4) = 88 kips. The prestress loss at the bottom fiber is computed as

$$f_{bl} = (P/A) + (M/S) = (88x10^3/789) + (88x10^3)(22.23)/10540 = 111 + 186 = 297 lb/in2,$$

The stress of 297 psi must be restored by the splice rods. This added prestress is due to two 1-in diameter rods Grade 160, stressed to a working load of 0.6 f_s = 0.6 x 160 = 96 kips/in².

Assuming that the conduit supports are at 10-ft centers, the bending stress is computed approximately as 5.5 kips/in². Hence, the working load of two rods is

 $(96 - 5.5) \times 10^{3} (0.785)(2) = 142 \text{ kips.}$ The prestress gain is computed as

 $f_b = (142 \times 10^3 / 1257) + (142 \times 10^3)(19.8) / 15,570 = 113 + 180 = 293 \text{ lb/in}^2 \approx 297 \text{ lb/in}^2 \text{ OK}$

Ultimate strength

The depth a of the equivalent compression stress block is $a = A_s f_{ps}/(0.85 f_c b)$

where $f_c' = 3500 \text{ lb/in}^2$, b = 90 in, and $A_s f_{ps}$ is now computed based on the remaining thirty effective strands and the two added rods.

• Strands: $A_s f_{ps} = (30)(0.153)(270)(0.85) = 1053 \text{ kips}$

• Rods: $A_s f_{ps} = (0.785 \text{x2})(160)(0.85) = 213 \text{ kips}$

Hence, a = (213 + 1053)/[0.85(3.5)(90)] = 4.7 in

The center of gravity of the thirty strands is 6.47 in above the bottom of the girder. The moment arm of the strands with respect to the concrete in compression is 60.5 - 6.47 - 2.35 = 51.68 in. Likewise, the moment arm of the rods is 60.5 - 17.00 - 2.35 = 41.15 in.

For strands: $M_u = 51.68 \times 1053/12 = 4538 \text{ ft-kips}$

For rods: $M_u = 213 \times 41.15/12 = 731 \text{ ft-kips}$

Total: $M_u = 4538 + 731 = 5269 \text{ ft-kips}$

Factored Moment = 1.3[DL + 1.67(LL + I)] = 1.3(1329 + 1.67x1058) = 4020 ft-kips.

The ultimate capacity available at the splice is adequate. The analysis should also consider the adequacy of bearing on the corbel, and the number and size of ties.

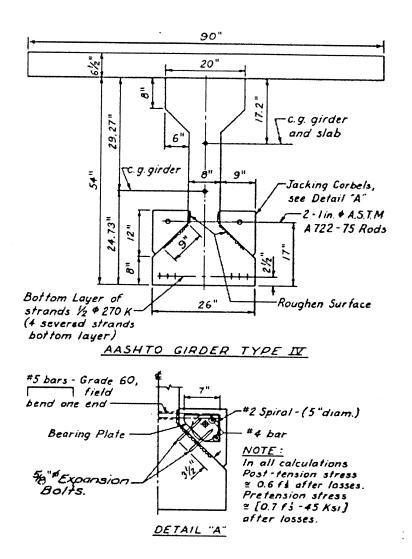


Figure 3.55 Type A splice detail (Xanthakos, 1995)

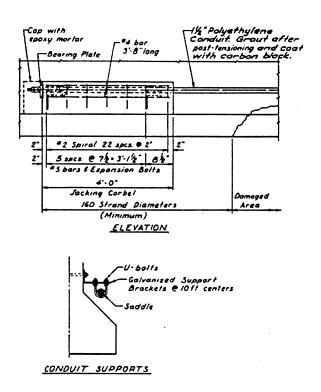


Figure 3.56 Corbel elevation and U bolts for posttensioning splice type A (Xanthakos, 1995)

ii) Posttensioning with splice type B.

Type B splice details are shown in Figures 3.57 and 3.58. This type involves the use of six posttensioned 1/2-in diameter 270ksi strands to restore the prestress loss by the fracture of three 1/2-in strands in an AASHTO girder Type III. The strands are posttensioned separately and protected from corrosion by a cast-in-place corbel that is continuous for the full splice length (Shanafelt and Horn, 1985). The construction procedure involves four steps:

- i) Apply preload if required.
- ii) Repair damaged concrete and preferably cast corbels at this stage.
- iii) Remove preload when concrete has attained the required strength.
- iv) Posttension the system.

Splice capacity analysis.

Given data:

Span length = 60 ft.

Girder spacing = 7 ft. 6 in.

Values of f_c ', f_c ", and E_c , are the same as in the example with splice type A.

· Section properties for the girder:

 $A = 559.5 \text{ in}^2$

I = 125,400 in⁴

 $S_b = 6190 \text{ in}^3$

· Section properties for girder and slab:

 $A = 1027 \text{ in}^2$

I = 326,000 in⁴

 $S_b = 9,895 \text{ in}^3$

Section properties for girder, slab, and corbels:

 $A = 1129 \text{ in}^2$

I = 361,500 in⁴

 $S_b = 11,550 \text{ in}^3$

From AASHTO tables, live-load moment (one lane) = 807 ft-kips (HS 20). Distribution factor = 7.5/5.5 = 1.36. Hence, $M_{LL+1} = 697$ ft-kips.

The stress f_b in the composite section at service load including impact (LL + I) is $f_b = 697x12/9895 = 845 \text{ lb/in}^2$. Three strands are assumed to be severed in the bottom layer. From the previous example, the working capacity of three strands is 22x3 = 66 kips, so that the prestress loss is now

$$f_{bi} = (66x10^3/559) + (66x10^3)(17.77)/6190 = 118 + 189 = 307 lb/in^2$$

which must be restored by the new posttensioned strands.

The weight of corbels is computed as 122lb/ft, and for an assumed corbel length of 30 ft, the resulting moment at center of the 60-ft span is 41.2 ft-kips. The bottom stress due to the weight of corbel is $(41)(12)/9895 = 50 lb/in^2$, which must be added to the stress loss of 307 lb/in².

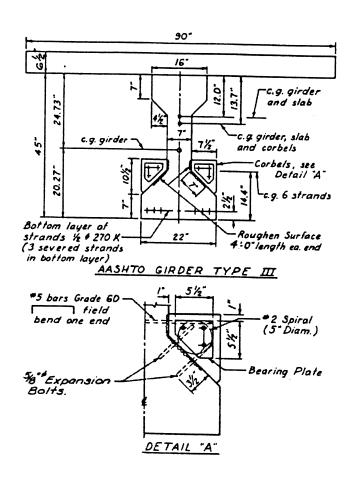


Figure 3.57 Type B splice detail

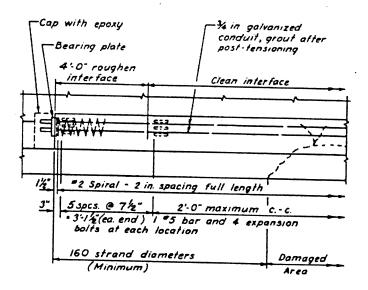


Figure 3.58 Corbel elevation and details for posttensioned splice type B

Assume now that six $\frac{1}{2}$ -in diameter 270 ksi strands are added. The working stress per strand after losses is $0.6f_s = 0.6x270 = 162 \text{ kips/in}^2$. The prestressing force in the six strands = 162x0.153x6 = 148 kips. The bottom stress resulting from this prestress is

$$f_b = (148 \times 10^3 / 1129) + (148 \times 10^3)(16.9) / 11550 = 131 + 217 = 348 \text{ lb/in}^2$$

The prestress gain of 348 lb/in² is nearly the same as the prestress loss (357 lb/in²), hence the splice is satisfactory. The splice capacity is also checked at the ultimate strength state and found to be adequate.

Corbel reinforcement.

From Figure 3.56, Detail "A", the area of the bearing plate is 24 in². The factored strength of three strands is

$$nP_n = 270(0.95)(3)(0.153) = 118 \text{ kips}$$

Working load per corbel = 148/2 = 74 kips. The bearing stress is

$$f_c = 74/24 = 3080 \text{ lb/in}^2 \text{ under plate} > f_{c(all)} = 3000 \text{ lb/in}^2$$

An overstress of 2.7 percent is acceptable with spiral containment. Area of corbel = 51in^2 . At ultimate load, the bearing on corbel is $118\times10^3/51 = 2300 \text{ lb/in}^2$, OK. The design of the splice is completed by checking the number and size of ties is required.

3.5.2 Metal sleeve splice

This is an external procedure for splicing a damaged beam as shown in Figure 3.56. It does not normally restore prestress, although preloading may restore partial or full prestress. This splice is often used to restore the beam to its design capacity, when there are many severed strands, or where a large quantity of concrete is missing. The plates are normally galvanized A-36 metals bonded to the concrete beam by injecting an epoxy grout into a 1/16-inch gap between the materials. Construction normally begins by applying the necessary preloading. Then the concrete is repaired. After the concrete has gained sufficient strength, the preloading is removed and the metal sleeve installed. Alternatively, the preloading may be left in place until after the metal sleeve has been placed and grouted. The latter procedure enhances the capacity of the splice by precompressing it under no live load.

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

^{*}Can be improved to excellent by extending corbels on fascia girder. N/A means not applicable.

Table 3.6 Criteria for Selecting Repair Methods, Prestressed Concrete Beams (Shanafelt and Horn, 1985).

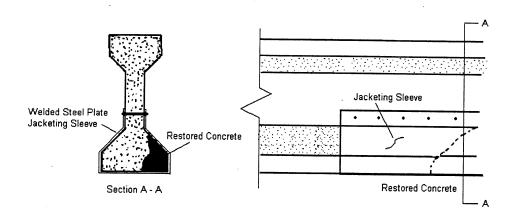


Figure 3.59 Metal sleeve splice

i) Posttensioning with metal sleeve splice

Details for a metal sleeve splice are shown in Figure 3.60. In this example, a metal sleeve is used to splice ten severed ½-in diameter 270ksi strands for an AASHTO girder Type IV. The loss of concrete can also be restored provided that only few strands are damaged. This splice can, therefore, restore strength and durability.

There is no upper limit regarding the maximum number of strands that can be spliced with this procedure (Shanafelt and Horn, 1985). The splice should, however, be extended upward to cover the entire web and most of the top flange. Both the plate thickness and the splice development length could be increased so that connection could be used to splice the entire portion of a damaged beam.

In this context, the splice is not used to restore lost prestress, but partial or full prestress may be regained by preloading. At the splice ends, full original prestress is restored. Intermediate hairline cracks are covered by the splice so that their presence becomes inconsequential. The minimum splice length may be determined by the bond length required to develop the broken strands. The bond

length shown in Figure 3.60 complies with AASHTO specifications and the strand is assumed to be bonded at the ends.

Bonding of the metal sleeve to the beam is provided by pressure injection of epoxy resin. The necessary clearance of 1/6 in. in between the sleeve and the beam is maintained with the use of 1/6 in. thick metal spacers at suitable intervals attached to the sleeve plate. The injection pressure is nominal, and for better bond the concrete interface should be completely clean and free of residual material. A minimum splice metal thickness of 5/16 in is recommended, and if bolts must be raised to avoid draped or harped strands, the top vertical plates and bolts may have to be redesigned. The plate material is galvanized A-36 metal. The inside surfaces of the plates must be scored vertically by wire brushing before assembling. The brushing should, however, be light so that while it scores the surface, it removes relatively little of the zinc coating. Zinc-rich paint should be applied to field welds. In addition, exterior girders should receive one or more coats of "concrete gray" for appearance. The plates may be either precut and welded, or cut in the field and field welded, the choice depending mainly on the accuracy of field measurements.

For a complete splice assembly, a field weld may be required at the top outside corner of the bottom flange.

The construction procedure involves the following steps:

- i) Apply preload when required.
- ii) Repair concrete damage.
- iii) When the concrete repair has gained sufficient strength, remove the preload.
- iv) Continue with the installation and splicing of the metal sleeve.

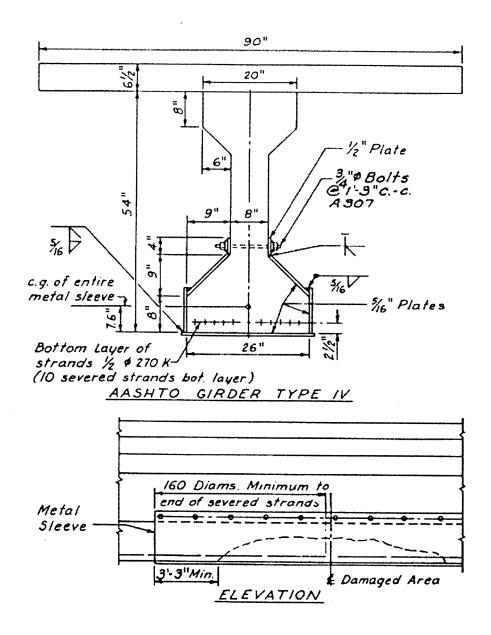


Figure 3.60 Metal sleeve splice

Splice capacity analysis.

Given data: (Figure 3.54)

Span length = 85 ft.

Girder spacing = 7ft. 6in.

Loading: HS 20

$$f_c = 4000 \text{ psi}$$

b = 90 in.

Inspection has revealed that ten strands in the bottom layer are severed. The working strength per strand = [0.7(270) - 45](0.153) = 22 kips/strand, and the nominal capacity is $270 \times 0.153 = 41.3$ kips/strand.

The strength of the metal sleeve may be calculated approximately as follows:

- Area = 26(5/16) + 2(8)(5/16) + 2(1)(5/16) = 13.7 in²
 Working strength = 13.7(20) = 274 kips
 Yield strength = 13.7(36) = 493 kips
- Number of broken strands whose strength can be replaced by the splice sleeve = 274/22 = 12.4, or 493/41.3 = 11.9, say 11 >10, OK.
- Bond on sleeve ultimate strength = $10(41.3x10^3)/[(26+16)(39)] = 252 \text{ lb/in}^2$ <350 lb/in² (allowable).
- Bond length of strands: According to AASHTO Article 9.27.1, the development length for strands (3- or 7-wire) should not be less than (f_{su} f_{se} D)

where D = nominal strength diameter (in) $f_{su}^{\dagger} = \text{average stress in prestressing steel at ultimate load}$ $f_{se} = \text{effective steel prestress after losses}$

or $I_d = [256 - (2/3)(144)](0.5) = 80$ in Use $I_d = 160$ diameters = $160 \times 0.5 = 80$ in

Approximate ultimate strength

The equivalent depth a of the compression zone is $a = A_s f_y / (0.85 f_c b)$. Given $f_c = 4000 \text{ lb/in}^2$, and b = 90 in. (Figure 3.58). Assume thirty-four strands, of which ten are severed.

The entire splice sleeve has a cross section, $A_s = 25.4 \text{ in}^2$;

$$A_{s.} f_y = 25.4x36 = 914 \text{ kips.}$$

Twenty-four strands have a total cross section, $A_s^* = 24(0.153) = 3.67 \text{ in}^2$;

$$A_s^*$$
. f_{ps} = 3.67(270)(0.85) = 842 kips

Then a = (914+842)/(0.85x4.0x90) = 5.7 in. (flange section) and a/2 = 2.85 in. Assume that the c.g. of thirty-four strands is 6 in above the bottom of the girder.

The c.g. of twenty-four strands is then computed as 7.46 in. above the bottom of the girder.

The ultimate moment capacity of the sleeve is

$$M_u(sleeve) = (914)(50.0)/12 = 3810 \text{ ft-kips}$$

Ultimate moment capacity of 24 strands,

$$M_u$$
 (strands) = 842(50.2)/12 = 3520 ft-kips

Total
$$M_u = 3810 + 3520 = 7330$$
 ft-kips

The unit weight of the girder plus slab is 822 + 650 = 1.472 kips/ft

For a span length of 85 ft, M_{DL} = 1329 ff-kips

For beam spacing 7 ft 6 in and HS 20 loading, $M_{LL+1} = 1058$ ft-kips.

Factored moment = 1.3[1329 + 1.67(1058)] = 4020 ft-kips < 7350 OK

ii) Posttensioning with internal strand splices.

Shanafelt and Horn (1985) describe two types of internal strand splice: single-strand, and 2-strand.

a) Single-strand splice.

Details for a single-strand splice are shown in Figure 3.61. Several strands in the same girder may be spliced in this manner. This splice restores lost strength internally, and combined with preloading, the procedure ensures that the beam will return to its original condition and capacity. Stressing is possible by applying a torque to the splice until the working strength of adjoining strands is reached. For ½-in diameter 270 ksi strands, this load is approximately 22 kips. The strand grip shown in Figure 3.61 may be used without modification. The transition from strand to rod is made with the use of two additional steel splices.

The construction procedure involves the following steps:

- i) Determine the preload requirements. Preferably the preload may be applied after the splice is stressed.
- ii) Assemble the splice, locating the splice sleeves and strand grips to allow seating of the strand grips and sufficient thread length in the splice sleeves.
- iii) Apply a torque to the splice sleeve to the working strength of the strand (22 kips). The strand grips should not be allowed to rotate.
- Step (ii) and (iii) are repeated for severed strands in the system.
- iv) Apply reload.
- v) Repair concrete, and after it has attained sufficient strength, remove preload.

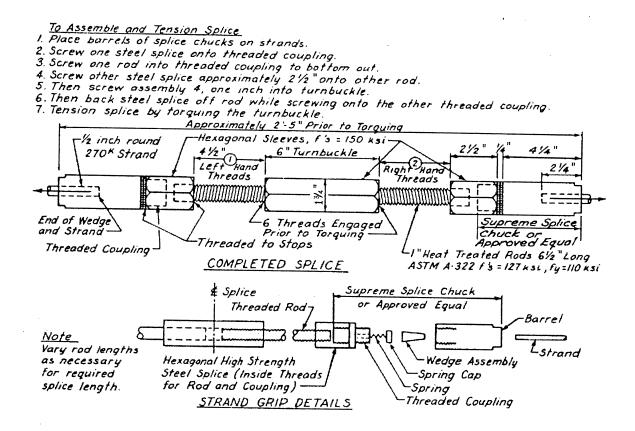


Figure 3.61 Single-strand internal splice (Xanthakos, 1995)

Example:

For a given prestressed concrete girder, the working strength per strand is 22 kips. The splicing rod is 1-in diameter threaded rod, Grade 150, $f_s = 150 \text{ kips/in}^2$.

Net area $A_{net} = 0.551$ in², and allowable $f_s = 0.6x150 = 90$ kips/in². At strand working load the stress in the rod, $f_s = 22/0.551 = 40$ kips/in² < 90, OK. Likewise, it is necessary to analyze the capacity at ultimate strength. At ultimate strength, $f_s = 41.3/0.551 = 74.25$ ksi < 150, OK.

b) Two strand splice.

Details for a 2-strand splice are shown in Figure 3.62, and illustrate the use of

one 1-in diameter high-strength bar to splice directly a pair of severed strands (1/2-in diameter, 270ksi). Several pairs of strands may be spliced in one girder, with an upper limit imposed by constructibility and access considerations. As in the previous splice, the procedure restores strength internally, and when combined with preloading, the beam is restored to the original condition. The strand spacing is taken as 2 in vertically and horizontally. The stressing is accomplished as before by applying a torque to the splice sleeve to approximately 44 kips, which is the working strength of two strands.

The construction procedure involves the following steps:

- i) Determine the preload requirements from a stress analysis. Preferably the preload may be applied after the splice is stressed.
- ii) Insert rod or bolt through transfer plates and connect sleeve.
- iii) Insert strands with swage fittings through transfer plates and connect strand splices (see also details in Figure 3.61).
- iv) Apply a torque to the lubricated splice sleeve to 44 kips (the working strength of two strands). The transfer plates should be prevented from rotating. Repeat the foregoing steps for all pairs of severed strands.
- v) Apply preload where necessary.
- vi) Repair the concrete, and after it has gained sufficient strength, remove the preload.

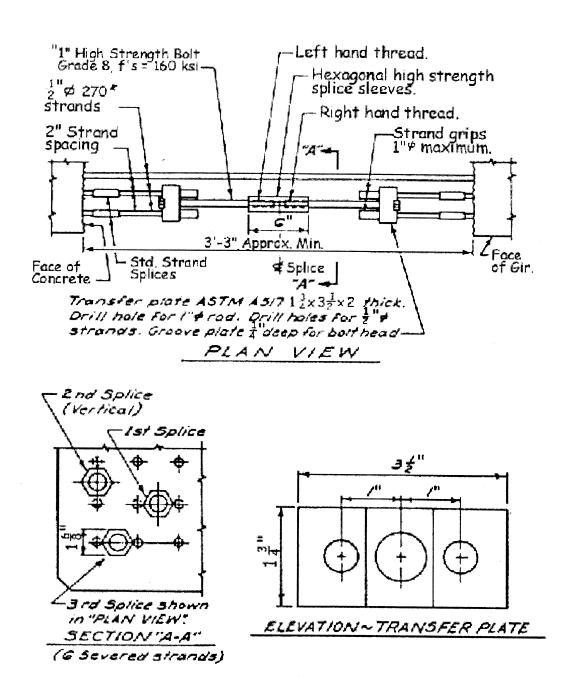


Figure 3.62 Two - strand internal splice detail

Example:

The following case is considered: For a given prestressed concrete girder, the working strength is 22 kips/strand. The splicing rods are 1-in diameter Grade

160, working stress = $0.6x160 = 96 \text{ kips/in}^2$, and $A_{\text{net}} = 0.551 \text{ in}^2$. The stress in the splice components (for two strands) is $22x2/0.551 = 79.9 \text{ kips/in}^2 < 96 \text{ kips/in}^2$. Likewise, the ultimate strength per strand is 270x0.153 = 41.3 kips/strand. At ultimate state the stress in the splice for two strands is $f_s = 41.3(2)/0.551 = 149.9 \text{ kips/in}^2 < 160 \text{ OK}$. The adequacy of the splice detail should also be ensured by checking the stresses in the transfer plate.

3.5.2.1 Preloading

Preloading in the form of a temporary load application during the repair process of damaged parts or concrete loss can restore and maintain the balance of stresses until the added concrete has gained its strength. Considerable damage or loss of concrete may occur in the bottom flange of a prestressed concrete girder while few, if any, of the prestressing elements are severed. In this case the original prestress is still applied but the reduced flange area in the bottom results in excessive compressive stresses. The application of a temporary preload induces tension in this area and balances the additional (undesirable) compression.

Correct dimensions and shape are essential for computing the section properties of the remaining cross section. If the computations show that the allowable stress is exceeded (making provisions for permissible overstress), preload may be applied while repairs are made to bring this stress within the allowable range. For a beam with a simple span, the maximum compressive stress occurs under dead load alone with the initial prestress.

Preload may be applied with a loaded vehicle or by vertical jacking. Procedures for providing preload by vertical jacking are shown in Figure 3.63. This arrangement requires fewer roadways, hence it offers an obvious advantage. If the preload is applied by a single jack, the design should check the temporary

stresses induced in other bridge elements. In this case, part of the jacking load is transferred through the slab and diaphragm to adjacent girders through the deck stiffness. If this distribution can result in excessive stresses, more than one jack should be used.

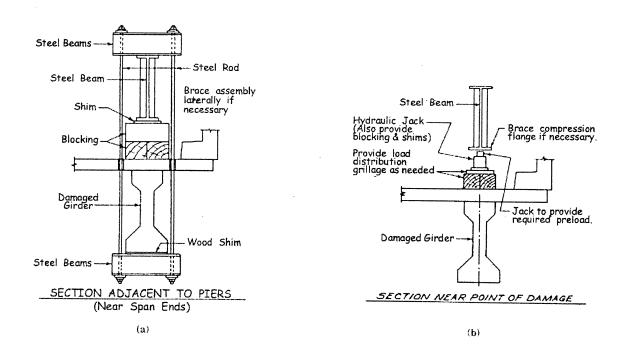


Figure 3.63 Preload by vertical jacking

A suggested way to determine the actual amount of preload transmitted to the damaged girder is by measuring flexural elongation with a strain gage. If the preload is applied with a loaded truck, the AASHTO distribution factors are applicable. Several states and the province of Ontario use preload to implement damage repairs.

Preload example (i)

The girder details are shown in Figure 3.64. Design data are as follows: $f'c = 5000 \text{ lb/in}^2 \text{ (girder)}$, $E_c(\text{slab}) = 0.8E_c(\text{girder})$.

Span length = 85 ft.

Girder spacing = 7 ft. 6 in.

Live load: Hs 20

- Girder section properties:

A = 789 in², I = 260,700 in⁴,
$$S_b = 10,540 in^3$$

- Section properties of girder and slab:

A = 1257 in², I = 573,000 in⁴,
$$S_b = 15,570 in^3$$

- Section properties of damaged girder:

A = 659 in², I = 184,000 in⁴,
$$S_b = 7,610 in^3$$

- Section properties of girder and slab, deducting damaged area:

A = 1127 in², I = 402,000 in⁴,
$$S_b = 11,230 in^3$$

The span length is 85 ft (simple span), and the girder spacing is 7 ft 6 in. The live load plus impact moment per girder is $M_{LL+1} = 1058$ ft-kips (HS -20).

The maximum stresses are computed for various loading conditions. The dead load due to curbs and other superimposed weights is not included in the sample calculations. Normally this superimposed load should be resisted by the composite slab-girder action.

Slab and damaged girder:

$$f_{b(LL+1)} = 1058 \times 12/11,230 = 1130 \text{ lb/in}^2 \text{ (tension)}$$

Dead load slab and girder:

DL w =
$$822 + 650 = 1,472$$
 lbs/ft MDL= 1329 ft-kips

Undamaged girder:

$$f_{b(DL)} = 1329 \text{ x } 12/10,540 = 1513 \text{ lb/in}^2 \text{ (tension)}$$

Prestress acting on undamaged girder (working strength):

$$f_{b(prestress)} = 34(22)789 + 34x22(18.73)/10,540 = 948 + 1330$$

= 2278 lb/in² (compression)

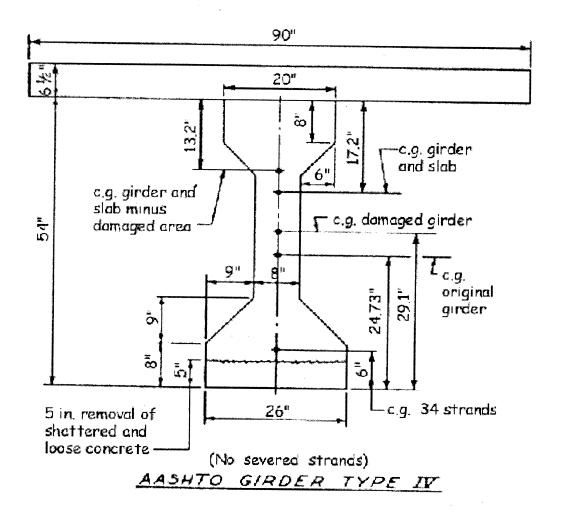


Figure 3.64 Girder detail for preload example (i)

Girder and slab (composite):

 $f_{b(LL+1)} = 1058 \times 12/15,570 = 815 \text{ lb/in}^2 \text{ (tension)}$

Dead load plus live load plus impact plus prestress (undamaged girder):

 $f_b = 1513 + 815 - 2278 = 50 \text{ lb/in}^2 \text{ (tension)}$

There is some uncertainty regarding stress redistribution upward through the roadway slab, which may or may not occur depending on the extent of damaged area and the number of exposed strands. To account for this uncertainty, the following alternate calculations are recommended, assuming (i) the prestress

plus the dead load (weight of slab and girder) act on the damaged girder cross section alone, and (ii) the prestress plus dead load (slab and girder) act on the composite damaged girder and slab.

Under assumption (i):

$$f_{b(prestress)} = 748/659 + 748(23.1)/7610 = 1135 + 2270 = 3405 \text{ lb/in}^2 \text{ (compression)}$$
 $f_{b(DL)} = 1329 \times 12/7610 = 2096 \text{ lb/in}^2 \text{ (tension)}$
Net $f_b = 3405 - 2096 = 1309 \text{ lb/in}^2 \text{ (compression)}$

Under assumption (ii):

$$f_{b(prestress)} = 748/1127 + 748(34.8)/11,230 = 664 + 2320$$

= 2984 lb/in² (compression)

$$f_{b(DL)} = 1329(12)/11,230 = 1420 \text{ lb/in}^2 \text{ (tension)}$$

Net
$$f_b = 2984 - 1420 = 1564 \text{ lb/ in}^2 \text{ (compression)}$$

Allowable
$$f_c = 0.4 f_c' = 0.4(5,000) = 2,000 \text{ lb/in}^2$$
.

Under either assumption the stresses are within the allowable limits.

The amount of prestress that could be restored by preloading is computed next. Under assumption (i), $f_b = 1309 \text{ lb/in}^2 = (\text{dead load plus prestress})$, bottom of damaged section (compression). Also $f_b = 1130 \text{ lb/in}^2$ (live load plus impact), bottom of composite damaged section (tension).

The difference is 1309 - 1130 = 179 lb/in² (compression). From the foregoing, it follows that a preload moment equal to the full live load plus impact moment can be applied to restore the original prestress effect at the bottom of the patched area.

Using the expression, M= PL/4 for a load P applied at midspan,

$$P=4 M_{LL+I}/L = 4(1058)/85 = 49.8 \text{ kips}$$

Preload example (ii)

The girder details are shown in Figure 3.65.

Design data are as follows: $f_c' = 5,000 \text{ lb/in}^2$ (girder), E_c (slab) = 0.8 E_c (girder). Girder section properties, span length, girder spacing, and loading same as in Preload Example (i). Also , $M_{LL+l} = 1058 \text{ ft-kips}$, $M_{DL} = 1329 \text{ ft-kips}$. It is assumed that four strands in the bottom layer are severed, and two 1-inch diameter posttensioned rods are used. The example illustrates whether a preload combined with the added posttensioning can reduce the live load plus impact tensile stress to 50 lb/in², which is the value in the original girder design.

From the added posttensioning
$$f_c = 142/1257 + 142(19.8)/15,570$$

= 293 lb/in² (compression)

From live load plus impact,
$$f_t = \frac{1058x12x1000}{15570} = 815 \text{ lb/in}^2$$

Net stress = 815 - 293 = 522 lb/in² (tension)
 If the allowable tension is
$$6\sqrt{f_c}$$
 = 424 psi,
 the excess tension = 522 - 424 = 98 lb/in²

 Assume that the prestress plus dead load of girder and slab are resisted by the damaged girder section alone.

$$f_{b(prestress)} = 30(22)/659 + 660(22.63)/7610 = 1000 + 1960$$

= 2960 lb/in² (compression)
 $f_{b(DL)} = 1329(12)/7610 = 2100$ lb/in² (tension)
Net stress = 2960 - 2100 = 860 lb/in² (compression)

 Assume that prestress plus dead load of girder and slab act on the composite damaged girder and slab. $f_{b(prestress)} = 30(22)/1127 + 660(34.33)/11,230 = 586 + 2018 = 2604 lb/in^2$ (compression)

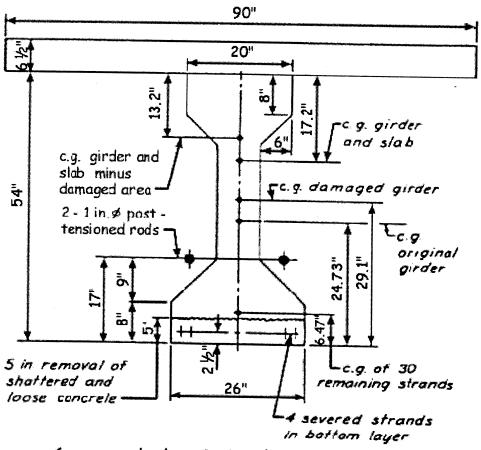
 $f_{b(DL)} = 1329(12)/11,230 = 1420 \text{ lb/in}^2 \text{ (tension)}$

Net stress = $2604 - 1420 = 1184 \text{ lb/in}^2$

Additional stress necessary to reduce the live load and impact tensile stress to $50 \text{ lb/in}^2 = 522 - 50 = 472 \text{ lb/in}^2$.

Moment =
$$0.472(15,570)/12 = 612$$
 ft-kips = $\frac{Px85}{4}$

Midspan load, P = 28.9 kips



4 severed strands in the bottom layer
Splice No.1 being used.

AASHTO GIRDER TYPE IV

Figure 3.65 Girder details

3.5.2.2 Load capacity analysis (Overload) (Xanthakos, 1995)

An analysis may be necessary to show that the repair girders have adequate capacity at service loads and the ultimate state. Small differences may be acceptable, if they are within the overload capacity provisions. The method of analysis is illustrated, which is in compliance with AASHTO specifications and standards.

Overload capacity

The overload capacity of repaired members should not be less than the allowable overload capacity of the bridge. In this context, extra legal loads that operate under permits in certain states are not considered in the analysis.

In assessing the overload capacity, it may be expedient to clearly designate the location of a damaged member. For example, if the member is located behind a barrier curb or under a sidewalk, it may not receive the same live load as an interior girder. Alternatively, the analysis may resort to a consideration of a modified transverse stiffness, to obtain a more favorable load distribution. However, assigning additional overload capacity to the repaired girder above the capacity of adjacent girders may not be normally necessary.

Figure 3.46 shows the overload permitted by the state of California. Bridges are designed for the HS 20 loading and a loading system that has axle weights approximately 1.5 times the HS 20 loading. The overload moments used in this illustration are based on the California criteria, and shown in Figure 3.66. The live load distribution is as per AASHTO standards.

The initial design data are as follows:

Precast concrete $f_c = 5000 \text{ lb/in}^2$

Cast-in-place concrete f_c " = 4000 lb/in²

Girder spacing = 7 ft 6 in

Distribution factor = 7.5/5.5 = 1.36; Impact = 27%

The live load plus impact per girder is computed from Figure 3.48 as M_{LL+l} = 1120 ft-kips (overload).

Overload stresses in original girder

From available data, the following are computed.

Dead load, weight of girder = 583 lb/ft

Dead load, weight of slab = 650 lb/ft

Total DL = 1233 lb/ft

$$M_{DL} = \frac{1.233x60^2}{8} = 555 \text{ ft-kips}$$

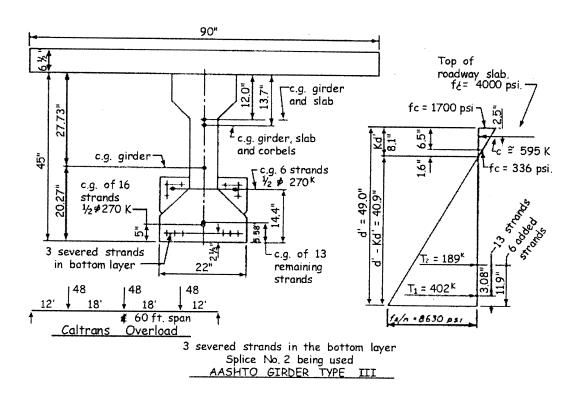


Figure 3.66 Overload capacity example

Likewise, for HS 20 loading, M_{LL+1} = 697 ft-kips.

At service loads the bottom stress is

$$f_b = 1076 + 845 = 1921$$
 psi (tension)

Prestressing force = 16, 22 kips/strand = 352 kips.

$$f_b = \frac{352}{559} + \frac{(352)(1527)}{6190} = 1498 \text{ psi (compression)}$$

Net f_b = 1921 - 1498 = 423 psi (tension)

For overload,
$$f_b = \frac{1120x12}{9895} = 1358 \text{ psi (tension)}$$

Net
$$f_b = 1076 + 1358 - 1498 = 936 \text{ psi (tension)}$$

Allowable cracking stress = 7.5 $\sqrt{f_c}$ = 530 psi

The stress due to CalTrans overload exceeds the allowable by 77 percent.

Overload stresses for repaired girders.

Assume that loss of prestress may be neglected. Stress in the bottom due to overload moment is given by

$$f_b = \frac{1120x12}{11550} = 1160 \text{ psi (tension)}$$

Prestressing force = 13, 22 kips/strand, = 286 kips.

$$f_b = \frac{286}{559} + \frac{(286)(14.7)}{6190} = 1190 \text{ psi (compression)}$$

Stress in the bottom due to added posttensioning (6 strands)

$$f_b = \frac{6x24.7}{1129} + \frac{6(24.7)(16)}{11550} = 350 \text{ psi (compression)}$$

From dead load, $f_b = 1076$ psi (tension)

Moment due to weight of corbels = 41.2 ft-kips

$$f_b = 41.2 \times 12/9895 = 50 \text{ psi (tension)}$$

Net stress for repaired girders

$$f_b = -1190 - 350 + 1076 + 50 + 1160 = 746 \text{ psi (tension)}$$

The stress due to overload exceeds the allowable by 41 percent.

Next, it is assumed that 5 inches of shattered and loose concrete is removed from the bottom flange. If preloading is not used, the only prestress at the bottom of the girder in the damaged area will be provided by the six posttensioned strands.

Added posttensioning

 $f_b = 350 \text{ psi (compression)}$

Overload

 $f_b = 1160 \text{ psi (tension)}$

Net (Overload) $f_b = -350 + 1160 = 810 \text{ psi (tension)}$

Therefore, the stress due to overload exceeds the allowable by 810/530 = 53 percent.

Capacity at ultimate state

The ratio of prestressing steel, $\rho_{\rm p} = \frac{A_{\rm ps}}{bd_{\rm p}}$ is computed as:

$$\rho_p = 16(0.153)/90(46.5) = 0.000585$$

The average stress in prestressing steel at ultimate load is determined from:

$$f_{ps} = f_{pu} \left[1 - \frac{0.5 \rho_p fpu}{f_c} \right]$$

where:

 f_c ' = compressive strength of cast-in-place concrete (28 days)

 $f_{ps} = 270 [1.0 - (0.5)(0.000585)(270/4.0] = 265 \text{ kips/in}^2$

 $M_u = \phi M_n$, and for $\phi = 0.9$

The flexural capacity for rectangular sections or flanged sections where the

neutral axis lies in the flange is given by:

$$\phi M_n = M_u = A_{ps} f_{ps} d_p \left[1 - \frac{0.5 \rho_p fpu}{f_c'} \right]$$

 M_u = 0.9x2.45x265x46.5x [1.0 - (0.6x0.000585x265/4.0)]/12 = 2214 ft-kips

which is the ultimate strength of the original girder

Likewise, the capacity of the repaired girder at ultimate state is computed as follows:

$$\rho_p = 19(0.153)/90(43.2) = 0.000747$$
 $f_{ps} = 270[1.0 - 0.5(0.000747x270)/4.0]$

 $= 263 \text{ kips/in}^2$

 M_u = 0.9x2.91x263x43.2x[I - (0.6x0.000747x263/4.0)]/12 = 2403 ft-kips The required moment is the sum of the factored moments, or M_u = 1.3(555 + 41.2 + 1.67 x 697) = 2285 ft-kips, 2285/2214 = 1.03, say OK

AASHTO Criteria (Manual for Maintenance Inspection)

The 1983 AASHTO specifications contain provisions for rating of prestressed concrete beams for allowable stress method. For this example, the rating will be checked referring to the 1978 AASHTO specifications.

The maximum allowable operating moment is

$$M_{op} = 1.3(M_{DL} + M_{LL+1}) = 1.3(555 + 41.2 + 697) = 1680 \text{ ft-kips}$$

Using CalTrans criteria, the overload moment is

$$M_{op}$$
 = 555 + 41.2 + 1120 = 1716 ft-kips

which gives an overstressing above the allowable by 1716/1680 = 2 percent

3.5.2.3 Fatigue considerations (Xanthakos, 1995)

Tests show that prestressing strands in cracked prestressed concrete girders have a lower fatigue life than bare strands. When the fatigue loading is of sufficient magnitude to cause a flexural crack in the girder, stress concentrations appear on the prestressing elements. The result is higher stress ranges, hence a lower fatigue life. Girders subjected to accidental damage and cracking may have a reduced fatigue life. It follows, therefore, that the fatigue life of damaged prestressed concrete girders repaired in place deserves consideration.

Shanafelt and Horn (1985) report results of fatigue tests of prestressing bare strands subjected to a maximum stress range of 0.10 times the ultimate strength; the strands have a fatigue life of about 2 million cycles. Almost all grade separation structures will fall in this category.

Fatigue life of prestressed concrete beams

Prestressing elements in girders not subjected to cracking loads have the same fatigue life as bare strands subjected to the same stresses. There are no known fatigue tests on beams with fractured concrete because of overweight vehicle impact. However there has been fatigue tests on girders cracked by test loads (Bazant, 1985; James, Zimmerman, and McCreary, 1987). In one series carried out to determine the effect of repetitive loading on girders with draped and blanketed strands, cracks occurred under service load tensile stresses of $6\sqrt{f_c}$ for a corresponding minimum fatigue life of 3 million cycles. The minimum fatigue life for loads that produced zero tension in the concrete was 5,000,000 cycles. In another series of tests on prestressed girders, results showed that girder loading causing flexural cracks to open should not necessarily reduce fatigue life, provided the stresses in the strand reinforcement are smaller than the fatigue limit.

Manson, et al. (1978) have carried out fatigue tests to assess the fatigue life of prestressed concrete I -beams which have been overloaded to cause flexural and inclined cracking prior to repeated loading. Beams were subjected to an initial static loading of approximately 80 percent of the ultimate flexural strength, sufficient to cause significant inclined cracking in both shear spans. The beams were next subjected to repeated load cycles ranging from 19 to 45 percent of the ultimate flexural capacity of the specimen. Each beam sustained 2,000,000 cycles of loading. The equivalent tensile stress in the concrete was $6\sqrt{f_c}$, producing a flexural crack width less than 0.010 in. The cyclic loading was then stopped. Subsequent cyclic loading were applied at 48 to 54 percent of the ultimate flexural capacity, resulting in flexural fatigue failures at 458,000 to 1,200,000 cycles.

Concrete decks

Progressive overload-induced damage to concrete decks has been documented in the field. Interaction between physical damage directly attributable to wheel loads and other damage mechanisms (such as corrosion) have been confirmed through long-term monitoring and test programs. Deck damage may appear in several forms, but the most important damage mechanism is associated with transverse and longitudinal cracking. Wheel-loadrelated cracking is more severe in structures with a lower ratio of dead to total load, which is typically the case in concrete slabs on beams where the live-load moment is several times the dead-load moment. Reinforced concrete deck slabs on steel I -beams are more susceptible to this damage than decks on prestressed girders because of the inherent greater flexibility of the steel beams. Over- weight vehicle effects, particularly on deck cracking, are manifested in conjunction with other ongoing damage mechanisms. Corrosion reinforcement is promoted by the presence of cracks, and spalling resulting from steel bar corrosion is certainly accelerated by traffic.

James et al. (1987) conclude that cracking may occur at tensile stresses below the expected value of $6\sqrt{f_c}$ or $7.5\sqrt{f_c}$ but whether this is caused by residual stresses due to creep or shrinkage is uncertain. Other investigators have reported cracking at stresses close to $2.7\sqrt{f_c}$ to $4.5\sqrt{f_c}$.

Concrete bridge decks appear to respond to overloading by increased longitudinal and transverse cracking, although cracking may also result from normal traffic levels. The increased cracking accelerates corrosion attack and spalling or scaling. Mechanisms are available to articulate the interaction between mechanical and physical effects in progressive overload-induced damage, but specific procedures for studying the rate of deterioration are yet to be developed.

Progressive damage of cracked prestressed concrete girders can be detected and assessed as long as the girder is subjected to moments large enough to reopen the initial cracks.

Suggested guidelines

With regard to fatigue, Shanafelt and Horn (1985) recommend the following:

- i) if the original girders were designed for tension under working stresses, the repaired girders must be able to withstand the same tensile stresses with the same factor of safety.
- ii) The maximum tensile stress in the concrete under working load should not exceed $6\sqrt{f_c^{'}}$
 - iii) The maximum allowable prestressing steel stress caused by live load

plus impact should be limited to 10 kips/in² for 7-wire strands grades 240k to 270k.

iv) The maximum tensile stress in the prestressing elements under working loads and after losses should be limited to 0.6 f's.

The investigators make no recommendations regarding the fatigue life of accidentally cracked girders prestressed with high-strength bars or rods. Based on test data, the conclusion is that damage that does not cause cracking completely through the precompressed area (containing high-strength rods) should not reduce fatigue life.

With regard to durability, present practice appears to accept the fact that epoxy mortars and special concrete mixes available today meet the strength requirements of original materials. Useful guidelines that may ensure an acceptable level of durability are as follows:

- i). All unsound concrete should be removed, and surface preparation should be complete and thorough.
- ii) Application of epoxy bonding, epoxy grout, and epoxy injection materials should follow fully tested and approved procedures and particular requirements about ambient temperatures should be observed.
- iii) Additional reinforcement to bond new materials to existing surfaces should be considered and, if warranted, incorporated in the repair program.
- iv) Where necessary, preloading should be used to ensure that the repair section would not be subjected to greater tensile stress under live load than the original section.
- v) The repaired areas should be sealed with a proven water sealant.
- vi) Preventive maintenance following a completed repair program can extend durability.

With regard to repair methods, the selection should be based on an engineering

analysis considering cost, safety, and aesthetics. Useful criteria are summarized in Table 3.6 and articulate the differences between repair methods to be used to restore serious damage. Serious damage is defined as damage, requiring splicing of strands accompanied by major loss of concrete.

3.5.2.4 Case study

i) Clint Moore Bridge, Boca Raton, Florida

This bridge is located in Boca Raton, Florida and carries north bound Florida Turnpike (SR 91) traffic over Clint Moore Road. This three span prestressed concrete I beam bridge was built in 1963. The bridge was damaged by an over height vehicle and subsequently repaired in 1996. The damage to the girder included concrete section loss and severed prestressing strands. The repair consisted of restoration of the concrete with polymer-modified grout and the placement of a 3/4-inch steel plate sleeve that was bonded to the restored section. This repair has been in place for 2 years with a high Turnpike ADTT. The most recent inspection report (12/97) states that, "The beams with impact spalls have been repaired satisfactorily." A visual inspection performed on the repair on 4/15/98 showed no evidence of deterioration. Figures 3.67 and 3.68 show the metal sleeve repair.



Figure 3.67 Metal sleeve splice repair - Clint Moore Bridge, Florida



Figure 3.68 Metal sleeve splice repair - Clint Moore Bridge, Florida

ii) Hospital Street/Highway 63 overpass, Fort McMurray, Alberta, Canada (Lanyi, 1993)

This structure is a typical bridge over the main highway through the northern Alberta city of Fort McMurray. Each span is simply supported and consists of 16 precast prestressed concrete channel girders, which have been transversely post tensioned. The superstructure was hit by an overheight backhoe

mounted on a flat bed trailer. Nine of the sixteen girders were damaged; however, only the exterior leg of the exterior girder had damaged prestressed strands. Six of nine horizontal strands were severed between the third point and midspan. None of the deflected strands were damaged. The ends of the girders received little to no damage. The exterior face of the damaged girder had a pigmented sealer coating. A three-man crew completed the repairs to the exterior girder over a three-week period. It took approximately 10 working days to prepare the damaged area, restress the damaged prestressing strands and repour the girder leg concrete. The cost to complete this portion of the work, including engineering and detours, was approximately \$ 50,000 or \$ 8,300 per strand. Due to the remote location of the construction site, mobilization costs were significantly higher than expected for central locations. Replacement of the girder was estimated at over \$ 200,000 and would have taken between two or three months completion time.

iii) Ross Street eastbound/Waskasoo Creek Bridge Red Deer, Alberta, Canada (Lanyi, 1993)

This city of Red Deer is located in central Alberta. Eastbound Ross Street carries three lanes of traffic across the Waskasoo Creek. This structure is a single span channel crossing consisting of eight precast, prestressed concrete channel girders. The entire structure is superelevated towards the outside of the bridge. Originally, there was a 50 mm (2") asphalt wearing surface directly over the precast girders. Approximately 10 years after construction, the asphalt-wearing surface was removed and replaced with a 50 mm (2") high density concrete overlay. Over the past 25 years, moisture has been leaking through the longitudinal joint between the overlay and the exterior sidewalk. Moisture has penetrated through the grout pocket between the first two exterior girders and accumulated along both legs of the adjacent girders.

Routine maintenance inspection had identified evidence of prestressing strand corrosion in a few local areas. When a test area was opened up at the bottom of the adjacent legs of adjacent girders, it was found that extensive corrosion of the bottom row of prestressing existed for almost the entire length of both girders. The ends of both girders were inspected closely and tested in order to confirm that field prestressing strand forces could be fully developed in these regions.

Construction access was excellent above and below the girders. The eastbound sidewalk and outside eastbound lane along Ross Street were barricaded. The contractor fenced off an adequate staging area below and erected his scaffolding across the Waskasoo Creek. Due to the proximity of the water to the underside of the structure, overhead clearance was tight.

It was originally planned to restress the four bottom strands in each leg. Conditions in the exterior girder leg were worse than anticipated, necessitating the restressing of two additional strands in the second row. All strands, which required restressing, were straight and each had a significant amount of section loss due to corrosion. Preloading consisted of a bin filled with pit run gravel and placed on the sidewalk at midspan.

A three-man crew completed the repairs to both the girders over a three-week period. The work included preparing the damaged areas, restressing the damaged prestressing strands and repouring the girder leg concrete. The cost to complete this portion of the work, including engineering and detours, was approximately \$ 50,000, or \$ 5,000 per strand. Replacement of the girder was estimated at over \$ 200,000 and would have taken between two to three months completion time.

Field prestressing of damaged strands in precast concrete bridge girders is a relatively quick and cost effective means of restoring the structural integrity of damaged members. The technique developed by Alberta Transportation and Utilities has proven this to be the case. This technique has successfully been used to repair girders damaged by high load impact and corrosion of the prestressing reinforcement.

The work generally requires a relatively small work force in the field and incorporates standard materials and easy to follow procedures. Costs to complete the work generally range between \$ 5,000 and \$ 10,000 per strand, depending on the extent of the damage, construction access, detour, mobilization and finishing requirements. Prequalification of contractors and tendering on a invitational basis are strongly recommended practices.

3.5.3 Internal prestress strand/field prestress splicing (Lanyi, 1993)

Historically, precast prestressed concrete girder repair options included either girder replacement or external post tensioning. Girder replacement is relatively expensive, time consuming and can be physically difficult to complete. External post tensioning is generally more cost effective than replacement, quicker and easier to construct. As an alternate method of repair to either of the above, Alberta Transportation and Utilities, Bridge Engineering Branch, has developed a technique for field prestress splicing of new sections of strand in an existing damaged girder. This technique is typically less expensive, quicker and easier to construct than other alternatives.

3.5.3.1 Repair procedure

The basic concept of field prestress strand splicing involves removing a damaged length of strand within the overall length of a girder, coupling a new piece of strand in its place and restressing the spliced strand with the use of a turn buckle device known as a "Splice Nut Assembly". This procedure is shown

in Figures 3.69 through 3.72. Applied stress to the new strand is recorded as a measure of its elongation under load. This stressing device does not have any losses due to anchor set. On short stressing lengths, anchor set can preclude the use of normal stressing jacks.

There are limitations concerning the application of this repair technique. In order to develop the required force in the new section of strand, the concrete around the remaining end sections of the original strands must be able to develop the same force. In other words, adequate bond between the concrete and the original portions of strand, at the ends, must exist in order to prevent those portions of strand from pulling out during stressing operations.

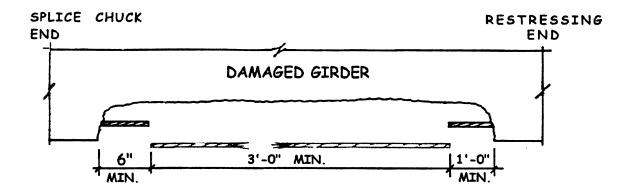


Figure 3.69 Removing a damaged length of strand (Lanyi, 1993)

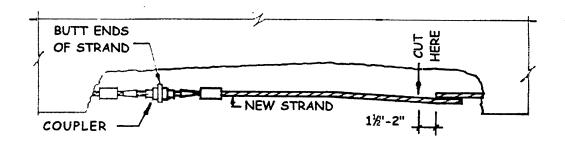


Figure 3.70 Coupling of new piece of strand (Lanyi, 1993)

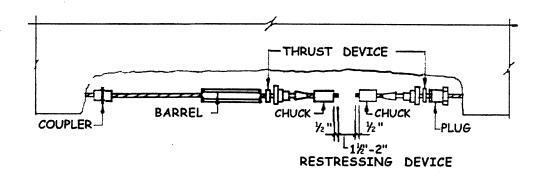


Figure 3.71 Restressing the spliced Strand (Lanyi, 1993)

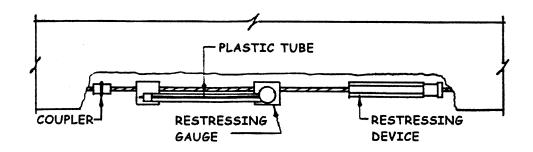


Figure 3.72 Splice nut assembly (Lanyi, 1993)

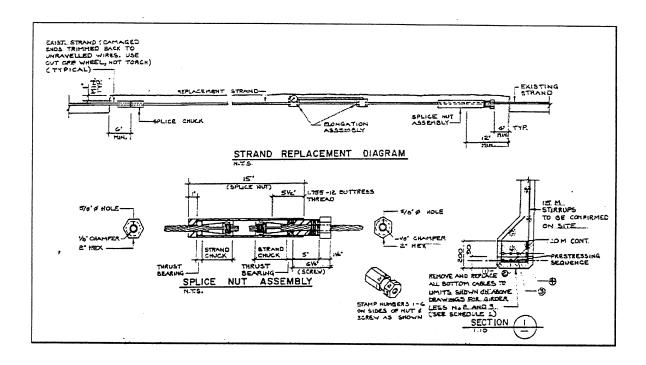


Figure 3.73 Stressing hardware and layout (Lanyi, 1993)

3.5.3.2 Stressing

The sequence of prestressing strand splice activities is illustrated in Figures 3.69 to 3.72. Stressing hardware and layout is illustrated in Figure 3.73. Also shown in Figure 3.73 is a typical section through the web of a damaged precast channel girder. This system is unique in that it makes use of standard prestressing strand coupling hardware, which can be easily obtained. The only item requiring fabrication is the splice nut assembly. With this assembly, simply turning the nut into the thread cylinder completes stressing.

Installation of the hardware is completed in the following sequence:

- Coupling of the dead end of the new strand to one of the original portions of strand, with a standard strand coupling device and standard chucks;
- Cut new strand to leave a gap of 50 mm (2") between the live end of the new strand and the end of the original strand on the opposite side;

- Coupling of the live end of the new strand to the cylinder portion of the splice nut assembly with standard chucks;
- Coupling of the other portion of the original strand to the nut portion of the splice nut assembly, with standard chucks;
- Attaching of the dial elongation gage to the new strand, and
- Fitting of the nut into the cylinder portion of the splice nut assembly.

3.5.3.3 Concrete repair and finishing

After all the damaged strands have been spliced, the exposed surfaces of the damaged concrete are prepared prior to the concrete pour. Preparation includes either a "light" sandblast or needle scaling in order to remove loose concrete flakes left behind from the demolition work. In the case, where there are corrosion materials present, they are to be removed from the surfaces of the concrete and reinforcing.

The girder is then formed to its original shape. Expansion anchors attached to the girder typically support forms. The engineer should be informed of the proposed locations of these anchors prior to installation in order to ensure they do not interfere with the location of other prestressing reinforcement. Concrete is usually specified as the repair material. Compatibility of materials is important to ensure a durable repair.

After forms are stripped, there are typically imperfections along the joint between old and new concrete, especially along the top of the repair. It is common practice to epoxy inject these areas and then finish the larger imperfections with a dry pack grout. Once this has been completed, the preload can be removed, as long as the required compressive strength has been obtained. The entire exterior face of the girder can be sealed with a pigmented sealer in order to provide a uniform finish.

3.6 Bearings

Regular bridge bearing maintenance should be directed towards keeping the bearings clean and protecting them from water salt, and debris. Cleaning methods depend on the type of bearing. If compressed air is available, it can be used to blow debris away from the bearings. Hoses and water are often used. A small garden hose is a good tool for pulling debris from pier tops. On older timber trestles, the hose can be used to scrape the dirt from the tops of the caps between the stringers.

Some bearings are not accessible. Designers like to enclose them to improve their appearance. This can mean that the bearings will be left unattended because of the difficulty in removing the housing to inspect them. Regardless of the difficulty involved, bearings should be regularly inspected and cleaned.

3.6.1 Basic types of bearings

3.6.1.1 Masonry plates

Masonry plates are common in bridge bearings and serve to distribute the vertical load to the concrete support. They are usually constructed of steel and attached to the concrete by means of anchor bolts. Sometimes steel retainer plates are attached to the masonry plates to keep other parts of the bearing system in the horizontal alignment.

3.6.1.2 Roller expansion bearings

Roller expansion bearings normally incorporate a masonry plate, and are usually constructed of structural steel. The action of a roller or a combination of rollers accommodates superstructure translation. Rotation may occur by the action of a roller or with the use of a bearing pin. The rollers of these devices may not

always be visible unless part of the bearing is dismantled.

3.6.1.3 Fixed steel bearings

Fixed bearings restrain longitudinal superstructure movement, but should allow rotation in the same direction. The use of rollers, bearing pins, curved plates, or other devices of similar configuration may accomplish this. Fixed steel bearings are normally set on a masonry plate usually welded to the main bearing device. Details are worked out from a consideration of the vertical load, the rotational requirements, and the longitudinal forces that must be resisted at the fixed bearing.

3.6.1.4 Rocker expansion bearings

Rocker expansion bearings are usually used in conjunction with a large vertical load, where considerable superstructure movement is anticipated, or a combination of both. However, many states have developed standardized details of rocker expansion bearings for use in the interior as well as in end supports where expansion must be provided for. This bearing type is constructed of steel and includes a masonry plate.

3.6.1.5 Sliding bearings

Sliding bearings are relatively a recent development. Since the introduction of the teflon (TFE), resistance to sliding is no longer a problem because this material has the lowest coefficient of friction of any solid material. Standard bridge bearings are available with a sliding surface of teflon combined with other materials to provide appropriate strength. The overall requirement of the design is to produce moderate compressive strength, chemical inertness, and low friction. In the usual forms, this material is bonded to special backing plates of

carbon steel, stainless steel, and neoprene. Teflon bearing surfaces are fully accredited by the AASHTO specifications along with testing and acceptance criteria. Most of the older sliding bearings were, however, constructed of steel, and relied on the action of one plate sliding on another to allow for superstructure translation. Depending on total span length and vertical deflection, these bearings were not always designed to allow for superstructure rotation.

3.6.1.6 Pot bearings

In principle, elastomeric pot bearings have become popular because of their increased load-carrying capacity. The pots may be made of either steel or aluminum. The most commonly used types are:

- Those permitting rotation in all directions but fixed against translation in any direction.
- Those permitting rotation in all directions, guided for translation in one direction, and fixed against horizontal translation at right angles to the direction of translation.
- Those permitting rotation and translation in all directions (non-guided expansion bearings).

In pot bearings permitting translation, the sliding surfaces are commonly stainless steel and a teflon (TFE) product as shown in Figure 3.74. If the anticipated rotation is high, special spherical or cylindrical elastomeric bearings are available as fixed, guided, or floating arrangements, as shown in Figure 3.75.

3.6.1.7 Elastomeric bearings

Elastomeric bearings are made partially or wholly of elastomer (synthetic rubber) that develops adequate strength to support bridge loads. Two basic types

are available: plain pads (consisting of elastomer only), and reinforced bearings (layers of elastomer and carbon steel molded into a solid void-free mass). The elastomeric expansion bearing is designed to accommodate both horizontal and vertical movement by distortion of the bearing itself. The fixed elastomeric bearing is usually restrained against horizontal movement by the use of anchoring dowels extending into the substructure. Load plates may be incorporated on top and under the bearing to hold the assembly in position and help in distributing the loads. Elastomeric bearings have been used frequently on prestressed concrete or curved steel girders.

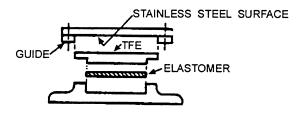


Figure 3.74 Elastomeric Pot Bearing (Xanthakos, 1996)

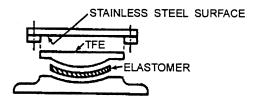


Figure 3.75 Elastomeric Spherical or Cylindrical Bearings (Xanthakos, 1996)

3.6.2 Problems and reasons for failure of bridge bearings (National Cooperative Highway Research Program synthesis of highway practice) (Irick, 1977)

Bearings are provided to enable the structure to move as it changes its dimensions as a result of changes in temperature and other causes. The primary failure is that the bearing does not perform; it freezes or locks up so that no movement is possible. This transfers the stress back into the structure, overloading some elements and causing failure.

3.6.2.1 Unequal bearing action

In a long bridge with several expansion joints, it is the designer's intention that each joint should take its share of the total movement. Without actually freezing, some bearings naturally offer more resistance than others; it is possible that all of the movement for an entire bridge might be transferred into one moving joint with disastrous results. Of two movable bearings supposed to act in tandem, the one that moves the easiest will probably take all of the movement, unless prevented in some way. Therefore, provision must be made with bolts/ties, or movement limiters across the joint to prevent any one joint from taking more than its designed amount of movement and force the excess movement to other joints. If a long bridge had four joints, each of which might move 2 in (50 mm) without limiting devices, the easiest moving joint would probably accept the whole 8 in (200 mm) of movement. This might result tearing that bearing apart and opening a considerable crack in the roadway. Thus, it is necessary to make provision to force each bearing system to perform as designed.

A common solution is the use of bolts across the joints with compressible material under one end so that the allotted movement is possible after which the bolt becomes tight and forces the movement to the next joint. Three 1.5 in (38)

mm) bolts across an expansion joint in a two lane bridge would be sufficient. Ties of this type are not the same as the ties placed across expansion joints to secure the joint from coming apart in an earthquake. Earthquake ties must be considerably stronger.

3.6.2.2 Twisted bearings

Rockers without pintle pins, linkages between the rocker and plates, or keeper plates often move to one extreme of their movement and then lock there and slide. If such a condition already exists, the bearing should be removed and keeper plates installed to force the rocker to act and remain in position. For new work, a rocker should never be designed without pintle pins, gear teeth, or some positive linkage to prevent the rocker from gradually working out of place.

3.6.2.3 Dirt and failure

Dirt, salt, and water are often the primary factors that cause in the failure of almost any mechanical bearing. Mechanical blockage and corrosion can occur simultaneously. Galvanizing is only a temporary (but worthwhile) solution from eventual trouble. Dirt penetrates and grinds the bearing mechanism. The solution is to keep the bearings open and free so that wind can blow debris away or maintenance employees can brush dirt away with a broom stick, compressed air, or water.

3.6.2.4 Wear on pins

Pins that experience movement will wear. Few pins are provided with any means of lubrication and for many locations, the presence of grease would only attract dirt and increase the wear. Hence, the wear is to be expected after a long period of time and should be watched for. Worn pins or bearings should either be

replaced or built up by welding. Larger pins, 6 in (150 mm) and greater, probably do not experience enough wear to make replacement necessary within the life of the bridge.

3.6.3 Bearing failure in skewed and curved bridges

Skewed bridges present a problem to any designer. Often bridges are very wide, and some of the skews are extreme. Normal expansion and contraction do not occur in a direction that is parallel to the centerline of the roadway. Skewed bridges with intermediate expansion joints should be very carefully studied. It has been found that the skew can cause considerable longitudinal and horizontal forces on the joint and its bearings and distress results unless the design takes this into account. Special devices may have to be designed to carry these forces across the skewed joint. On curved bridges, the expansion tends to be parallel to a chord drawn between the bearings rather than on a tangent to the curve. However, without proper instructions, field crews could set the bearings parallel to the tangent rather than the chord, which may lead to bearing failure.

3.6.4 Elastomeric pad failures

The failures of elastomeric pads are sometimes hard to detect. Sometimes when the pad crushes and fails, the vertical deflection may not be easy to detect. If there are voids under the pad the bump in the roadway becomes more noticeable. When the pads are not completely enclosed in the structure, they should be carefully checked for cracks (especially in the bulge), for apparent aging of the material, and for any evident deterioration.

Elastomeric pads do not normally stay from proper position, whether cemented to the bearing surface or not. Therefore, if pads are visible, their position should always be checked. Also, they should be in a vertical position, with due

consideration of temperature deflection. In addition, they should not be pulled excessively in one direction.

3.6.5 Signs of failure

Secondary failures that should alert maintenance crew include the following:

- Cracks on the face or side of an abutment or pier originating in the vicinity of a bearing,
- Spalled concrete (the stage following the initial cracking),
- A bump at a bridge joint,
- A deflection in the bridge railing at a joint,
- A tilted pier or abutment,
- An expansion joint open wide, even though other joints are closed or at normal opening,
- Streaks of rust on the face of a pier from a bearing,
- An expansion joint filled with debris,
- Rockers tilted beyond would be expected from the current temperature condition, and
- Rockers positioned other than 90 degrees to the line of movement.

3.6.6 Corrective maintenance alternatives

Corrective maintenance often entails complete bearing replacement. When something fails completely, replacement is the best solution. Repairs may take many forms, depending on the bearing type. Some older bearing types, such as those with small rollers, may not be repairable and may have to be replaced. Some states have standard plans showing how to replace older bearings with elastomeric pads. Elastomeric pads of inferior quality may have to be replaced. Structures in areas, where there have been earthquakes, slides, or other earth distortions should have their bearings checked, repaired, or replaced as necessary.

3.6.7 Jacking

Jacking up of the superstructure is necessary when replacing an existing bearing or repair of concrete under bearing seats is required. The load from the superstructure member must be relieved from the bearing and arrangements shall be made to temporarily carry the loads while bearing is removed and repaired or replaced. Raising up of entire bearing line simultaneously is preferable to jacking up only one beam, when all the bearings are to repaired or replaced. This will reduce undesirable stresses in the deck slab and could avoid cracking of the deck.

In cases in which one beam is jacked and the adjacent one is not, it is recommended that the beam not be lifted, but that jacking proceed in discret intervals until the load on the bearing is relieved, but before it is actually lifted up. By selecting small enough intervals of incremental jacking, substantial lifting can be avoided. The bearing should be checked after every increment of jacking to see if it has been freed. Also, instrumentation can be used to detect even minute movements. Where every bearing at a bearing line is being replaced some lifting can be permitted in simple spans without fear of cracking the deck slab. However, in continuous spans there are concerns about lifting the spans and inducing cracks in the deck.

At any span where lifting is permitted, there are also other concerns with the magnitude of the lift. Stability of the jacking system can become a problem during jacking, if large lifts are required. Raising up the superstructure more than 1/2 inch would present problems with the deck joints

Temporary Support: Once the structure has been jacked up to the specified height, the structure needs to be secured before the existing bearing is removed. This can be done by one of two methods. One way would be to provide

temporary blocking to support the loads. This can be done in several ways, depending on where the jacking is being done. Frequently either short built- up columns or blocks of steel are used and shims are inserted to fully engage the loads. Another method of temporary support is to use locknut jacks to secure the loads. However, the support must accommodate thermal movements as well as normal vertical and horizontal forces. Provision for expansion and contraction shall be made, if the jacking and shoring system must be maintained for any length of time.

Jacking of the superstructure members can either be done off of the existing substructure seat or on separate jacking frames. If there is room on the existing seats for jacks, a major expense of providing a jacking frame can be avoided. For most bridges with longitudinal stringers or girders, there is not sufficient room to seat a jack under the girders in front of the existing bearing. Furthermore, placing the jack so closely in front of the bearing would make removal of the existing bearing much more difficult.

A much more desirable situation would be to jack off of the existing end diaphragms. Many agencies currently require that end diaphragms be designed for jacking, but unfortunately, not all the existing bridges have end diaphragms so designed. The first option when investigating methods to jack up the superstructure should be to determine whether the existing diaphragms are capable of being used for jacking.

If the diaphragms prove to be sufficient, significant cost savings can be achieved. If not, then consideration should be given to strengthening or even replacing the diaphragms to provide a method to jack, particularly for steel bridges, where steel sections can easily be strengthened by the addition of plates or other sections. On concrete bridges it may be possible to add temporary transverse past tensioning so that the diaphragm can be jacked.

If the end diaphragms cannot be used or strengthened cost effectively, and there is not sufficient room on the seat to jack, then other methods might be considered to increase the seat area. For example, providing steel collars wider than the column diameter can be considered around the top of the column at the seat area. For short stub abutments building a concrete pedestal can become a permanent part of the abutment. If the other options do not prove to be cost effective or practical, then jacking frames should be used to provide a seat for the jacks like a braced frame.

Careful consideration must be paid to the foundation for the jacking frame. A geotechnical evaluation of the existing soil should be done. The point chosen should be as close as possible to the existing bearing for minimizing the negative moment due to overhanging beyond the jacking point. For steel girders, jacking points may sometimes be located under existing intermediate stiffeners, utilizing these stiffeners as bearing stiffeners; if no stiffeners available, additional stiffeners can be added accordingly.

Potential movements should be monitored to ensure that lateral deflections of jacking system remains plumb. Vertical deflections should be monitored to detect when movement is initiated and jacking can be suspended. Electronic monitoring standard surveying tools may be warranted for nonredundant girders.

3.6.8 Summary

From the foregoing discussions, it appears that bearings are the only moving component of a normal static structure, hence they must satisfy special requirements and their performance must be consistent with the intent of the design. Problems common to all types of bearings are dirt accumulation, leakage, corrosion, and rusting.

Various types of bearings are shown in Figures 3.61 – 3.81 and include the simple expansion bearing, the sliding expansion bearing, the fixed bearing, the rocker expansion bearing, and the roller expansion bearing. All types of bearing devices have common problems. The accumulation of dirt and debris on the bridge seat attracts and retains moisture. Continuous exposure to moist conditions, particularly when combined with deicing chemicals, will eventually cause corrosion of any steel member. Although lubricants are frequently applied on steel interface surface, the intermittent flushing with salt and chemical-laden water leaking through the deck joint eventually removes the lubricant.

A rusted sliding surface develops a high degree of fixity against motion and requires considerable force to unlock this fixity and overcome friction. This condition, therefore, is in clear conflict with the assumption made in the design, and very often expansion piers and abutments must resist the same longitudinal forces as the fixed piers. Unless grillage reinforcement is provided and suitably arranged around the anchor bolt, the forces associated with superstructure movement will cause the concrete around the bearing to crack and spall, especially along the vertical face in front of the anchor bolts.

Similar problems are encountered with rockers and rollers. Accumulation of rust under a rocker plate will eventually impede movement on the masonry plate. The rocker may thus be prevented from moving or it may be restrained from returning to its normal position. When these problems are fully manifested, the bearing device becomes locked, and no longer accommodates translation or rotation.

In tilted bearings the loads are eccentrically applied, causing non-uniform pressures and probably overstressing the concrete locally. When the tilting becomes excessive, it may lead to the instability of the bearing and its eventual collapse.

The decision to repair or replace bearings should have explicit criteria, and be based on the ability of these devices to transfer the vertical loads and also accommodate superstructure movement. Deficiencies that warrant repair include light rust or surface scaling of non-contact surfaces, loss of lubrication, debris and dirt accumulation on the bearing seat, minor tilting and displacement of bearing components, rusted masonry and keeper plates, and missing nuts or deteriorated anchor bolts.

Bearings requiring replacement are devices that have undergone severe deterioration, have suffered loss of function, and exhibit signs of imminent structural instability. When new bearing types are considered to replace existing bearings, similarity is nonessential. Instead, the choice may be based on the following:

- the ability of the bearing to provide the same functions in terms of load transfer and movement;
- the compatibility of the bearings with the environmental conditions;
- dimensional harmony of the new bearing, and particularly the overall height, so that adjustments in bearing seat elevations will be minor, and
- the structural compatibility of the bearing with other bridge components in terms of stiffness, so that loads and forces are distributed as initially intended.

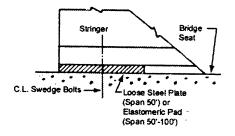


Figure 3.76 Simple expansion bearing (Xanthakos, 1996)

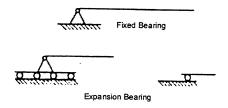


Figure 3.77 Bearing nomenclature (Xanthakos, 1996)

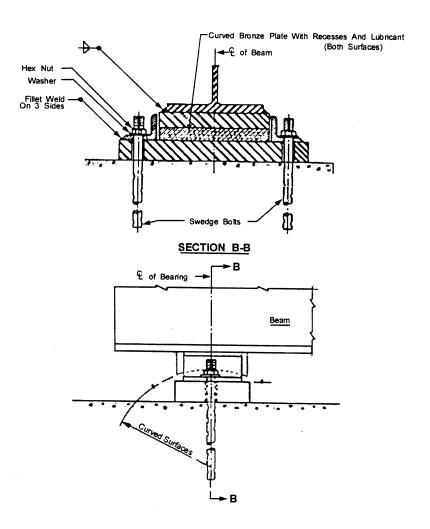


Figure 3.78 Sliding expansion bearing

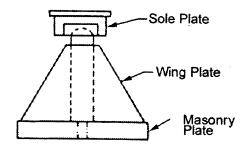


Figure 3.79 Fixed bearing

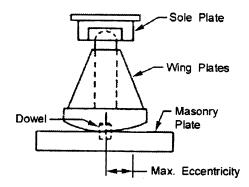


Figure 3.80 Expansion bearing

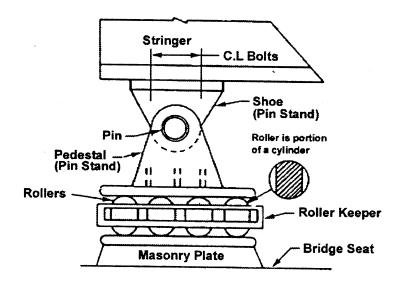


Figure 3.81 Roller expansion bearing

3.6.9 Recommendations

- As a general policy, a bridge should be designed with as few movable bearings as possible. Where allowable, the structure should be designed so that it can absorb normal movements within its elastic system, rather than having mechanical movable bearings. This can be accomplished through flexible piers, longer lengths of continuous super-structure, or limiting all expansion movement to the joints at the abutments.
- Bridge bearings are active mechanisms. They should be designed as such and maintained in the same way.
- Bearings should be designed to require a minimum of maintenance. This applies to the basic design as well as to the inclusion of details that will make any bridge bearing easy to inspect and clean.
- Elastomeric bearings can be designed for concealed locations only after the local experience record has proven them to be reliable.
- Bearings do fail; therefore, provision should be made so that jacks can easily be installed, the structure lifted, and the bearings either adjusted or replaced.
- Rollers and rockers are relatively trouble free devices when properly maintained. Rollers should never be less than 4 inch (100 mm) in diameter and preferably should be larger.
- Bridges with multiple expansion joints should have restrainer bolts placed across the joints so that the expansion movement will be divided approximately equal among the joints, making it impossible for all the movement to be transferred to one joint. Joints in earthquake-prone areas should be more strongly restrained.
- Extra caution should be taken in the placement of concrete around elastomeric pads. It is not easy to place the concrete firmly against the bottom of the pad.

- Material quality is of primary importance in elastomeric bearings. Quality
 of material must be carefully specified. In addition, an adequate inspection
 and testing program should be in operation.
- Bridge bearings should be protected from accumulation of dirt and water. The number of deck joints should be kept to a minimum. Where joints are used, they should be sealed or specific shielding provided to keep deck drainage from getting to the bridge bearings and seats.

The following bearing types should be avoided:

- i) Roller nests. These are impossible to maintain under normal circumstances. Dirt and corrosion inevitably cause failure.
- ii) Bent steel plates with lead sheets between them. These are impossible to maintain or keep clean; the lead works out and the bearing tends to freeze and lock up.
- iii) Bolster shoes pinned through a girder web. The pin almost always freezes and locks the joint.
- iv) Wheel type bearings running on smaller axles. The axles always seem to freeze and lock the joint.
- v) Cast-steel bearings. They are generally too expensive compared with weldments.

3.6.10 Case studies

i) Mosel Bridge (Zichner, 1989)

Built in 1963, the bridge has four prestressed concrete spans as shown in Figure 3.82, in which the short northern span was anchored to prevent uplift. At the beginning of the 1980's, the bridge showed two severe defects:

- a heavily corroded and immovable anchorage link and
- a deflection of the hinge at the center of the main span.

These defects adversely affected stability and serviceability. The kink in the river span was unsightly and affected driver comfort. The following repair work was completed in 1986.

- The deck was lifted to its correct level.
- Bearings were replaced.
- The center hinge joint was closed.
- Supplementary prestressing was installed as shown in Figure 3.83 (b) and (c).
- Defective anchorage was no longer required and hence removed.
- Concrete ballast placed in hollow box in north span.
- Waterproofing, surface finishes, etc. were applied.

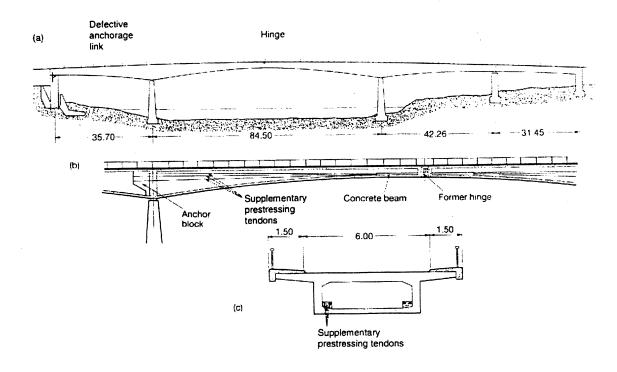


Figure 3.82 Mosel Bridge (Zichner, 1989)

ii) San Milano Bridge, Bruncu Teula, Sardinia (Mallet, 1994)

This five span reinforced concrete bridge was built in 1936. Severe deterioration was found at the joints of the central span. Carbonation was 50 to 60 mm deep and about half the cross-sectional area of the reinforcement had been lost by corrosion. Repair involved the removal of poor concrete, adding new reinforcement, placing new concrete cover and renewing the bearings. To avoid lifting from above and obstructing traffic, the suspended span was jacked against temporary steel girders underneath as shown in Figure 3.83.

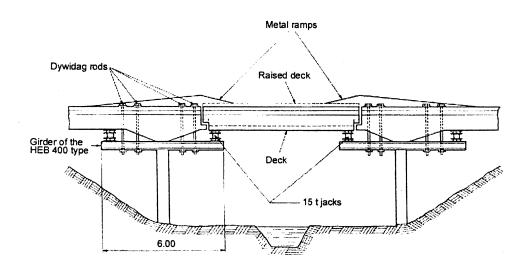


Figure 3.83 Equipment for lifting suspended span

CHAPTER 4

SUBSTRUCTURE REPAIR

4.1 Introduction

In the marine environment, wave action, corrosion, UV deterioration, marine organisms and other forces are constantly attacking the substructure. Use of concrete in marine environment has received somewhat mixed reviews. Its durability has been well demonstrated on several marine structures, yet examples of concrete vulnerability have also been observed frequently. The major problem of concrete during service life is corrosion, which leads to chemical attack of its aggregate/cement matrix and the steel reinforcement. Bridge substructures in marine environment are more exposed to corrosion, since most part of it is located in the splash zone.

Although a vast revenue resource is expended in building bridges, 'managing' their maintenance and actually 'executing the maintenance work' can prove even more exacting and costly, if what has been built must remain operational for the intended long-term safe use. This requires the following:

- a) A thorough examination of the detailed inventories,
- b) Carrying out detailed condition surveys and visual and hands-on inspections,
- c) Analysis of observations and structures in order to unfold the causes of structural distresses,
- d) Carrying out the structural computations and, if any, the appropriate in-situ tests on material samples and on existing structures, and
- e) Preparation of specifications for rehabilitation and repair or outright

demolition and replacement, as necessary.

Repair and strengthening practice are based only on intuition, and reliable experimental data available on the repaired strengthened reinforced concrete structural member behavior are very limited. However, the subject has recently received considerable attention, and few experimental studies are being carried out in Europe and North America. Case studies on bridge substructures are evaluated in this report with emphasis on the condition and behavior of substructures under marine environment.

4.2 General concrete repair techniques for substructures below the waterline

The life of any bridge depends on the preservation of the physical integrity of both the superstructure and substructure; therefore the implementation of an adequate inspection, maintenance, and repair program for the entire structure is essential. However, because there have been few catastrophic failures of bridges, there has been minimal interest in underwater inspection. Unfortunately, in many cases, the condition of the parts of the substructure located underwater is not as easily determined as the condition of those parts located above water.

The quality of inspection under water should be equal to the quality of inspection above water. Inspection of the underwater portions of structures is more difficult; the harsher environment affects the inspector's mobility, visibility, and such functions as cleaning and sampling. However, properly trained, equipped, and supervised personnel can do an effective job.

4.2.1 Scour

Scour is the removal of streambed, backfill, slopes, or other supporting material

by stream, tidal action, dredging, and propeller backwash. The degree of damage depends on such factors as the character of the streambed, the volume of water, and the shape and elevation of the structure. All piers and abutments erected in water should be designed to be protected from scour and erosion; however, both stream and structure conditions may change during the life of the structure.

Piers and abutments are obstructions in a stream that reduce the channel cross-section and increase the velocity, thus causing a disturbance of the flow (Figure 4.1). To avert failures caused by currents, the bridge engineer incorporates in the bridge design the following preventive measures:

- provision for an adequate channel to keep flow velocity low;
- training structures;
- armor protection (concrete rip rap);
- bottom of footing designed to be well below maximum scour elevation;
- extra length piles; and
- streamlining and alignment of piers.

The change in characteristics of a swift river or a strong tide demands constant vigilance. Unusual rainfall and storms boat movement, and dredging of a channel can result in scour around the foundation of piers and abutments. Structures with foundations that are subjected to scour should be inspected immediately after storms that result in flooding and strong currents.

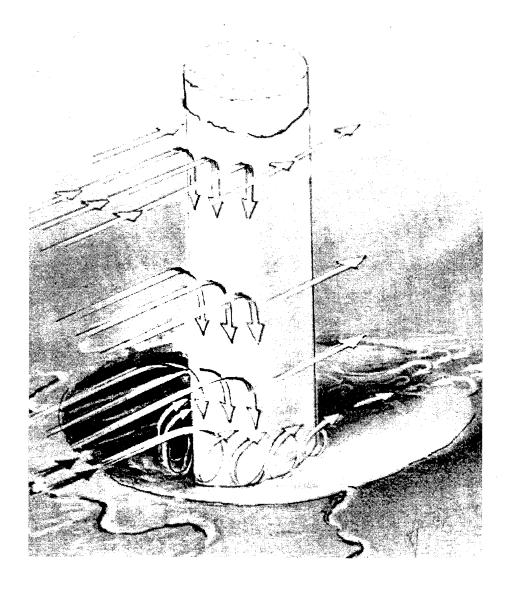


Figure 4.1 Scour around circular shaft

4.2.2 Deterioration of concrete

Concrete is used in pier footings, abutment walls, piles, and seals. Below-the-waterline deterioration of concrete is readily identified by the presence of cracks, spalls, and cavities; however, determining the cause of deterioration is rather difficult (Figure 4.2).

Concrete members, in some instances, deteriorate because of chemical

processes that occur in a marine environment. Magnesium ions in seawater salts attack the concrete by reacting with the calcium. Sulfate solutions react with the tricalcium aluminate hydrate, which is a normal constituent of concrete, forming calcium sulfur aluminate hydrate. Substantial expansion follows this reaction and causes cracking and spalling. Cement contains tricalcium aluminate, which reacts with chloride ions in salt water. Acid generated by bacteria also attacks concrete. Sometimes deterioration is produced by chemical reaction within the concrete mass, such as the reaction of high-alkali cement with minerals in certain aggregates. Also air and moisture that penetrate the concrete cover cause the reinforcing bars to corrode; the swelling of the corrosion products can cause the surrounding concrete to crack and spall. Several types of marine organisms (e.g., Pholas) bore into concrete. However, this damage occurs infrequently in tropical or semitropical waters and appears to take place only in low quality concrete.



Figure 4.2 Concrete deterioration

4.2.3 Structural damage

Structural damage to the parts of a bridge substructure below the waterline, can compromise the structural integrity of the facility, and can be classified as follows:

- construction damage or error,
- collision.
- storm,
- abrasion (erosion), and
- deferred maintenance.

4.2.3.1 Construction damage or error

Construction damage can result from improper procedures or accidents during the construction phase.

Concrete

- often pieces of steel are left protruding from the finished concrete surface, including hardware used to secure the reinforcing steel and formwork in place or lifting rings not removed from precast concrete piles. With time, the corrosion of these metals will cause cracks and spalls in the adjacent concrete area.
- during setting of the concrete, cracks will appear in the structure, if the formwork moves or as a result of vibrations caused by pile driving.
- the concrete in the substructure, if not properly placed, is subjected to cracking caused by differential settlement of the concrete suspension and by initial and drying shrinkage.
- stresses resulting from changes in the atmospheric temperature or in the internal temperature of the concrete mass can cause cracking:
- if the shoring system or framework is prematurely removed, the concrete

may crack severely; and

- when the reinforcing steel is placed with insufficient cover, corrosion may occur.

Piles

Overdriving causes all piles to buckle. Rough or improper handling and overdriving cause cracking on precast concrete piles.

Inspection

Even though the possibility of construction-related damage exists on any job, the underwater structural members of bridges are not routinely inspected by state agencies during or immediately after construction. With the passage of time, the discovery of such damage (cracks, spalls, honeycomb areas, splits, deformed or crippled areas, etc.) becomes more difficult and expensive as a result of the amount of cleaning required before close visual inspection is possible.

Underwater inspections should be conducted during and at the end of the construction phase (after pile driving, underwater concrete placement, etc.) before the bridge is opened for traffic. A thorough underwater inspection ensures the quality of the underwater construction. Underwater inspection after completion of the structure provides data for a valid determination of compliance with the contract specifications and details, and also a base line for future underwater inspections.

4.2.3.2 Collision

Structural damage to underwater portions of bridge substructures can either be collision-related or the direct result of a collision. Collision impact can damage the immediate area, adjacent area or even distant structural components,

depending on the structural frame and connection system used. The degree of damage can range from superficial, affecting only the appearance of a structure, to complete destruction or failure of all or part of the structural system. Examples of the types of damage that can occur are:

- crippled flanges on short, steel batter piles, or similar damage on long, vertical steel piles,
- severed spalling on mass concrete footings,
- full-depth cracks in a concrete column or beam.

4.2.3.3 Storms

Storms can have a detrimental effect on bridge substructures. The velocity of the water and the debris carried increase in relation to the intensity and duration of the storm. The increased velocity of the current during a storm leads to a greater transport of granular materials. Floating debris carried downstream by the water flow may exert horizontal forces against the bridge substructure. Debris accumulation at bridges also increases the potential for scour by concentrating the flow. Waves generated by wind induce hydrodynamic pressures on substructure components and piles.

Underwater investigations should be conducted after a major storms or flood in order to determine if the bridge substructure has been damaged. The diver should look for any evidence of storm damage to the substructure components, fenders, and any other underwater installation, such as submarine cables and cathodic protection for steel piles.

4.2.3.4 Abrasion

Deterioration of substructures by abrasion is caused by wave action, the velocity

of the currents, and the action of suspended particles of sand and silt in moving waters. Any one or a combination of these agents can reduce the section with passage of time.

Deterioration of sections resulting from abrasion can be identified by the worn, smooth appearance of the surface. When the abrasive agent is no longer active, the existing abrasion is not as obvious, but usually can be detected by a general depression of the area.

4.2.3.5 Deferred maintenance

Many problems are associated with lack of maintenance of the parts of bridge substructures located under water. If cracks, spalls, and voids in the concrete are not repaired on time, distress or even failure may occur. Scour at foundations of piers and abutments should be minimized; and accumulation of debris should be removed. Fender systems, navigation aids, warning devices, and clearances must be maintained in order to protect bridges from damage by boats, ships, and barges.

To implement and assure a proper maintenance program for below the water portions of bridge substructures, it is essential to conduct periodic underwater inspections by qualified personnel. Underwater inspections after unusually high waters or storms should also be performed. All deficiencies must be reported and evaluated, and remedial action taken for immediate repair.

4.2.4 Structural failure

Structural failure refers to the reduction of the capability of the bridge substructure or component to such a degree that it cannot safely serve its intended purpose. The factors that may cause or lead to structural failure include

the following:

- absence of proper soil investigation or incorrect interpretation of the results of an investigation,
- inadequate design of the structure,
- poor workmanship and construction materials,
- insufficient provisions in design for exceptional phenomena, such as thermal and biological conditions, rainfall, ice loads, and floods,
- settlement due to overloading,
- deterioration of foundation concrete caused by concrete sulfate,
- scour of the substructure support,
- actual loads heavier than design loads, and
- deferred maintenance.

4.2.5 Foundation distress

Foundation distress refers to an impairment of the strength or load response of the substructure that may limit its intended use. Scour, slides, rotation of piers, or collapse of subterranean caverns can cause foundation distress. Usually evidence of a problem is visible in alignment or grade changes of the deck or roadway surface. This type of information is collected from above-water inspections or testing to estimate the extent and severity of the problem.

4.2.6 Maintenance and repair

Bridge maintenance should include regular inspections of the channel around the structure to identify and report potential problems. The removal of debris that has accumulated in the channel can reduce turbulence or prevent a diversion or blockage of the water. Regular maintenance inspections should also identify changes in the channel profile before there is a threat to the structure, and then a protection system can be added. Inspection is especially important after major

storms.

When scour damage to a substructure element is identified, efforts must be made to re-establish bearing and protect the substructure unit from further scour damage. Typically, remedial schemes use a combination of repair/protection techniques.

4.2.6.1 Bagged concrete

Concrete in bags may be used either to armor the foundation material from further scour or as a form for placing concrete, if it is necessary to restore foundation bearing. Figure 4.4 shows a typical installation. Johann Steere (1922) first patented bagged concrete in Norway, and repair using bagged concrete was first used in the United States in 1968. Typically the bags are filled with the dry concrete ingredients and placed in position. The bags are anchored together and to the substrata with dowels. The cement hydrates and hardens when exposed to water.

Bags may also be filled after being positioned underwater. Modern synthetic fibers such as nylon permit the bleedwater to be expelled, making this technology possible. Because the synthetic fibers are woven into water-permeable fabric, the water cement ratio can be quite low at the surface of the concrete. This method of scour protection is used in Oregon, Pennsylvania, and New Hampshire. Bagged concrete offers the following advantages:

- it is very versatile in size and application;
- it is well suited for a small crew with limited equipment since it can be handled;
- and placed relatively easy in underwater; and
- it eliminates the need for forms.

it eliminates the need for forms.

Bagged concrete has some disadvantages, shown below:

- the dry ingredients are often hand mixed and the water cement ratio cannot be controlled, the strength and durability may not be as good as conventional cast in place concrete,
- the filled bags must be stored and kept dry before they are used; and
- bond and watertightness between bags is not as good as cast in place concrete.

4.2.6.2 Concrete-filled tubes

Flexible nylon tube forms can be filled with grout or concrete to fill scour pockets under substructure units. The nylon tube is placed in position and injected with cement grout or concrete as shown in Figures 4.5 and 4.6. The tubes may be fabricated to the required dimensions; when filled, the tube shown is 10 ft long (305 cm), 45 in (115 cm) wide by 25 in (64 cm) thick. The tubes may be fabricated to the required dimensions. Before the grout or concrete hardens, the tubes are anchored together by steel dowels that are punched through the tube. Tubes are filled in place using a stay-in-place valve on the tube. This method of scour repair is used in the state of Maine.

Concrete filled tubes and concrete-filled bags made of synthetic fibers are basically the same, except for size and shape. Some agencies refer to both as bags. The advantages and disadvantages are basically the same. The tube could be considered as refinement of the bag with the following advantages:

- the tube is normally larger;
- the tube is sized to fit a void; and
- the tube permits the concrete to shape itself to fill the void

4.2.6.3 Prepacked aggregate concrete

Scour voids can also be filled with prepacked, open graded concrete contained by forms and injected with cement grout through pipes. When this repair is performed underwater, the grout is injected from the bottom up. The injection pipe is lifted as grouting operations continue, ensuring that the bottom end of the pipe remains embedded within the freshly placed grout. The Corps of Engineers and the state of Florida have reported the use of this repair technique.

The forms should be watertight to control the loss of the grout, vented only at the top to indicate when the forms are full. The tops of the forms should consist of a highly permeable fabric held down with a wire mesh, which in turn is held down by plywood. This system allows venting while preventing the loss of fines and cement. The forms need to be held down by grouted doweling because of the pressure generated by the grout.

Prepacked concrete has the following advantages:

- water-cement ratio and shrinkage can be low;
- bonding characteristics are excellent;
- concrete is not affected by dilution or washout;
- forms need not be dewatered;
- the forms create flat, neat surfaces; and
- this method fills voids that are difficult to access.

Prepacked aggregate concrete has the following disadvantages:

- marine growth can accumulate on the preplaced aggregate in polluted waters if there is a delay in the construction operation, resulting in poor grout/aggregate bond;
- due to pressure of pumping grout, special anchoring is recommended to hold

the forms in place; and

 the procedure may be relatively expensive. A cost analysis is recommended to compare alternatives such as a dewatering cofferdam for direct concrete placement.

4.2.6.4 Sheet piling

Sheet piling or corrugated metal can be used as formwork to retain either concrete or riprap for repair of scour damage. If riprap is used within the form, a concrete cap may be used to retain the riprap. Additional riprap may be placed around the outside of the form to protect against further scour. This is the oldest and most straightforward method of scour repair. A typical installation is shown in Figure 4.7.

Advantages

- normally, it does not obstruct the channel opening;
- it is economical and simple to design; and
- it can be constructed and repaired in a short time.

Disadvantages

- the steel form is not corrosion-resistant; and
- special equipment is required to drive the sheet piling

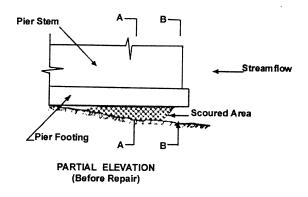


Figure 4.3 Foundation before repair

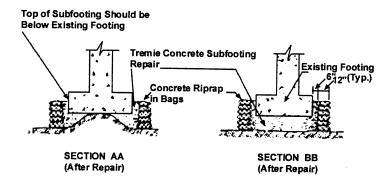


Figure 4.4 Foundation after repair by bagged concrete

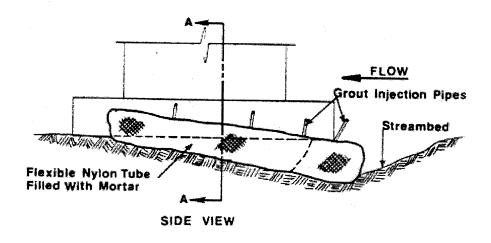


Figure 4.5 Concrete filled tubes

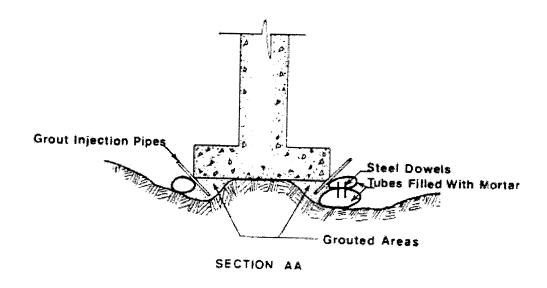


Figure 4.6 Concrete filled tubes (cross section)

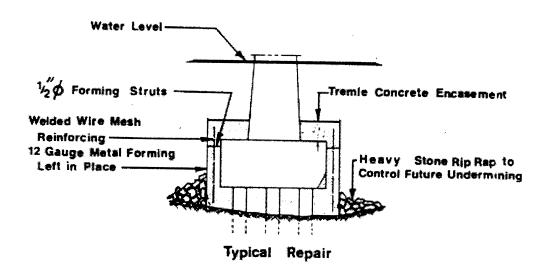


Figure 4.7 Sheet piling scour rehabilitation

The most cost effective repair method will be used based on the degree of concrete deterioration as shown below:

- i) Initiation stage: (a) hydraulic training and (b) penetrating sealant
- ii) Beginning of the propagation stage: (a) surface coatings and (b) veneers
- iii) End of the propagation stage: (a) wraps and (b) jackets.
- iv) Beginning of the destruction stage: (a) crack sealing, (b) sacrificial concrete collar; (c) grout repair; (d) passive cathodic protection and (e) active cathodic protection.

4.3 Abutments

An abutment is a structure located at the end of a bridge, which provides the following basic functions:

- Supporting the end of the first or last span;
- Retaining earth underneath and adjacent to the approach roadway, and if necessary; and
- Supporting part of the approach roadway or approach slabs.

A variety of abutment forms are used to provide this functionality. The style of abutment chosen for a given bridge varies depending on the geometry of the site, size of the structure, and preferences of the owner. A simplification would be to think of an abutment as a retaining wall equipped with a bridge seat. The following discussion describes some of the more popular types of abutments in use, presents a design example for a typical abutment, and covers some of the general maintenance and rehabilitation considerations.

4.3.1 Types of abutments

Most abutments are variations on retaining wall configurations. With the

exception of a crib wall, most retaining wall system, when equipped with a bridge seat and designed to withstand the severe live load conditions present in highway bridge structures, can be used as an abutment. Another difference between a conventional retaining wall system and a bridge abutment is that the latter is typically equipped with adjoining flared walls known as wingwalls.

Wingwalls are designed to assist the principal retaining wall component of an abutment in confining the earth behind the abutment. Examples of wingwalls are given in Figures 4.7 - 4.11 The principal retaining wall component mentioned above is usually called the backwall or stem of the abutment. The bridge seat, upon which the superstructure actually rests, is typically composed of either freestanding pedestals or a continuous breastwall. The pedestal or breastwall is designed to support individual primary member and transfer girder reactions to the foundation. They are located just in front of the backwall and sit on top of the abutment footing. In general, the following types give the major types of abutments presently in use.

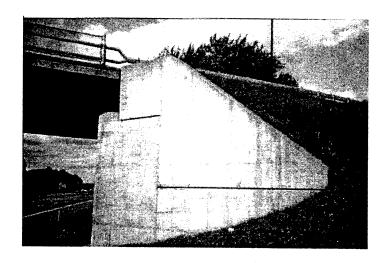


Figure 4.8 Wingwall of a two-span bridge (D.E. Tonias, 1995)

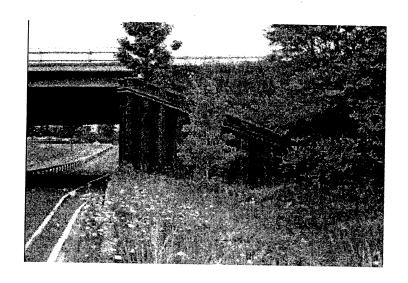


Figure 4.9 Steel sheeting being used as an abutment material (D.E. Tonias, 1995)

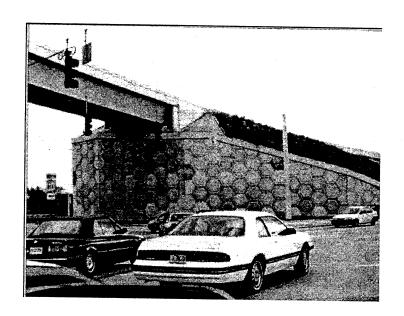


Figure 4.10 Reinforced earth abutment is used to support this composite steel bridge (D.E Tonias, 1995)

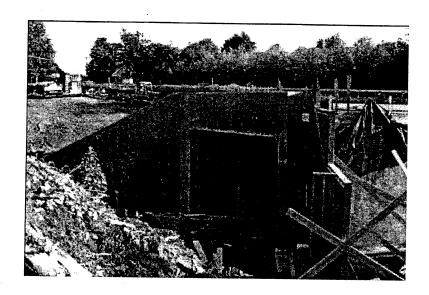


Figure 4.11 Single Span Bridge under construction crossing a stream

4.3.2 Gravity abutment

A gravity abutment resists horizontal earth pressure with its own dead weight. By nature, this leads to abutments, which are rather heavy. Gravity abutments are most often constructed using concrete, however, stone masonry is also sometimes used. As described above, a gravity abutment is composed of a backwall and flared wingwalls which rest on top of a footing (Figure 4.12).

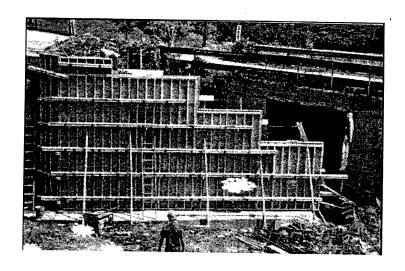


Figure 4.12 Typical gravity abutment with wingwalls under construction (Tonias, 1995)

4.3.3 U abutment

When the wingwalls of a gravity abutment are placed at right angles to the backwalls, the abutment is known as a U abutment. The name "U Abutment" comes from the shape the abutment has when viewed in plan. The wingwalls are typically cast monolithically with the abutment backwall and cantilevered both vertically and horizontally. Because there is a tendency for the wingwalls to overturn, the two wings sometimes tied together. To provide for a better economy of material, the wingwall thickness can be varied linearly, from a maximum at the backwall-wingwall interface to a minimum thickness at the wingwall free end. The maximum thickness is dependent on the size of the cantilever moment induced by lateral forces from the retained earth.

4.3.4 Cantilever abutment

A cantilever abutment is virtually identical to a cantilever retaining wall (i.e., a wall or stem extending up from a footing) except that a cantilever abutment is designed to accommodate larger vertical loads and equipped with a bridge seat.

The stem of a cantilever abutment, along with its breastwall or pedestals, is rigidly attached to the footing and acts as a cantilever beam. The stem transmits horizontal earth pressures to the footing with stability being maintained through the abutment's own dead weight and soil mass resting on the rear part of the footing. The front face of an abutment footing is known as the toe and the rear face as the heel. At times it may desirable to vary the thickness of the stem to achieve an economy of materials. Cantilever abutments are feasible for heights up to approximately 21-ft (6.5 m). If the required height exceeds this value, an alternative, such as counterfort abutment should be investigated.

4.3.5 Full height abutment

A full height abutment is a cantilever which extends from the underpass grade line (either roadway or water body) to the grade line of the overpass roadway above.

4.3.6 Stub abutment

Stub abutments are relatively short abutments, which are placed at the top of an embankment or slope. Unless sufficient rock exists at the site, stub abutments generally are supported on piles, which extend through the embankment.

4.3.7 Semi-stub abutment

As its name would imply, a semi-stub abutment is in between the size of a full height and stub abutment. A semi-stub abutment is founded at an intermediate location along the embankment.

4.3.8 Counterfort abutment

A counterfort abutment, similar to a counterfort retaining wall, utilizes a stem and footing, which is braced with vertical slabs, known as counterforts, which are spaced at intervals along the length of the footing (Figure 4.13). These thin slabs join with the stem and footing at right angles. The counterforts allow the abutment breastwall to be designed as a horizontal beam between the counterforts rather than as a cantilevered stem. Generally, counterfort abutments are used when very high walls are required.

4.3.9 Spill-through abutment

A spill-through abutment utilizes two or more vertical columns or buttresses, which have a cap beam on top of them. The cap beam is, in turn, used to support the bridge seat upon which the superstructure rests. The fill extends from the bottom of the cap beam and is allowed to spill-through the open spaces between the vertical columns, so that only a portion of the embankment is retained by the abutment.

4.3.10 Pile bent abutment

Similar in nature to a spill-through abutment, a pile bent abutment consists of a single cap beam, acting as a bridge seat, supported by one or two rows of piles. Batter piles are used to prevent overturning.

4.3.11 Reinforced earth systems

These systems utilize modular facing units, generally made of unreinforced concrete, with metal (either in the form of strips or mesh) attached to the back. The facing units are cast in the form of a geometric shape, which lends itself to being assembled into a uniform wall (e.g., hexagon, diamond). The metal

strips or mesh are thereby layered in the retained fill, which is compacted. These strips act as reinforcement, transforming the granular soil into a coherent material, which can support both its own weight and that of applied vertical loading. The name, reinforced earth is derived from this effect. A more generic name that is sometimes used is that of Mechanically Stabilized Earth System.

Two obvious concerns with these systems are their longevity and future maintenance requirements. The metal used to reinforce the earth is typically protected through use of galvanized coating. Since reinforced earth systems have a relatively short service record, it is difficult to compare with the traditional, reinforced concrete. Many transportation departments, however, have begun utilizing reinforced earth as an abutment material, because of the obvious economic and even aesthetic reasons. Exactly how these abutments will be performing over a thirty or forty-year life cycle remains to be seen.

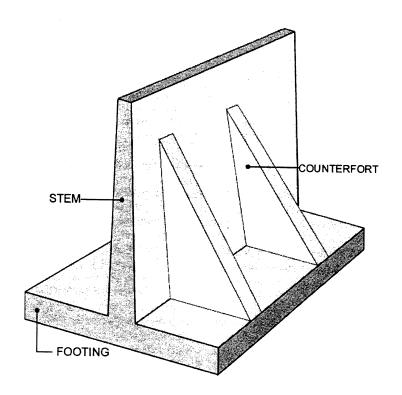


Figure 4.13 Typical counterfort retaining wall (Tonias 1995)

4.4 Rehabilitation and maintenance of abutments

Like any other component of a highway bridge, abutments are susceptible to the effects of deterioration. Particularly in structures with a deck joint located over the abutment support point, a bridge abutment can undergo some of the most severe damage caused by exposure to harsh environment.

Some of the major types of deterioration and maintenance problems, which can occur in an abutment, are the following:

- Settlement or movement,
- Vertical cracking,
- Surface deterioration,
- Deterioration at the water line,
- Spalling under bearing masonry plates, and
- Backwall undermining.

The following discussions cover some of the principal forms of deterioration in bridge abutments, as well as some of the remedial methods. Attention is also given to some of the situations, which can lead to abutment deterioration so that potential preventive maintenance measures can be taken to correct them.

4.4.1 Cracking

Cracks in abutments can develop as a result of a wide variety of situations. Vertical cracking in an abutment backwall can often be initiated by differential (i.e. uneven) settlement of the abutment. Cracks can also be induced by shrinkage. If proper drainage is not provided for the abutment backfill, a situation can arise where the backfill side of an abutment backwall is continually moist, while the exposed face of the wall becomes wet and then dries. This differential between the two faces of the backwall can lead to the formation of shrinkage cracks.

One obvious solution to this problem is to provide adequate drainage of the abutment. This can be facilitated by the incorporation of an under-drain system, which takes runoff and channels it away from the abutment. The under drain conduit is typically made of a corrugated metal pipe, which is perforated to facilitate drainage. The size of the pipe used will depend on the amount of runoff at the site. In the trench and culvert system, runoff is drained into a channel, which is created in a side slope of the abutment. This runoff is then channeled into a storm-water drainage system for eventual disposal.

Alternately, weep tubes can be placed into the abutment backwall at specified intervals to assist in draining the backfill material. A problem with this approach, however, is that the weep tubes themselves can become damaged, either through deterioration or vandalism. This can further exacerbate the problem by allowing moisture to collect directly upon the face of a backwall. The backfill material itself should be of a composition that facilitates proper drainage. Gravel and sand are backfill materials, which are well suited for this. The applicability of any method will vary depending on the specifics of the project site and the owner's preferences.

Another cause of cracking, particularly in older structures, is the use of poor concrete mix or inadequate reinforcement. When such a situation occurs, replacement rather than rehabilitation may be the only solution.

The type of rehabilitation selected to repair cracks will obviously vary depending on the size and magnitude of the cracks present. Some shrinkage cracks, for example, can be relatively fine and, therefore, will not require extensive rehabilitation.

4.4.1 Surface deterioration

Like any other exposed concrete element, abutments can suffer from surface deterioration problems, which are manifested by the presence of

- Spalling,
- Scaling,
- Pop-outs, and
- Spalling off corners

A variety of factors can lead to any of the above situations occurring. Chief among the contributing causes is the chemical attack; either through deicing agents or other corrosive materials being sprayed onto exposed concrete. Other potential causes for surface deterioration are the use of poor concrete mix, poor aggregates, or thermal expansion and contraction of the abutment.

Repair of abutments damaged by surface deterioration is very similar to that described earlier for concrete decks. First, the deteriorated concrete must be removed to a depth, where sound concrete is present. Generally, this is a point beneath the reinforcing steel. This surface should be clean and free from debris, so that new concrete may be bonded into the void. Bonding is typically made with some form of epoxy bonding compound, although mechanical bonding is also sometimes used.

4.4.2 Stability problems

Vertical cracking can result from differential settlement of the abutment. Stability problems such as this can arise from situations ranging from change in soil characteristics to poor design. Some of the major stability problems in an abutment, which could potentially arise as a result of these adverse conditions, or other reasons, are:

Differential settlement or other vertical movement, Lateral movement (sliding), and Rotational movement (tipping, overturning).

Differential settlement can arise from the soil consolidation or soil bearing failure. Lateral movement or sliding of the abutment can be caused by changes in soil characteristics, consolidation, seepage, or failure of the slope. Rotational movement can result from the backfill material becoming saturated with water or erosion of the side slopes.

Another factor, which can initiate any of the above referenced stability problems, is scour. When an abutment is fully exposed to underpass traffic, either vehicular or marine, the potential also exists for impact damage. Generally, this is more of a problem for piers than it is for abutments; however, some site geometry necessitates the exposure of abutments to traffic. If such a situation occurs, the abutment (like piers) should be equipped with a protective barrier system to ensure that the stability of the abutment is not compromised by an accidental impact. This is particularly a problem for abutments, which are supported by exposed piles. Such a configuration is often used in bridges present in marine environments. When piles are exposed to marine traffic, collision can result in possible damage to the abutment itself, as well as failure of the end span. To protect the abutment, a fender or dolphin system should be installed. A dolphin is a collection of piles, equipped with protective caps, which are positioned in a circle around a center pile with their upper end joined together. Dolphins can be constructed out of timber, steel tubes, or sheet piling.

A fender is a protective system, which can be composed of driven piles, rubber, spring elements, or hydraulic-pneumatic components, designed to absorb the impact from a vehicle and thereby protect the bridge element behind it. Fender systems come in a variety of sizes and can be specified to withstand

loads, which range from a small to a very large value.

4.4.4 Bridge seat deterioration

Extreme deterioration of a bridge seat can severely compromise the integrity of the structure. Damage to bridge seats, whether they are in the form of individual pedestal or a single bench (e.g. breastwall) is evidenced by spalling, scaling, pop-outs, and/or spalling off at corners.

Some of the potential causes of deterioration to an abutment bridge seat are:

Chemical attack,

Poor aggregates,

Thermal expansion and contraction, and

Inadequate reinforcement.

Once again, the important factor is a leaking joint. Therefore, the first line of defense that a maintenance department has in protecting an abutment bridge seat is to ensure that the joint over the abutment is functioning properly. If a structure is due for rehabilitation, serious consideration should be given to eliminating joints, whenever and wherever possible. This is not always an option because of obvious economic reasons. If pursued, however, the elimination of joints will greatly enhance the longevity of a bridge. Another form of preventive maintenance, which can be taken, is to apply a protective sealer to pedestals to guard against deterioration due to moisture.

If deterioration of the bridge seat, directly under the bearing masonry plate, has become very severe, then temporary supports, generally in the form of steel bents, must be erected to accommodate the transfer of superstructure loads. Repair of damaged bridge seats may require jacking of the superstructure to correct the problem. The level of repair will naturally vary depending on the

extent of deterioration present. At one end of the spectrum, repair could consist of simply removing concrete to the level of sound concrete and patching the deteriorated area. If inadequate reinforcement was specified in the original design, supplementary reinforcement should be added. Inadequate reinforcement can be evidenced by shear-related problems, like the breaking-off of the concrete or shear cracks. These difficulties arise from the very high shears typical at the abutment bridge seat location.

Another problem, which can lead to deterioration of bridge seats, is the incorrect location of bearing stiffeners. If a bearing stiffener is not placed properly, it can lead to an eccentric loading, which may initiate cracking at the face of a bridge seat. The most economical solution to this problem is to extend the pedestal.

If a bridge seat suffers severe deterioration, remedial patches will not create an adequate bearing surface; then complete removal and replacement of the bridge seat will be required. The existing bridge seat is raised to the top of footing elevation and a new one constructed. If at all possible, it is desirable to preserve the existing reinforcement (provided that it is cleaned properly and in good condition). It may be also desirable to supplement the vertical reinforcement with additional dowels. This can be accomplished by drilling and grouting new reinforcement into the existing footing. If, however, the existing reinforcement is deteriorated to an extent that it cannot be salvaged, then new steel must be used entirely.

Whenever possible, the rehabilitation of any component in a highway bridge should be made to conform to the current details and practices. With an eye toward aesthetics any partial replacement work performed should be detailed so as to match with other components of a structure (e.g., both abutments should either use free-standing pedestals or breastwalls, not a mix of the two).

4.4.5 Sheet piling abutments

An abutment constructed with steel sheet piling shown in Figure 4.8. is also sometimes used in marine environments. When placed in such an environment, steel sheeting is susceptible to deterioration through the presence of high water of varying wet-dry cycles. When placed close to underpass traffic, accidental collision with traffic is another potential problem.

An obvious remedial measure, which can be taken, is to remove and replace any damaged or deteriorated sheeting. Another option is to drive new sheet piling around the existing abutment, filling the void between the two with new backing material. Like other abutments, the material behind a steel sheet-piling abutment must be kept free from water. To enhance the longevity of this type of abutment a coating or waterproofing material (e.g., synthetic resin, linseed oil, etc.) should be used. Areas on steel sheet piling abutments, which are continually exposed to wet-dry cycles, should receive a protective coating in these areas.

4.4.6 Stone masonry abutments (Tonias, 1995)

Stone masonry abutments need to be maintained so that, cracks are not allowed to develop, especially at mortared joints. While stone does not deteriorate as fast as concrete, its tensile strength is generally less than that of concrete. This can lead to problems, particularly when differential settlement occurs. Since deterioration in stone masonry abutments can propagate quickly, it is important that any cracks, which are present, be immediately sealed with mortar to prevent the intrusion of moisture. Defective joints should be raked out and then repointed. This can be accomplished with pressure injection with epoxy. Also any vegetation, which has begun to grow into cracks, should be removed.

4.4.7 Reinforced earth systems (Tonias, 1995)

The metal reinforcing is generally galvanized to prevent deterioration; however, extreme moisture conditions could potentially lead to deterioration of the reinforcement. Another potential difficulty is erosion of the backfill material, which could destabilize the wall.

When such situations arise, the failed sections will require replacement. If the problem occurs at the lower portion of the abutment, then a large segment of the reinforced earth wall will need to be removed in order to carry out repairs. The backfill should be placed in lifts no greater than 15 in (381 mm) and then compacted prior to placing new material. Maintaining and rehabilitating footings (and piles) requires the difficult task of inspecting the components. Unless the footings or piles are exposed, these elements are buried under several feet of earth. There are, however, indicators of footing problems, which can be evidenced, by associated problems in the portion of the abutment, which is above ground. The presence of excessive vertical cracking could indicate foundation problems below the surface of the earth. Therefore, inspectors should look for cracks, which run straight up the abutment stem. If the footing is exposed (i.e., located above grade), either by design or through erosion and scour, spalling and other forms of deterioration of the concrete should be investigated. To repair this damage, it may be necessary to construct a cofferdam around the footing either through the use of steel sheet piling or sandbags.

4.5 Underwater concrete repair (Xanthakos, 1996)

4.5.1 Grout repair

Grouting is a common repair method for underwater concrete. A variety of

epoxies and concretes with modifiers or admixtures are available for underwater resurfacing of large areas of deterioration or section loss. These may be used together with pile jackets or formwork, and may or may not include sandblasting of the reinforcing steel. With proper grouts, the repair can be completely effective. Some of the more commonly used grouts are:

Portland cement concrete with modifiers and mixtures;

Latex-modified concrete slurry;

High alumina cement;

Sulfur-impregnated concrete;

Epoxy mortar grout;

Gun-applied mortar; and

Preplaced aggregate concrete.

The question is often raised as to whether forms left in place permanently enhance protection or simply conceal additional deterioration. For example water absorbent forms such as wood will dewater the concrete mix adjacent to them, and make the surface harder. If these forms are left in place until the repair is fully cured, they may then be removed to allow assessment of the repair. A corrosion cell can be established, if during patching of chloride-contaminated concrete with fresh concrete, the patch comes in contact with the reinforcing steel. In this case, there is a higher corrosion potential that shifts to surrounding cathodic areas of the same bars forming an anodic area, and corrosion thus continues. This problem may be remedied by coating the inside of the concrete cavity and all exposed reinforcement with epoxy bonding compound to insulate the fresh patch electrically.

4.6 Chemical grouting (US Army Corps of Engineers, 1995)

Chemical grouts consist of solutions of two or more chemicals that react to form a gel or solid precipitate as opposed to cement grouts that consists of

suspensions of solid particles in a fluid. The reaction in the solution may be either chemical or physiochemical and may involve only the constituents of the solution or may include the interaction of the constituents of the solution with other substances encountered in the use of the grout. The reaction causes a decrease in fluidity and a tendency to solidify and fill voids in the material into which the grout has been injected.

Applications and limitations

Cracks in concrete as narrow as 0.05 mm (0.002 in) are filled with chemical grout. The advantages of chemical grout include their applicability in moist environments, wide limits of control of gel time, and their application in very fine fractures. Disadvantages are the high degree of skill needed for satisfactory use, lack of strength, and, for some grouts, the requirement that the grout does not dry out in service. Also some grouts are highly flammable and cannot be used in enclosed spaces.

4.7 Hydraulic cement grouting

Hydraulic cement grouting involves the use of a grout that depends upon the hydration of portland cement, portland cement plus slag, or pozzolans such as fly ash for strength gain. These grouts may be sanded or unsanded as required by the particular application. Various chemical admixtures are typically included in the grout. Latex additives are sometimes used to improve the bond.

Applications and limitations

Hydraulic cement grouts may be used to seal dormant cracks, to bond subsequent lifts of concrete that are being used as a repair material, or to fill voids around and under concrete structures. Hydraulic cement grouts are usually

less expensive than chemical grouts and are better suited for large volume applications. Hydraulic cement has a tendency to separate under pressure and thus prevent 100 percent filling of the crack. Normally the crack width at the point of introduction should be at least 3 mm (1/8 in). Also if the crack cannot be sealed or otherwise confined on all sides, the repair may be only partially effective. Hydraulic cement grouts are also used extensively for foundation sealing and treatments during new construction.

Procedure

The procedure consists of cleaning the concrete along the crack. Installing built up seats (grout nipples) at intervals across the crack to provide a pressure-tight contact with the injection apparatus, sealing the crack between the seats, flushing the crack to clean it and test the seal, and then grouting the entire area. Grout mixtures may vary in volumetric proportion from one part cement and five parts water to one part cement and one part water, depending on the width of the crack. The water cement ratio should be kept as low as practical to maximize strength and minimize shrinkage. For small volumes a manual injection gun may be used; for larger volumes a pump should be used. After the crack is filled, the pressure should be maintained for several minutes to ensure good penetration.

4.8 Jacketing (US Army Corps of Engineers, 1995)

Jacketing consists of restoring or increasing the section of an existing member (principally a compression member) by encasing it in new concrete. The original member does not need to be concrete; steel and timber sections can also be jacketed.

Applications and limitations

The most frequent use of jacketing is in the repair of piling that has been

damaged by impact or is disintegrating because of environmental conditions. It is specially useful, where all or a portion of the section to be repaired is under water. When properly applied, jacketing will strengthen the repaired member as well as provide some degree of protection against further deterioration. However, if a concrete pile is deteriorating because of exposure to acidic water, for example, jacketing with conventional Portland-cement concrete will not ensure against future disintegration.

Procedure

The removal of the existing damaged concrete or other material is usually necessary to ensure that the repair material bonds well to the original material that is left in place. If a significant amount of removal is necessary, temporary support may have to be provided to the structure during the jacketing process. Any suitable form material may be used. A variety of proprietary form systems are available specifically for jacketing. These systems employ fabric, steel, or fiberglass forms. A steel reinforcement cage may be constructed around the damaged section. Once the form is in place, it may be filled with any suitable material. Choice of the filling material should be based upon the environment in which it will serve as well as knowledge of what caused the original material to fail. Filling may be accomplished by pumping, by tremie placement, by preplaced aggregate techniques, or by conventional concrete placement, if the site can be dewatered.

4.8.1 Reinforced concrete pile jackets (US Army Corps of Engineers, 1995)

Reinforced concrete pile jackets are used to restore or increase the capacity of an existing member.

Materials

Class III concrete,

Concrete spacers, and

Reinforcing steel (epoxy coated).

Installation

- i) Remove all cracked and unsound concrete.
- ii) Clean pile surfaces of oil, grease, dirt and other foreign materials, which would prevent proper bonding.
- iii) Sandblast exposed reinforcing steel to near white metal.
- iv) Place reinforcing steel cage around pile.
- v) Set forms for concrete jacket (Treat forms with an approved form release agent before placing concrete.)
- vi) Dewater forms and place concrete.
- vii) Leave forms in place for a minimum of 72 hours.

4.8.2 Typical case studies

i) Repair of Belgian road bridge

The continuous bridge was constructed in 1960 with a central span of 52 m and two side spans of 26 m. External prestressing tendons comprised wires anchored by the Magnel 'sandwich' system. Each pair of tendons consisted of 104 wires, which were brought together to form a single bundle at midspan between the inverted T-beams, encased in cement mortar, for protection from the environment. With no waterproofing on the bridge deck, the wires became so corroded that all of the cables had to be replaced in 1976. New BBRV tendons were protected by PVC sheaths and cement grout

ii) Marsh Mills Viaduct

The viaduct was constructed in 1969-70 and, in the late 1970s, cracking developed in the structure. This was thoroughly investigated and in 1980, its cause was diagnosed as expansion due to alkali-silica reaction. It severely affected piles and pile caps with cracks up to 20 mm wide and heavy spalling. It was also identified in piers, crossheads, deck slabs and abutments. Following careful analysis of the structure, the carriageway was reduced from three lanes to two and load restrictions were applied. Steel props were installed to transfer load directly to the piles, where pile caps were reactive. Also various measures were taken to reduce the incursion of water into the structure to reduce the development rate of the reaction. The measures included were waterproofing, improved deck drainage and shielding of exposed concrete copings by the use of preformed panels. The condition of the structure was monitored until its replacement in 1994-95.

iii) Florida bridges

Site visits were arranged to inspect bridges located in Jupiter and West Palm Beach. The piles in the bridges were repaired using fiberglass and reinforced concrete pile jackets.

Bridge number 930075 (Jupiter)

This bridge was built in 1957 by the Union Bridge and Central Florida Construction Company. Barrier walls and new approach guardrails were added in 1988 (Figure 4.14). The bridge was rehabilitated In 1995 to include pile jacketing, crash wall repair, end bent slope repair and expansion joint replacement.

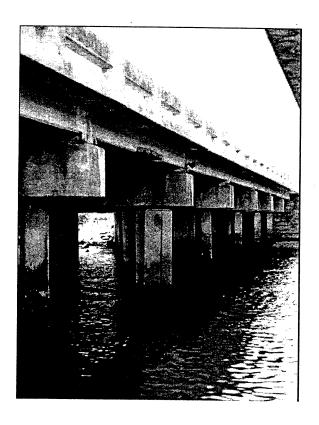


Figure 4.14 Elevation of bridge number 930075 (Jupiter)

Bridge description

The bridge has a length of 225' with 9 spans. The bridge is located on Northbound S.R. 5 (U.S.1) over relief canal. This location is ¼ mile north of the intersection of S.R. 5 (U.S.1) and S.R. 706 (Indiantown Rd.) The bridge substructure comprises of 14" diameter precast piles.

Deficiencies based on inspection

Certain piles were observed to exhibit incipient spalls. No reinforcing steel was exposed due to the spall and minor cracks were also observed below the cap and above the pile jacket. The diver inspection report indicated that one structural jacket exhibited crumbling concrete exposing the reinforcing steel.

Bridge number 930116 (Jupiter)

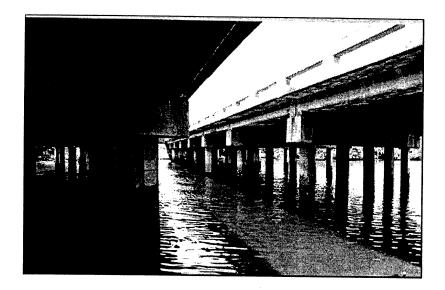


Figure 4.15 Elevation of bridge number 930116 (Jupiter)

This structure was constructed in 1957 by the Union Bridge and Central Florida Construction Company. In 1998 barriers and new approach guardrails were added to the bridge. The bridge was rehabilitated in 1995 to include pile jacketing, crash wall repair, end bent slope repair and expansion joint replacement (Figure 4.15).

Bridge Description

The bridge has identical features as the previous one and they are parallel to each other.

Deficiencies based on inspection

One pile exhibited a crack without exposure of any reinforcing steel.

Bridge number 930087 (Jupiter)

This structure was built in 1957 by Union Bridge and Central Florida Construction Company. Pile repairs were completed in September 1985 including installation of pile jackets (Figure 4.16). In early 1988 barrier walls and new approach guardrails were added together with repair of abutments and joints and addition of new riprap



Figure 4.16 Underside view of bridge number 930087 (Jupiter)

Bridge Description

The bridge has a length of 68.5 m with 9 spans. The bridge is located on Northbound S.R. 5 (US1) over F.E.C. canal. This bridge is about $\frac{1}{4}$ mile south of Bascule Bridge 930005. The bridge has 54 precast prestressed piles.

Deficiencies based on inspection

Many piles showed severe section loss exposing the tensioning strands and pile deficiencies involving vertical hairline cracks were discovered below and close to the cap.

Bridge number 930117 (Jupiter)

This structure was built in 1957 by the Union Bridge and Central Florida Construction Company. Pile repairs were completed in September 1985 including installation of pile jackets (Figure 4.17). In early 1988 barrier walls and new approach guardrails were added with repairs to abutments and joints and addition of new riprap.

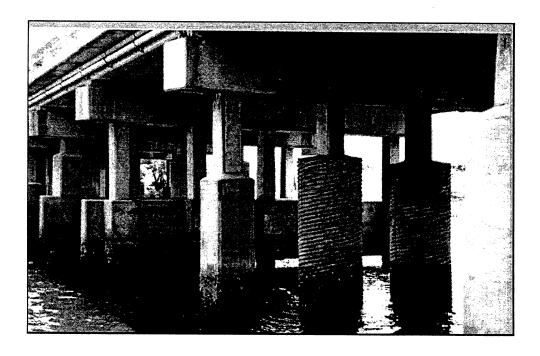


Figure 4.17 View showing the pile jackets of bridge number 930117 (Jupiter)

Bridge description

The description of this bridge is identical to the previous one.

Deficiencies based on inspection

Many of the piles have severe section loss exposing the tensioning strands. New pile deficiencies involving vertical hairline cracks above the mudline and below the cap were observed during inspection.

Bridge number 930004 (West Palm Beach)

Parker bridge is located on Federal Highway (US1), south of PGA Boulevard, West Palm Beach, Florida. This bridge was constructed in 1956 by Cleary Brothers Construction Company and rehabilitated in 1988. A hydraulic alternate was installed on the moveable portion of the bridge (Figure 4.18).

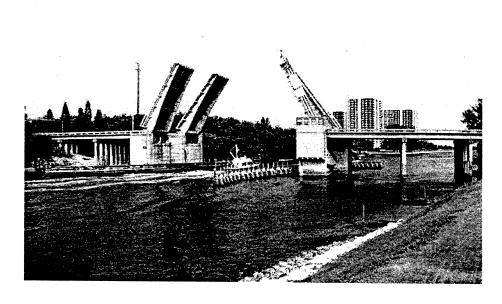


Figure 4.18 Side view of bridge number 930004 (West Palm Beach)

Bridge Description

Parker bridge is 444' x 52' steel and concrete double bascule, 8 span structure. The two 26' concrete roadways have a 9" high curb on either side, 25" wide. There is a 6'6" wide median strip 6" high curb. The draw span 5, is 116' 6" long and has standard steel grading, and the curb and bridge rails are also of steel in this span. The rest of the structure has a 25" high concrete post and bridge rail.

Spans 1 - 4 and 6 - 8 each contain 10 W 30 x 108 steel beams with fixed and sliding end bearings. The draw span 5 consists of 5 steel beam stringers W 16 x 40 on each of the 4 bascule leaves. There are two built-up plate girders on either side of each bascule leaf. The floor beams 1-3 are W 30 x 180 and floor beam 4 is a plate girder. Counter weight beams 1 and 2 are also of plate girders.

Bents 1 and 9 are end bents (abutments) and contain 12 precast concrete piling 18" x 18". Each of the bents 2, 3, 4, 7 and 8 have ten 18" x 18" precast concrete piles. Bents 5 and 6 are the bascule piers constructed of reinforced concrete resting upon 126 18" x 18" concrete piling; these piles are not visible. Bents 1 and 9 have reinforced concrete caps 3' x 3' x 70' 6". Bents 2- 4, 7 and 8 have reinforced concrete caps 2'8" x 2'8" x 63' 6". The bascule piers 5 and 6 are of reinforced concrete 23' 3" x 31' 2".

At each pile bent there is an expansion joint in the deck. The control house is constructed of reinforced concrete 9 1/2' wide x 19' long and has aluminum windows and doors with a float built-up type roof. There are traffic barriers at both ends of bridge equipped with red flashing lights but no audible bells. A new concrete pile fender system is constructed at the channel, running north and south. Vertical clearance is 25' above the waterline of the 90' wide channel.

Deficiencies based on inspection

Numerous fiberglass stay-in-place forms were removed and twelve (12) structural pile jackets installed. A special post repair inspection revealed cracks in eleven piles at Bents 3 and 4, where the fiberglass stay-in-place forms had been removed in the splash zone. Apparently the extent of the deterioration at these eleven piles did not warrant the installation of structural pile jackets. Spalling of concrete, exposure of reinforcing bar and horizontal hairline cracks were observed during the inspection. The diver inspection report showed crumbling concrete with exposure of reinforcing steel. Inspection of the structural pile jackets showed the bridge to be in good condition and the existence of the repair was barely noticeable from the original construction. The structural pile jackets has been in place for 7 months and is working very well. A visual inspection of bridge was performed on 4/16/98. The bridge was in good condition, and the existence of the repair was barely discernible from the original construction. (Figure 4.19).

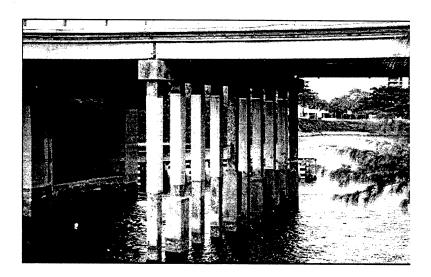


Figure 4.19 Side view of bridge number 930004 (West Palm Beach)

iv) Underwater inspection of the world's longest bridge (LeMieux, 1998)

Twin bridges crossing Lake Pantchartrain near New Orleans are each 24 miles in length and are supported on more than 9500 prestressed concrete cylindrical piles (Figure 4.20).

The first bridge was completed in 1955 and is one of the oldest prestressed concrete bridges built in the United States. The second bridge was completed and opened in 1965.

The bridge is supported on three pile bents instead of two pile bents that support the old structure. The vertical pilings supporting the deck are all 4.5-ft (1.5 m) in diameter. They were cast in 16-ft (5.33 m) sections. The method used to make the piling and the procedure used in driving them to a large extent governed how they have performed.

The primary purpose of the current inspection was to satisfy the requirements of the "National Bridge Inspection Standards". The secondary goal of the inspection and in-depth investigation was to provide information necessary to develop a pile rehabilitation program by which the life of both bridges can be extended.

Environmental conditions

Connected to the Gulf of Mexico, Lake Pantchartrain is brackish with a salt content that varies with location and season and can reach as high as 5 ppt (parts per thousand).

Findings

On the pilings supporting the older bridge, the inspection disclosed a number of vertical, mostly hairline cracks spaced around some pilings at about 10 inches (254 mm). The location of these cracks caused concern, since they

appeared to occur over the location in the vicinity of prestressing tendons. Another area of concern was with the joints between the 16-ft (5.33 m) sections that made up the pile. Grout was missing from a number of these joints. The inspector was able to push a knife blade through several joints to the inside of the pile. As the inspection on the older bridge proceeded to the south, the number of cracks diminished; after about the first $\frac{3}{4}$ mile (1322.5 m), they were found only infrequently.

Information obtained from the chief engineer of the construction company that drove the pilings revealed that they had trouble driving the pilings in the beginning. A new hammer had to be designed to eliminate some of the earlier driving problems. Today inspectors believe that most of these cracks were caused by overdriving. There was considerably more cracking in the pilings supporting the newer bridge. These cracks all ran vertically and some were visible above the waterline.

A joint between pile sections in the new bridge occurred a few feet under the water. The divers found numerous vertical cracks starting at this joint and extending downward into the mud. These cracks increased in size so that at the mud line many cracks were more than 1 inch (25.4 mm) in width. Very few of these cracks extended upwards above this joint.

Again a study of the methods of constructing the two bridges disclosed why, in the inspector's opinion, the newer piling showed more distress. The data collected supported the overdriving theory. A comparison of the driving records of the pilings for the two bridges convinced the inspectors that the problem was caused by driving the pilings to grade on the new bridge, which was less expensive to the contractor than cutting off approximately 1 ft (1/3 m) of pile when design refusal was met. The cracking occurred in a number of pilings all across the length of this bridge.

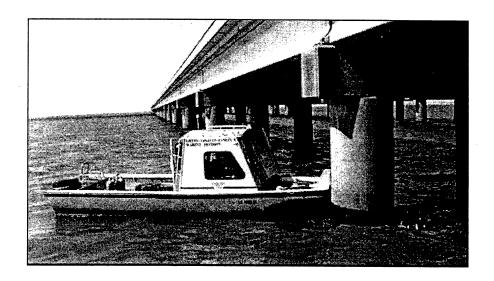


Figure 4.20 Pilings of the 24-mi. long bridges crossing Lake Pontchartrain

During the inspection of the 9500 piles supporting the twin bridges spanning Lake Pontchartrain, a concurrent study was undertaken into methods that could be used to arrest any deterioration uncovered. This study began with a literature search into techniques used for long term protection of piles and, more specifically, protection of Raymond cylinder piles.

The study was expanded to include four basic types of pile protection:

- a) Wraps with and without mastic infill,
- b) Bags with cementitious infill,
- c) Rigid jackets with cementitious infill, and
- d) Rigid jackets with polymer or polymer grout infill (all-polymer).

Letters were mailed to the manufacturers of encapsulation products requesting information on their product and locations, where inspection in the field could be made. The findings are briefly summarized as follows:

Wraps

Several wrap systems were examined and largely, they all employ a flexible

outer wrap and corrosion inhibiting inner liner or mastic coating. The inner liner is placed around the pile, overlapped with a flexible cover, and secured with a roller or bolted seam. Some of these systems have foam or sponge top and bottom seals, and some use stainless bands to hold the wraps in place.

The inspectors found several instances, where the soft outer wraps were damaged and saw considerable evidence of seam and seal failure. The chief inspector examined a wrap installation that was less than one year old and most of it was in some state of deterioration. The evidence showed that once the outer wrap, the seals, or the seams are compromised, wave and tidal activity would cause water to pump up and down inside the wrap, allowing fresh chlorides and oxygen to reach the concrete pile.

Bags

Nylon bags are placed around the pile and filled with a mixture of concrete or cementitious grout. The bottom of the bag is usually banded to the pile and the longitudinal seam is closed with a zipper. In some cases, the top of the bag is supported with a steel ring. Concrete or grout is placed into the bag through a hose that enters through a opening at the top of the bag or, in some cases, through openings in the bag. Typically, the annulus between the pile and the bag is 4 to 6 in. (101.6 to 152.4 mm), adding considerable weight and area for wave forces to act against the pile. Several reports showed that the bags will often stretch, increasing the annulus and weight substantially. The study also found several reports of bag failure.

Rigid jackets with cementitious infill

Rigid fiberglass jackets or forms are installed around the piles and filled with concrete or cementitious grout. The study revealed that the general practice of

installing this type of system involved filling the jacket from the top by hose or other means. The evidence is considerable that the infill could be of poor quality due to the concrete or grout falling, at least part of the way, through water. This can lead to water being trapped in the jacket. Because the jackets are opaque, defects in the infill go unnoticed. Like the bags, the annulus is usually several inches or more, adding unnecessary weight and wave forces acting on the structure.

Rigid jackets with polymer infill

Rigid fiberglass jackets installed around the piles are filled with either neat epoxy or epoxy grouts. In one process, the grout is poured from the top down. As found with the cementitious infill methods, most of the fiberglass jackets were opaque, making it unlikely that defects in the infill would be detected. The single exception, however, was the A-P-E (Advanced Pile Encapsulation) process that utilizes a translucent fiber reinforced plastic (FRP) jacket and pumps the epoxy grout into the jacket from the bottom up.

Advanced pile encapsulation (A-P-E) process

The glass fiber reinforced polymer (FRP) jackets supplied by Master Builders, Inc. as part of the A-P-E process are made up of marine grade laminate of glass woven roving and mat, impregnated with a clear, UV light-stabilized, polyester resin. The jackets are translucent to allow the progression of grout inside the jackets to be monitored from outside. The jackets are precisely molded to conform to the structure being encapsulated with grout injection ports and integral overlapping seams. The A-P-E pile grout is pumped into the jackets from the bottom up, through injection ports strategically placed along the length of the jackets. This assures a uniform, dense encapsulation, free of voids and air or water pockets.

Advantages

The inspectors had the opportunity for careful examination of approximately 20 A-P-E process installations 8 years old on the causeway. The conclusion was that the encapsulations were as good or better than when they were installed 8 years ago (Figure 4.21).

A core taken through the encapsulation and 5 in. (127-mm) wall of the pile was sent to the test laboratory, since the inspectors wanted to learn whether the chloride penetration into the pile had been arrested with the installation of the encapsulation. A comparison of the results from the encapsulated pile with those from piles not encapsulated was made. The total acid soluble chloride by weight (percent of sample) from the encapsulated pile was 0.007 %, while the total chloride from a non-encapsulated pile was 0.108 %.

Instead of the normal premixing of reactive components or "hot potting," the components are kept separate throughout the batching, mixing, and pumping phases to be blended just before entering the FRP jacket. This eliminates the need to purge the equipment and hoses periodically and, because the grout is not catalyzed, allows plain water to be used for cleanup. Environmentally harmful solvents are virtually eliminated.

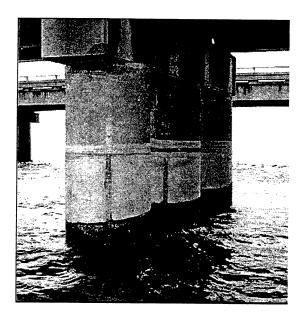


Figure 4.21 A trio of A-P-E repairs found to be performing well after 8 years in service

Specifications

After doing literature search and examining the existing encapsulation systems, the A-P-E process was considered the best available method of protection. A decision was made to encapsulate any pile with three cracks or more and to pressure inject epoxy into those cracks on piles that had two cracks or less.

vi) Boston street terminal in Baltimore (Master Builders, 1996)

Some older marine structures such as Pier 1 at the Boston Street terminal in Baltimore, are typical examples of pile jacketing failure. At various periods of this structure's history, 299 of the 342 concrete piles supporting the structure were jacketed, using a variety of methods and materials. Following are the different cases from a report of substructure Inspection on Pier 1, that categorizes the various types of pile jacketing deficiencies found during the inspection (Figures 4.22 to 4.27). The report also examined the remaining 43 piles that had no previous jacketing. The condition of the piles shown in Figure 4.22 (case 1)

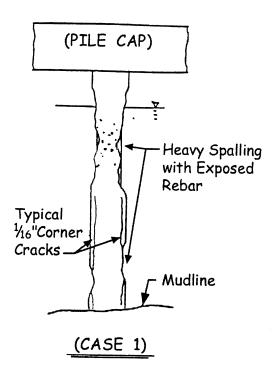


Figure 4.22 Pile with no previous jacketing

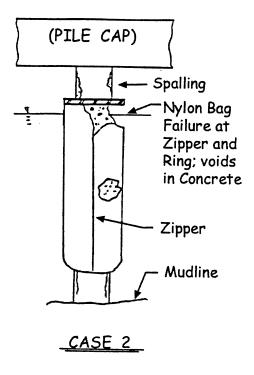


Figure 4.23 Nylon bag failure

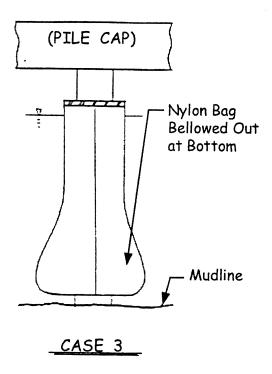


Figure 4.24 Nylon bag bellowed out at bottom

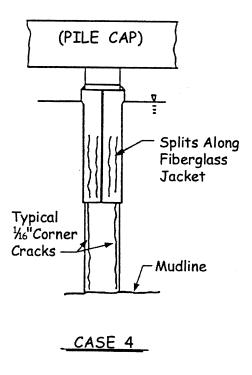


Figure 4.25 Fiberglass failure

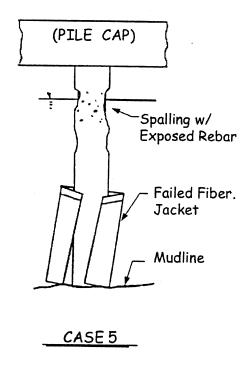


Figure 4.26 Failed Fiber Jacket

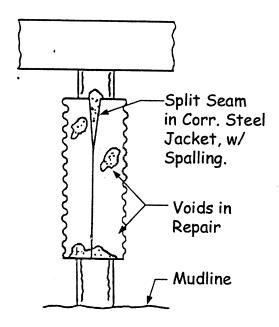


Figure 4.27 Split seam in steel jacket

One of the most common methods of pile jacketing is the combination of fiberglass reinforced polymer (FRP) jackets and Portland cement based

grouts or concrete, usually placed into a 3 to 5 inch (76.2 to 127 mm) space between the pile and the jacket. It is also a common practice to place a short lift of polymer grout [6 to 9 inches (152.4 to 203.2 mm) thick] at the top and bottom of the cement based grout.

If installed properly, pile jackets of this type would perform well for several years, but a large percentage of those observed have failed. The apparent causes of failure range from improper selection of materials to faulty installation techniques and, in many cases, to inadequacy of the construction specifications. The following presents the various mechanisms that cause failure:



Figure 4.28 Missing FRP jacket

Figure 4.28 shows that the FRP jacket is completely missing and the cement grout is discontinuous in the splash zone. Lack of bond between the jacket and the grout is the principal cause of jacket failure and the presence of water in the jacket during placement of the grout is the apparent cause of grout failure. This

is typical of installations, where the grout is poured into the top of the jacket, causing the grout to fall through water standing in the jacket. The specifications on this project require a 6" (152.4 mm) thickness of epoxy grout at the top and bottom of the cement grout, which was apparently not carried out correctly. Because the jacket was opaque, the defect could not be seen at the time of installation.

Usually the first visible sign of failure is the jacket disbonding from the encapsulation grout (Figure 4.29). This lack of bond between the FRP jacket and the grout can be caused by many factors, such as, improper surface preparation of the jacket or marine micro-organisms on the jacket surface, but the most common cause is placement of the grout into the jacket from the top down, with at least some of the grout passing through water. Portland cement based grouts have a much lower bond potential with FRP jackets than polymer based grouts and, when poured through water, have virtually no bond at all. Once bond is broken, wave action quickly overstresses the jacket at the seams or corners and the jacket fails.

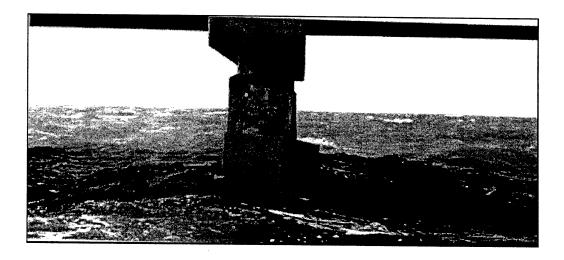


Figure 4.29 Jacket disbonding from the encapsulation grout

Once a jacket fails, the encapsulation grout is vulnerable to wave action and the other forces of deterioration. When the grout has been poured from the top (Figure 4.30) the grout is of low quality and quickly disbonds from the pile. The grout that remains is also very permeable. The chaulky consistency of the grout (laitance) at the bottom near water surface can be observed that is indicative of pouring through water.



Figure 4.30 Grout disbonding from the pile

Sometimes, encapsulations fail, but the jackets remain in place and at least partially hide the failure. Deterioration of the encapsulation grout can be very severe at or near the waterline, yet the jacket is held in place by the polymer grout cap that bonds more tightly with the FRP jacket (Figure 4.31). Figure 4.32 shows the missing jacket and the discontinuity of the grout at the waterline. There were 8 ft encapsulations, with approximately 4 ft above and 4 ft below the waterline. The original deterioration of the pile that necessitated the encapsulation still continues unabated.

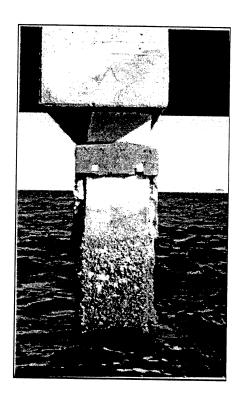


Figure 4.31 Deterioration of the encapsulation grout



Figure 4.32 Missing jacket and discontinuity of the grout

The specifications for the installation of the encapsulations shown did not stipulate the method of grout placement, either for the bottom seal polymer grout, the Portland cement based grout or the polymer top off grout. The jacket length underwater, It was expected the contractor would trime place the bottom seal and the cement based grout in the jacket with half of its length underwater. What was apparently overlooked, was the fact that the proximity of the pile cap to the top of the jacket virtually prohibited proper tremie placement. Also, some specifications call for wire mesh that is intended to hold the grout together, but this interferes with any effort to tremie place the grout.

Figure 4.33 summarizes some of the more common causes of failure found in encasement systems consisting of fiberglass jackets and Portland cement based grouts or concrete. These observations were made over a five-year period of time, involving numerous structures and hundreds of piles.

vii) Repairing columns without using forms (Tampellini, 1999)

To reduce time and costs in repairing 80 damaged columns of the concrete bridge overpasses, the New Jersey Turnpike Authority opted to use a new hardshell concrete restoration system. Forgoing the use of the conventional wet fiber wrapping process, the hardshell system and an epoxy adhesive were used to install composite jackets on eight columns a day. This reduced repair time by as much as 50 %. The composite fiberglass/vinylester shells are manufactured by a vacuum-assisted infusion molding process that produces large, high performance composite parts with a low void content and high fiber volume.

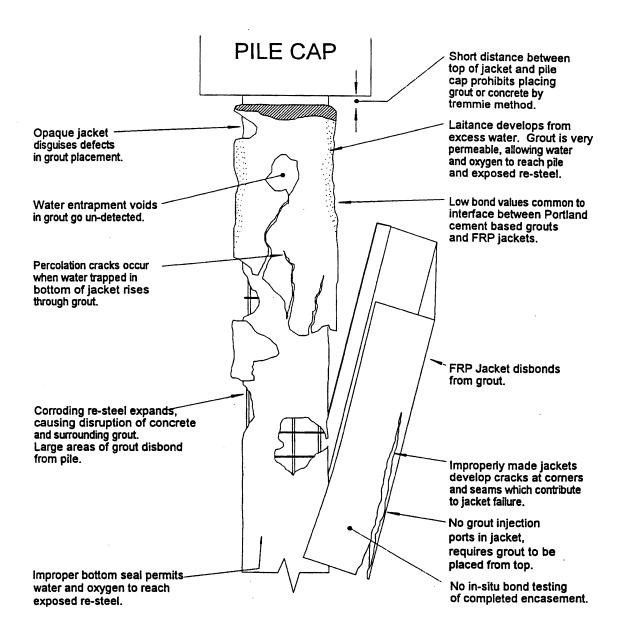


Figure 4.33 Summary of the most common causes of failure (Master Builders, 1996)

To extend the service life of the composite jackets, ultraviolet radiation inhibitors are added to the vinyl formulations and, for the New Jersey Turnpike project, UV resistant acrylic-bonded outer skins were added. The skins produce a resin rich surface to improve the shell's ability to withstand exposure to both sunlight and salt water. In service, the hardshell jackets offer the performance properties to

effectively reinforce columns, including a tensile strength of 90 psi (620 GPa) and tensile modulus of 5000 ksi (34.5 GPa).

For the Turnpike project, the columns ranging in size from 9 to 15 ft (3 to 4.5 m) in height and 28 in. (711 mm) in diameter, were excavated to about 12 in. (300 mm) below the mudline and 3 in. (76 mm) beyond their outer diameter. Highpressure water cleaning machines were used to remove the heavy oxides from the concrete surface. For the jacket installation, workers opened a jacket and place it around the column in the excavation area to verify length. A clearance of 2 to 6 in. (51 to 152 mm) was kept between the column surface and the installed jacket, into which grouting would later be pumped. The jacket was then removed and length trimmed, if necessary. Surfaces were sanded and cleaned with solvent to prepare for adhesive application. Ciba's TDT 177-149 epoxy adhesive was applied to the groove of the H connector along the length of the shell edge. This adhesive is specially formulated for use on the hardshell jackets, permitting installation in either wet or dry conditions. After adhesive application was completed, the jackets were reinstalled on the column and ratchet straps used to close the top and bottom of each shell. Excess adhesive was removed and a clean seamed appearance was the result.

Advantages

It eliminates setting forms to place concrete. Spalled concrete is simply removed and then columns are wrapped and filled in a fraction of the time required for similar projects using set fiber wrapping. In addition to speed, the hardshell jackets provide for neater installation than the wet wrapping processes, allowing workers to start and complete each column with minimum preparation and clean up.

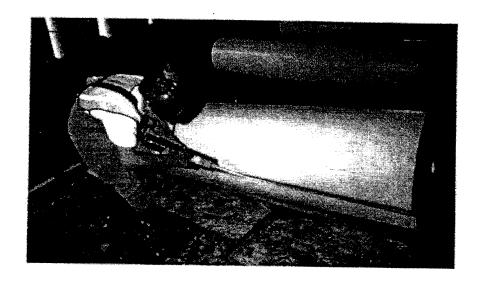


Figure 4.34 Jacket being coated with adhesive material (Tampellini, 1999)

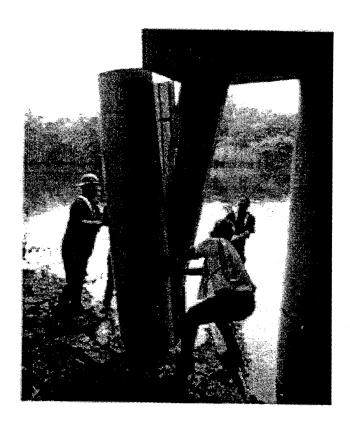


Figure 4.35 Jacket being installed around the column (Tampellini, 1999)

4.9 Piers

The use of bridge piers was generally confined to structures crossing rivers or railways. With the development of massive transportation network, like the US Interstate system, the need for land piers to facilitate grade-separated highways increased dramatically. A pier is a structure located at the end of a bridge span, which provides the basic function of supporting spans at intermediate points between end supports (abutments). Piers are predominantly constructed using concrete, although steel and, to a lesser degree, timber is also used. The reinforced concrete is generally used, although prestressed concrete is sometimes used as a pier material for special structures. The basic design functions of a highway bridge pier are the following:

- Carry its own weight,
- Sustain superstructure dead and live loads, and
- Transmit all loads to the foundation.

4.9.1 Types of piers

4.9.1.1 Hammerhead

A hammerhead pier utilizes one or more columns with a pier cap in the shape of a hammer. Figures 4.36, 4.37 and 4.38 show two types of hammerhead piers. Hammerhead piers (Figure 4.37) are constructed out of conventionally reinforced concrete. The supporting columns can be either rectangular (or other polygonal shape) or circular in shape and extend down to a supporting foundation.

Hammerhead piers are predominantly found in urban settings, because they are both attractive and occupy minimum space, thereby providing room for underpass traffic. Individual transportation departments often maintain standards as to the use of hammerheads.

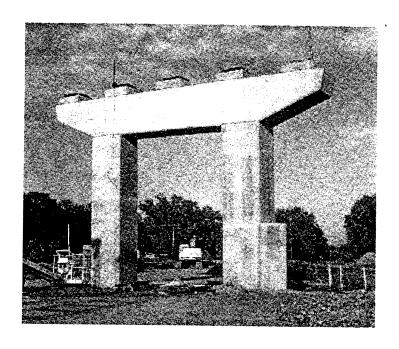


Figure 4.36 A two column, concrete hammerhead pier under construction (Tonias, 1995)

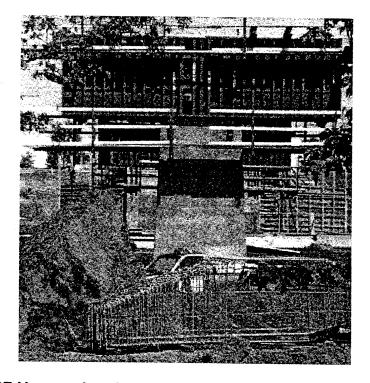


Figure 4.37 Hammerhead pier with its cap beam form work in place (Tonias, 1995)

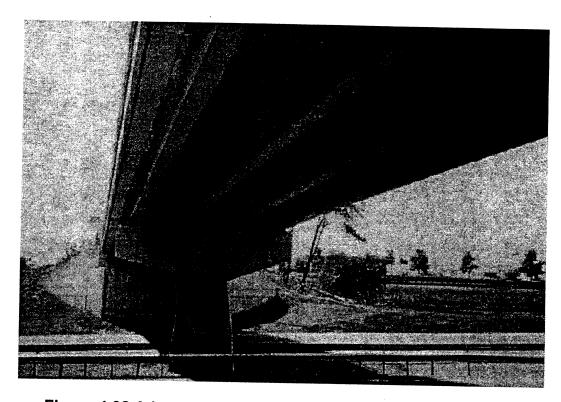


Figure 4.38 A hammerhead pier supporting a slab-on-stringer superstructure (Tonias, 1995)

4.9.1.2Column bent

A column bent pier consists of a cap beam and supporting columns in a frame type structure. Column bent piers represent one of the most popular forms of piers in use in highway bridges. This popularity is an outgrowth of the extensive use of column bent piers during the development of the U.S. Interstate system.

The column bent pier is supported on either spread footing or pile foundations and made of conventionally reinforced concrete. Like hammerhead piers, the supporting columns can be either circular or rectangular in cross section, although the former is by far more prevalent. The choice of column bent piers, like that of the hammerheads, should be somewhat judicious. For moderate clearance structures with plenty of room for underpass traffic, the column bent pier provides a very attractive solution.

4.9.1.3 Pile bent

The pile bent pier is a variation on the column bent pier with the supporting columns and footing replaced with individual supporting piles. The end piles are generally equipped with a batter in the transverse direction. Pile bent piers are extremely popular in marine environments where multiple, simple span structures cross relatively shallow water channels. Some maintenance problems generally associated with this type of pier, however, are deterioration of exposed piles, impact with marine traffic, and accumulation of debris. When provided with adequate protection against these adverse conditions, however, pile bent piers represent an economical solution for many bridges.

4.9.1.4 Solid wall

A solid wall pier (also known as a continuous wall pier) as its name would imply, consists of a solid wall which extends up from a foundation consisting of a footing or piles. The top of the wall is equipped with individual pedestals upon which the superstructure rests. Solid wall piers are often used at water crossings since they can be constructed to proportions that are both slender and streamlined. These features lend themselves well toward providing a minimal resistance to flood flows (Figure 4.36).

4.9.1.5 Integral pier

An integral pier has a pier cap to which the superstructure's primary members are rigidly connected. This type of pier is not altogether common and generally confined to special structures, particularly when tight vertical clearance constraints pose a problem.

4.9.1.6 Single column

An obvious advantage of these types of piers is that they occupy a minimal amount of space. Single column piers, like solid wall piers, are often tapered or provided with a flare so that the top of the column is wider than the base. This type of pier is extremely attractive when combined with prestressed concrete box type superstructures by providing an open and free-flowing appearance to traffic passing underneath the structure.



Figure 4.39 Solid wall pier to be replaced by a single column (Tonias, 1995)

4.9.2 Rehabilitation and maintenance

The principal causes of deterioration to a bridge pier are very similar to that of an abutment. Piers suffer from many of the same types of problems, which affect other substructure components at the ends of a bridge. Cracking, surface deterioration and stability problems compounded by the adverse effects of such physical conditions as differential settlement and scour are factors which are common to any substructure component. A pier, however, because of its location at intermediate support points, is often more prone to some of the problems

listed above than an abutment.

Some of the more common problems are the following:

- scour,
- collision with underpass traffic
- collision with ice floes

Piers are also more susceptible to overstressing than abutments, because a pier must support two spans rather than one span. While there are differences between the two substructure components in terms of deterioration magnitude, many of the problems faced by piers and abutments are the same.

Scour

When scour occurs at a specific localized point in the channel, such as a pier, abutment drainage structure, or some other obstruction, it is known as local scour. Local scour is evidenced by turbulence around piers, which erodes material from under the foundation. If scour takes place over a large area of the channel it is known as general scour. General scour occurs over a long period of time and is initiated by an alteration in channel flow patterns. General scour is not caused by man made constrictions, but rather results from a change in the supply of sediment to a large area. Scour, which results from a reduction of the cross-sectional area of a channel due to the placement of an obstruction such as highway bridge or drainage structure, is known as constriction scour. Since the cross-sectional area is decreased by the constriction, a higher water velocity results. This higher velocity will typically occur at the bridge location, increasing the potential for scour damage.

Scour rehabilitation and maintenance

If a pier is affected due to adverse scour conditions, the problem can be corrected by either changing the structure of the foundation, or replacing the material which has been washed away.

The first solution typically involves altering the foundation. This can be accomplished by enlarging the footer, strengthening or adding piles, or providing a sheet-piling barrier around the pier foundation. Replacement of material generally involves the placement of erosion resistant material such as riprap or broken concrete around the pier (or abutment) to offer a barrier to scour.

Obviously the solution selected will depend greatly on the scope of the problem (i.e., how severe is the scour) as well as available funds and material availability. Due to limited resources, is may be feasible to provide only temporary protection to a pier. When this is the case, replacement of the washed away material is the generally accepted solution. For temporary repairs, the soundness of the stone used is generally not important. Another factor, which could affect the method used, is the size of the water channel and other environmental considerations (i.e., impact on downstream conditions). The methods for either replacing washed away material or changing the structure to account for current water channel conditions are presented in the following:

Replacement of material

The replacement of washed away material can be made with broken stone or concrete. The size of stone used to protect against scour is dependent upon the characteristics of the bridge site. Some features, which are used to determine the size of stone to protect piers, are:

- type and orientation of pier or abutment.

- geometry and physical properties of stream,
- size and location of scour hole,
- local climate and environment, and
- material and vegetation of stream bed and banks.

The bridge crossing a stream (Figure 4.36) shows the type of stone used to protect substructures from scour. Because of the relatively large size of some of these stones, care must be taken by the maintenance personnel in placing them around a pier. Careless dumping or riprap can often damage the pier itself. Another concern is an uneven placement of riprap, which can lead to stability problems. For this reason, riprap should be placed in uniform lifts. If concrete is used as a repair material for scour, it will require either underwater placement or dewatering at the pier location. The latter method requires that water be removed from the pier location so that the concrete can be placed. This method has the inherent advantage of providing a dry environment for placement of the concrete.

Dewatering can be accomplished through the construction of a cofferdam around a pier using sheet piling. The driving of sheet piling under a bridge is difficult because of potential interference with the bridge superstructure overhead. Placement of the cofferdam may only be possible by removing a span. Another method for dewatering a pier is to divert channel flow away from the bridge. This method, however, could potentially create adverse effects at other points along the channel and will require a thorough investigation and analysis to ensure that no associated damage takes place as a result of the diversion.

There are a variety of methods available for underwater placement of concrete. Some of the more popular methods include:

- tremie,
- pumping,
- underwater bucket, and

bagged concrete.

A tremie utilizes gravity flow to place concrete underwater. A tremie is a tube, which is equipped with a discharge gate at one end and a hopper at the other. With the discharge gate closed, the tremie is filled with concrete and then submerged beneath the water to the location-requiring repair. The hopper must be continually filled with concrete and the discharge end submerged during the pour so that the tremie does not become filled with water. Pumping uses the same basic concept except the concrete is pumped through the tube. This provides for greater control in placing the concrete.

Underwater buckets, as the name would imply, consist of lowering buckets of concrete to the location-requiring repair. The bucket may be covered or uncovered; however, if it is uncovered, care must be taken during the placement operation. Bagged concrete consists of concrete, wrapped in a burlap bag, which is then placed around the repair location. A certain degree of bonding between adjacent bags takes place as a result of concrete seeping through the pores in the burlap bags. Some bagged concrete consists of dry concrete, which is placed and then wetted.

CHAPTER 5

SUMMARY

5.1 Summary

This study presents the findings of a comprehensive assessment of conventional methods of condition evaluation and repair of concrete bridges. Case studies involving different techniques practiced in the US and Europe are presented in the report.

5.2 Concrete evaluation and deck investigation

The causes of concrete deterioration are discussed in terms of the permeability of the concrete. The ultimate durability is affected by weathering and exposure to reactive aggregates, corrosion, water containing sulfates, leaching and mechanical ware. The steps identified by the US Bureau of Reclamation for standard concrete repairs are presented in this study. Typical detailed field investigation of existing bridge decks is described with emphasis on corrosion of epoxy-coated reinforcement in areas of cracking and insufficient concrete cover.

5.3 Superstructure repair

The repair techniques for superstructures are presented including patching, crack injection, deck overlays, sealers, expansion joints, prestressed concrete bridge girders, bearings and case studies.

The selection of materials for patching will be influenced by a) compatibility of the material to the original concrete, b) environmental considerations, including aesthetics, c) cost effectiveness, d) expected service life, e) availability, and f) familiarity of the contractors with the material under consideration. Most materials used for deep repair use portland cement binders and proportioned

aggregates. Durability for these materials can be increased using microsilica, latex, or admixtures that reduce permeability. Deck patches have a relatively short service life, because they do not address the corrosion of the reinforcing steel, but address only the disintegration due to spalling and delaminations.

Relatively wide, dormant cracks in bridge decks are repaired through gravity fill polymers such as high molecular weight methacrylate and low viscosity epoxies. Narrow cracks that are dormant maybe effectively sealed by epoxy injection. Both narrow and wide cracks that are dormant maybe repaired by routing and sealing, which is the simplest and most common technique for crack repair. Moving cracks can be repaired by flexible sealing.

Overlays used to restore the deck-riding surface to as-built quality and increase effective cover over the reinforcing steel include latex-modified concrete (LMC), low-slump dense concrete (LSDC), hot-mix asphalt concrete with a preformed membrane (HMAM), microsilica concrete, polymer overlays, asphalt concrete and high-early strength hydraulic cement concrete.

Only penetrating sealers, silanes and siloxanes are recommended for rapid deck surface protection. Detailed descriptions of the rehabilitation procedures for several joint types are presented in the study. Open joints are seldom-used in new bridge structures and are often worth replacing by other types during rehabilitation. The fieldformed sealers usually have limited service life because of poor installation conditions and workmanship encountered in the field. Preformed sealers are somewhat newer and hence have a shorter record of proven service than fieldformed sealers. An important advantage of these sealers is quick installation time and less interruption to traffic. Sliding plate joints are quite frequently used in the rehabilitation of existing deck joints.

Finger plate joints continue to be a popular option in deck joint rehabilitation as they are able to accommodate relatively largely movements. Sawtooth joints are still used in new bridges and considered as an alternative in deck joint replacement, where total movements in the range of three inches (76 mm) need to be provided. Performance and useful life of a neoprene compression seal in a new or rehabilitated joint depend primarily on the quality of the installation and the correct choice of the seal size and the material. Strip seal joints are popular in deck joint replacement, because of the locking nature of the seal. Sheet seals represent one of the possible choices for deck joint replacement in existing medium span bridges. Plank seals continue to be an alternative for the replacement of the existing joints in medium and long span bridges. The modular joint is usually the recommended choice when an existing joint with a large movement needs replacement or upgrading.

Longitudinal external post tensioning can be applied to damaged concrete girders. The strength loss can be regained by external post tensioning, internal splicing and with the use of a metal sleeve splice. Load capacity analyses are illustrated for post tensioning with splice types, post tensioning with metal sleeve splice and post tensioning with internal strand splices.

Regular bridge bearing maintenance should be directed towards keeping the bearings clean and protecting them from water, salt and debris. Corrective maintenance often entails complete bearing replacement.

Several case studies are presented for different repair procedures including patching, sealers, joint-armored compression seals, finger plate joint, repair of prestressed concrete girders, prestressed strand splicing and renewal of bearings.

5.4 Substructure repair

When scour damage to a substructure element is identified, efforts must be made to reestablish bearing and protect the substructure unit from further scour damage. Concrete in bags may be used either to armor the foundation material from further scour or as a form for placing concrete if it is necessary to restore

foundation bearing. Flexible nylon tube forms filled with grout or concrete can be used to fill scour pockets under substructure units. Scour voids can also be filled with prepacked, open graded concrete contained by forms and injected with cement grout through pipes. Sheet piling or corrugated metal can be used as formwork to retain either concrete or riprap for repair of scour damage.

The solution to vertical cracking in an abutment backwall initiated by differential settlement of the abutment is to provide adequate drainage of the abutment by an under-drain system. Repair of abutments damaged by surface deterioration is similar to that for concrete decks. Remedial patches in a deteriorated bridge seat will not create an adequate bearing surface; then complete removal and replacement of the bridge seat will be required.

Grouting using a variety of epoxies and concretes with modifiers or admixtures can be used for underwater resurfacing of large areas of deterioration. These epoxies and concretes may be used together with pile jackets or formwork, and may or may not include sandblasting of the reinforcing steel. Jacketing is used for restoring or increasing the section of an existing compression member by encasing it in new concrete.

When a pier is affected by adverse scour conditions, the problem can be corrected by either changing the structure of the foundation or replacing the washed-away-material. The replacement of washed-away-material can be made with broken stone or concrete. One of the methods-tremie, pumping, underwater bucket and bagged concrete can be used for underwater placement of concrete.

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