



Government of the People's Republic of Bangladesh
Local Government Engineering Department (LGED)

BRIDGE MAINTENANCE MANUAL



December, 2025



Message

It is a matter of great satisfaction that the **Bridge Maintenance Manual** has been developed under the Program for Supporting Rural Bridges (SupRB), jointly funded by the Government of Bangladesh (GoB) and the World Bank. This Manual is an important milestone in LGED's overall **Bridge Asset Management Framework**, designed to guide officials in performing essential maintenance activities for rural bridges and culverts.

The Manual provides clear directions to protect investments in bridge structures, maintain the service levels of the rural road network, and extend the life of these assets. It offers practical guidance on selecting appropriate types of maintenance based on inspected defects, thereby ensuring durability and serviceability of bridge structures.

I am confident that this Manual will greatly benefit LGED officials at Upazila, District, and program levels in developing and operating bridge and culvert maintenance programs. It will also strengthen institutional knowledge and capacity for sustainable asset management.

LGED reaffirms its commitment to safeguarding infrastructure investments and ensuring that bridge asset management remains efficient, accountable, and resilient.

A handwritten signature in black ink, appearing to read 'Kazi Golam Mustafa'.

(Kazi Golam Mustafa)

Chief Engineer

Local Government Engineering Department



Message

It is a great pleasure to present the **Bridge Maintenance Manual**, developed under the SupRB project. This Manual represents a significant advancement in LGED's bridge asset management practices, providing systematic guidance for maintenance planning and operations.

The Manual emphasizes the fundamentals of bridge and culvert maintenance, offering step-by-step directions for selecting appropriate maintenance types based on inspection results. It has been designed to complement the RuBIMS database, software, and Mobile App, making bridge asset management more efficient, technology-driven, and accountable.

This Manual will serve as a practical tool for contractors, engineers, and LGED officials at all levels, helping them to adopt standardized practices, minimize risks, and ensure the durability and serviceability of bridge structures. It will also contribute to the future development and upgrading of LGED's Bridge Asset Management Policy.

I extend my sincere appreciation to the World Bank Task Team and subject matter experts for their valuable comments and feedback during the preparation of this Manual. Together, we reaffirm our commitment to building infrastructure that is not only strong and resilient but also safe, responsible, and sustainable.

(Md. Belal Hossain)

Additional Chief Engineer &
Project Director
Program for Supporting Rural Bridges (SupRB)

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ACRONYMS AND ABBREVIATIONS

AAR	Alkali Aggregate Reaction	NDE	Non-Destructive Evaluation
AASHTO	American Association of State Highway and Transportation Official	NDT	Non-destructive Testing
ABC	Articulated Concrete Block	OBPA	Ogdensburg Bridge and Port Authority
AC	Asphalt Concrete	OHS	Occupational Health and safety
ACI	American Concrete Institute	pH	Potential of Hydrogen
ASR	Alkali Silica Reactivity	PCC	Portland Cement Concrete
ASTM	American Society for Testing and Materials	PM	Preventive Maintenance
APJ	Asphaltic Plug Joints	Psi	Pounds per Square Inch
BMM	Bridge Maintenance Manual	PVC	Polyvinyl Chloride
CE	Carbon Equivalent	QC	Quality Control
CFS	Carbon Fiber Sheet	QA	Quality Assurance
CS	Condition State	RAP	Repair Application Procedures
CP	Centipoise	RSDMS	Road and Structure Database Management System
DD	Damage Degree	RuBIMS	Rural Bridge Information Management Systems
DFT	Dry Film Thickness	SSD	Saturated Surface Dry
DTI	Direct Tension Indicators	SHRP	Strategic Highway Research Program
FHWA	Federal Highway Administration	TSZ	Thermally Sprayed Zinc
FRP	Fiber Reinforced Polymer	UPE	Ultrasonic Pulse Echo
GC	Growth Center	UPV	Ultrasonic Pulse Velocity
GDP	Gross Domestic Product	USW	Ultrasonic Surface Waves
GPR	Ground Penetrating Radar	USACE	United States Army Corps of Engineers
GPS	Global Positioning System	UV	Ultra Violet
HS	High Strength		
HDPE	High Density Polyethylene		
IE	Impact Echo		
IT	Infrared Thermography		
kPa	Kilopascal		
LGED	Local Government Engineering Department		
MPa	Mega Pascal		
MDOT	Michigan Department of Transporta		
MIO	Micaceous Iron Oxide		
MSDS	Material Safety Data Sheet		
MT	Magnetic-Particle Testing		

CHAPTER I: INTRODUCTION

1.0 Introduction

Local Government Engineering Department (LGED) of Bangladesh, one of the largest technical and infrastructure development organization in the public sector, is responsible for the development and maintenance of rural infrastructure in Bangladesh including the construction, maintenance, and management of rural roads and bridges. This document reviews the existing bridge and culvert maintenance guidelines of LGED and provides necessary upgrades to the same for more comprehensive and functional operation of maintenance activities in the field levels.

Bridges and culverts are the integral element of the Rural Road transportation system, form a vital and vulnerable link in connecting the Growth Centers (GC), Rural Markets, other small economic and services centers of Rural Bangladesh. Any damage caused to the same can not only affect the rural road network connectivity but also causes a major impact on the rural economy which can result in a slower GDP growth. Despite its importance, however, bridge and culvert are often the most neglected components of the road maintenance program which accounts only 4% of the total road maintenance fund. Demands on limited resources, especially competitive roadway priorities for increased capacity such as rehabilitation and widening, and improved riding surfaces, often result in deferred maintenance for bridges. The consequences are obvious – bridges are deteriorating far faster than they are being repaired. Without adequate attention, many require replacement or closure long before they are obsolete, further adding to the demand for limited funds, impacting safety, and discouraging both users and transportation providers.

Bridge construction requires a substantial amount of investment and its use involves public safety. A bridge failure can be catastrophic. It can cause injury or death and can be very expensive to restore. Since bridges cannot resist indefinitely all the natural forces and hazards including time-related degradation of materials, these structures usually have a limited-service life. The bridges have to be maintained in order to maintain certain level of services, prevent premature failure and to extend the service life.

1.1 LGED Bridge Maintenance Manual

LGED Bridge Maintenance Manual (BMM) provides general guidance on the preventative Maintenance and preservation of Rural Bridges and Culverts. This Manual will provide guidance for maintenance and repair activities of Structural elements of Sub-structure and Super-structural elements. BMM will also provide guidance on the treatment of defects of non-structural elements identified during Inspections. The scope of this Manual does not include strengthening as this is dealt-with on a case-by-case basis.

The BMM comprises of chapters on Overall Maintenance Concepts, General Maintenance, Preventative Maintenance and Repair and Rehabilitation of Bridges and Culverts. Also, there are Appendices on Bridge components and their durability as well on Case studies.

1.2 Background

This BMM should be used in conjunction with the Rural Bridge Inspection and Condition Assessment Guidelines. The BMM and the Rural Bridge Inspection and Condition Assessment Guidelines are intended to provide consistent guidance on the identification and treatment of different defects.

Minor Maintenance, Major Maintenance and Rehabilitation are complementary operations and are essential components of bridge asset management of LGED. Minor Maintenance must be cyclic activity which is repeated over the life of the structure. It is preventative in principle and is generally straightforward, routine and repetitive. Major Maintenance and Rehabilitation are non-cyclic and infrequent activity. If repeated, its scope is likely to change. Major Maintenance/Rehabilitation is remedial in nature – i.e. it is intended to restore the structure to its original condition and is likely to be a more complex and costly operation than minor maintenance.

Inadequate minor maintenance (Preventative) can result in more frequent and costly repairs. It is essential, therefore, to implement systematic and regular preventative and minor maintenance programs to minimise the frequency and cost of repairs. Preventative Maintenance is the recurrent activity which is required to preserve a structure so that it continues to perform its function. It involves the early repair of small, less serious defects which prevents long-term deterioration that would otherwise be more costly to repair.

The Inspection Manual provides guidance on identification of structural and non-structural defects. Structural defects should not be repaired before obtaining structural engineering advice. If structural defects are suspected these should be assessed by a structural engineer.

Guidance in the BMM is drawn from standard practices adopted by a variety of roads management authorities, and reflects the combined experience of major road authorities around the globe.

1.3 Objectives of this Manual

The BMM is intended to provide guidance to LGED Asset Managers regarding the maintenance and repair of existing Rural bridge assets to ensure that bridges are in a safe condition.

The BMM is intended as a reference for maintenance and repair of Rural bridge structures. It addresses the most common types of bridge and their modes of deterioration and defects and outlines common treatment methods for maintenance and repair of those defects. These procedures may not cover all circumstances and are not intended to rule out alternative maintenance procedures. Specialist technical advice should be sought from technical experts on Bridge Structure and Bridge Maintenance prior to use of alternative procedures.

Professional and LGED Officials involved in Bridge Maintenance management should adhere to the requirements of this BMM when selecting the appropriate maintenance and repair method and to the relevant LGED Standard Sections for procedures, material selection and workmanship. The BMM may be used for all structures; however, additional care should be taken to ensure maintenance or repairs are completed in a manner that is sensitive to the structure in terms of colour, materials, and structural form. If maintenance and repair methods are not covered in this BMM, the maintenance professional shall seek direction from LGED.

The BMM does not address environmental impact of maintenance activities and the maintenance professional must consider the safety and environmental impacts and incorporate their remedial solutions in the maintenance package. If required a safety and environmental risk assessments must be conducted and appropriate method statements prepared for all activities described in the LGED Standard.

1.4 Use of the Manual

The BMM must be read in conjunction with the Inspection Manual to provide LGED Asset Managers, Inspectors and Maintenance Management Officials with a full understanding of the possible maintenance and repair treatments appropriate to the observed defects.

The BMM addresses a selection of common treatments. It does not include all treatments that might be found necessary. In cases where the BMM is silent, further advice shall be obtained from LGED Asset Manager and or technical experts.

The sketches and diagrams are general and indicative and do not provide full details of the repairs.

1.5 Bridge Maintenance Planning

The key to the successful execution of any bridge maintenance project is the proper planning and scheduling of the work. A relatively brief amount of time spent doing quality planning will have a positive impact on performance and will create overall time savings. The planning and scheduling must be done prior to beginning the project and continue during the execution.

1.5.1 Long-Term Planning

1.5.1.1 Introduction to Long-Term Planning

Long-term planning for bridge maintenance is typically 3 to 5 years, but could be longer.

The dynamic nature of bridge maintenance needs requires that longer term plans remain flexible to adapt to unidentified urgent needs that may require the crew's attention. The continual identification of new bridge needs often necessitates that longer term planning be based on historical expenditure information to determine future budget staffing and other resources needs. Long-term planning involves the following steps:

- Needs are identified for all the bridges that are included.
- Preliminary cost estimates are developed, and a budget is prepared. After the budget is approved, a more detailed estimate is developed to determine the manpower and resource requirements to accomplish the work, and this is compared to the available manpower and resources.
- A decision is made to contract out the work or assign it to in-house crews depending on LGED's practices. This decision may be based on available staff and workload or on the need for special equipment or skills and financial threshold.

Priorities for work drive long-term schedules:

- The urgency of the work
- The year the work can best be performed
- When approvals or permits can be obtained
- When necessary, support, equipment, and materials are available

Realistic scheduling improves efficiency. Workers should be challenged. However, if too little time is allotted to a job, shortcuts and omissions may undermine the quality of the repair work. The long-term planning effort provides general guidance regarding what to do, when to do it, and what it will take to get it done.

Once the long-term planning has identified a set of work activities to be performed by the bridge maintenance crews, more detailed planning and scheduling can begin. The details of what exactly is to be done, who will do it, and what equipment and materials it will take to get the work done are included in a work order.

1.6 Reporting Bridge Maintenance Accomplishments

Once the bridge maintenance activity is complete, it is important to report the details of the repair operation into RuBIMS. It is important to document the repair because future decisions are based on the information collected from each maintenance. Information provided into RuBIMS allows managers to maintain:

- A historical record of maintenance and repair activities, quantities, and costs
- A record of regular periodic and special expenditures as a basis for developing future budgets
- A current record to establish cost/performance relationships
- A source of information used to identify trends, and the need for additional cost or work items

Reporting is the primary means of communication between field level maintenance and LGED management. Work completions and the use of resources are reported as they occur. Work completions inform management that planned and scheduled maintenance goals have been met and provide data to account for funds.

1.7 Quality Control and Quality Assurance

1.7.1 Introduction

Quality control and quality assurance programs are important parts of an agency's overall bridge maintenance program. Quality assurance establishes process and procedures that will lead to high quality maintenance. Quality control programs verify that the quality standards established are being met.

1.7.2 Quality Control / Quality Assurance Definitions

The terms quality control and quality assurance are often incorrectly used interchangeably. One basic distinction between the two is in the timing of when the quality is influenced. To properly apply these concepts, we need to first understand the difference between them as they apply to bridge maintenance.

1.7.2.1 Quality Assurance (QA)

Quality Assurance (QA) refers to the processes or procedures used to perform bridge maintenance. Quality assurance makes sure you are doing the right things and in the right way. Examples of quality assurance include process checklists, repair procedures and standards development.

Quality assurance activities are determined before production work begins. Quality assurance is also the compliance with those checklists, procedures, and standards as activities are performed while the repair is underway. QA aims to prevent inferior work with a focus on the process used to perform the repair. QA is a proactive quality process.

1.7.2.2 Quality Control (QC)

Quality Control (QC) refers to activities associated with the creation of project deliverables. Quality control is used to verify that deliverables are of acceptable quality and that they are complete and correct. In other words, quality control makes sure the results of what you've done are what you expected. Examples of quality control activities include inspection and testing process.

Quality control activities are performed after the work is complete. QC aims to identify and correct defects observed or measured in the finished product. Quality control is a reactive process.

1.7.3 Quality Management Plans

A Quality Management Plan (QMP) combines the objectives, standards, activities, and measures of the quality assurance and quality control program into a single plan to address the overall quality of bridge maintenance. The QMP defines the acceptable level of quality and describes how the maintenance crews will ensure this level of quality in their deliverables and work processes. The QMP describes the following quality management components:

- Quality objectives
- Project deliverables and processes to be reviewed for satisfactory quality level
- Quality standards
- Quality control and assurance activities
- Quality roles and responsibilities
- Plan for reporting quality control and assurance problems

1.7.4 Quality Assurance in Bridge Maintenance

A good bridge maintenance program requires that the agency ensure bridge maintenance activities are being done in the proper way. Agencies that employ quality assurance typically focus their quality assurance efforts in the following areas:

- Standards development
- Work Procedures
- Employee Training

1.7.4.1 Quality Assurance Standards

The quality assurance standards are a set of desired outcomes. These standards define quality bridge maintenance repair. For example, consider the desired quality standards for a concrete deck repair activity. A quality assurance standard for this type of repair might include the following quality standards:

- The newly placed concrete shall be solid and crack free
- The spall repaired area shall be at the same finished grade as the surrounding deck
- Acute angles shall not be permitted in a spall repair
- The defined surface friction shall be present in the repaired area

The criteria presented above establishes the expected quality outcome for a concrete bridge deck spall repair activity. The crew uses defined work procedures to achieve the desired quality in the finished repair.

1.7.4.2 Employee Training

Proper training is an important component of quality assurance for bridge maintenance activities. For example, it isn't reasonable to expect that a bridge crew member could meet the required quality standards for welding without appropriate training. Likewise, the resulting quality of a bridge painting project is likely to be below quality standards if the painters are untrained.

To help ensure that employees are able to meet the quality requirements, training standards for crew roles should be developed. The training standards should define a set of training that is required and desirable for various crew member functions. These training standards need to consider the type of work the crews will be performing and the need for specialized skills for crew members.

For example, a bridge painting crew is considered. Every employee working on the crew will require a suite of common health and safety training such as fall protection, confined spaces training and lead health and safety training. These training courses ensure that the employees are working safely but do not address the quality of the work being performed. Training courses related to the proper preparation of steel surfaces for painting, how to monitor weather during painting, and application techniques for paint ensure that the painting will meet the quality standards.

An agency with training standards defined for each role on their crew can compare the current staff training histories against the standards to identify training needs and to assign tasks in a project work plan.

1.7.5 Quality Control in Finished Products

Quality control processes are used to check or test completed work by identifying any defects observed or measured in the finished product. Quality control is a reactive process of sampling and testing to ensure conformance with established quality standards. Quality control is reactive in nature, but can be employed at the completion of intermediate phases of a larger project. For example, the QC activities associated with the placement of new paint on a bridge would certainly include some form of QC of the condition of the metal and quality of the surface preparation before the paint was applied.

It may be necessary to adjust the frequency of verification testing in accordance with the estimated number of QC tests to be performed to space out the testing dates. The rate of verification testing may also vary based on the risks involved. For example, testing structural concrete may be more frequent than testing embankment materials. QC testing may include visual inspection for workmanship and quality of the finished product, and material testing for adherence to established material specifications.

Quality control efforts require documentation of the results of the inspection, sampling or testing that is done. The agency should specify the minimum level of QC documentation that must be provided in the QMP. Materials or workmanship that do not meet the specified level of quality should be properly documented, including the nature of the non-conformance, location, extent, and disposition (e.g., removed and replaced, reworked, accepted based on engineering judgment, etc.). The documentation is also used to provide feedback for improvement of the quality assurance procedures.

CHAPTER II: MAINTENANCE OF STRUCTURES

2.1 Review of Existing LGED Bridge and Culvert Maintenance Guideline

The Individual Consultant reviewed numerous documents and materials to study the statistical records, existing practices of LGED for bridge and culvert maintenance and offered this updated bridge maintenance guideline. A very limited resource is currently available that represents the detailed maintenance activities and guidelines for bridge and culverts within LGED. In most cases, the guidelines are so generalized that they lack appropriate details to give a comprehensive understanding of the maintenance activities. Only a number of typical routine maintenance activities are mentioned in the Guideline for Implementation of Rural Roads and Culverts Maintenance Program, June 2010. Therefore, the Consultant finds it essential to review more national and international documents and servicing manuals in order to upgrade and refine the Bridge and Culvert Maintenance Guideline for LGED.

2.2 Characteristics of Well-Maintained Bridges and Culverts

Well-maintained bridges have the following characteristics:

- The operating width is the same as when the structure was constructed.
- Structural members are free from damage and corrosion and are in the same alignment as when they were built.
- The deck is free of debris and deterioration.
- All traffic control devices are in place, well maintained, and visible.
- The channel is free of erosion, and the channel and structural members are clear of debris.
- The bridge railing is in good condition, and any approach guardrail is in place and properly aligned.
- Approach paving provides smooth access to the bridge deck.
- All required non-structural elements are in place and in functioning condition.

2.3 Types of Maintenance Activities

Routine maintenance is the recurrent day-to-day, periodic, or scheduled work that is required to preserve or restore a bridge or culvert so that it can be effectively utilized as intended. It includes work to prevent damage or deterioration of a bridge or culvert that would otherwise be costlier to restore, and if left unchecked would eventually progress to structural damage. The concept of Maintenance involves the repair of small or potential problems in a timely manner so that they will not develop into expensive bridge rehabilitation or replacements, and as such is an important aspect of bridge maintenance.

Maintenance activities fall into two categories:

- Preventative maintenance, and
- Reactive maintenance.

Preventative maintenance consists of the systematic servicing of a structure on a scheduled basis, the frequency of which varies depending on:

- The type of work or activity being undertaken
- Environmental considerations
- Material type and construction type
- Age of the structure
- The volume of general traffic/heavy vehicles using the structure

Reactive maintenance is performed when the need for maintenance is deemed necessary to prevent further deterioration or development of defects. The need for this type of maintenance is normally identified during inspections or in the course of carrying out preventative maintenance.

The main reason for maintaining structures is to preserve the investment that has been made in the construction of this asset, as the cost of replacement is normally considerably higher than the initial construction cost. Similarly, the cost of program maintenance or rehabilitation is normally quite high, often many times the cost of carrying out normal servicing on a structure. This point is illustrated in the graph shown in Figure 2.1. Ensuring user safety is also an important requirement of regular servicing.

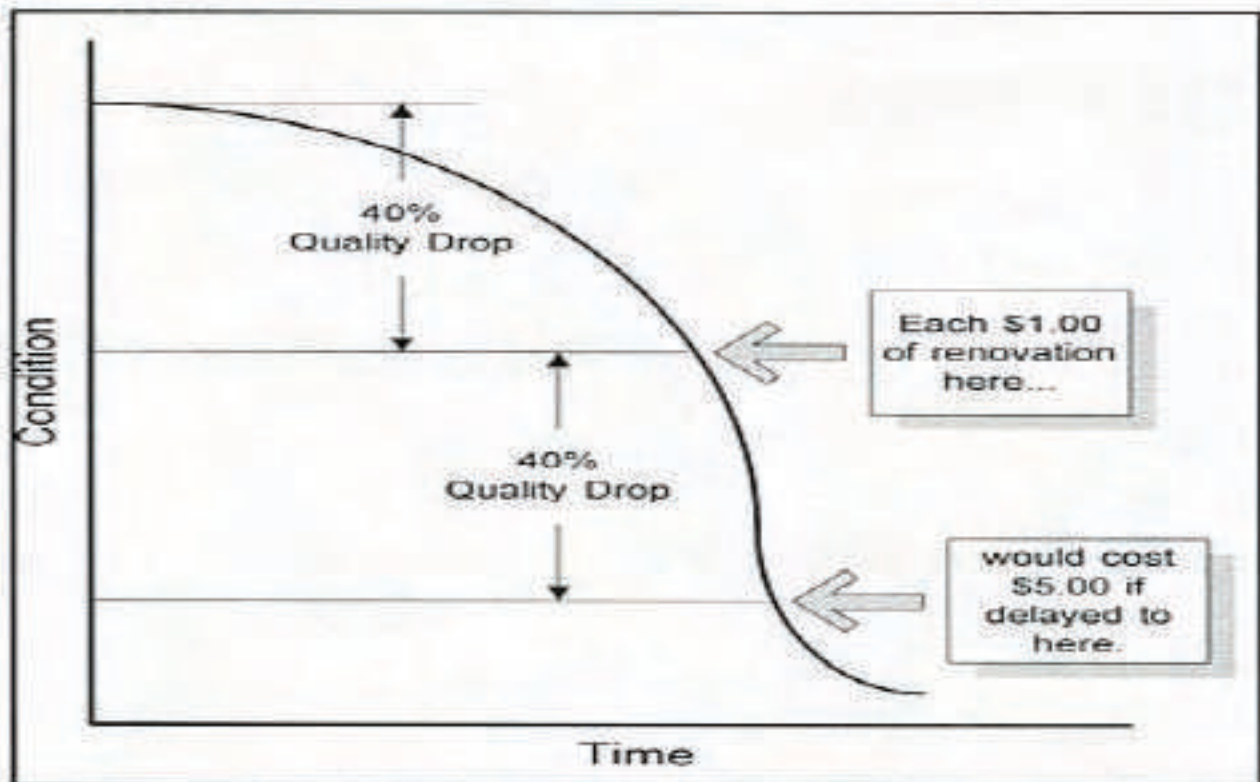


Figure 2.1: The cost of delaying maintenance (Source: FHWA Bridge Maintenance Manual)

2.4 Maintenance Requirements

LGED has a number of different types of structures on the road network constructed with a variety of materials, all of which require on-going maintenance. As these structures tend to have several common components, it is easier to describe servicing requirements in terms of these common groups rather than on the basis of a particular structure or material type.

A set of standardized servicing requirements have been prepared for the following component groups:

- Deck surface
- Bridge sub-structure
- Bridge super-structure
- Culverts
- Guardrail and bridge rail
- Waterway
- Approach embankment
- Other non-structural elements

Where possible, maintenance should be carried out prior to or in conjunction with scheduled inspections of the structure or when there is an ongoing road maintenance work adjacent to a bridge/culvert structure. This has the benefits of:

- ensuring that inspections may be conducted without unnecessary obstructions, and highlighting those defects which are beyond the scope of routine maintenance to resolve, and revealing those defects that may have been obscured by dirt or debris.
- minimizes the mobilization and traffic disruption cost.

CHAPTER III: GENERAL MAINTENANCE ACTIVITIES FOR BRIDGES AND CULVERTS

There are approximately 340,000 culvert/bridge structures within the LGED road network and accounts for total length of 1,650,000 meters. The enormous number of structures under the ownership of LGED requires regular maintenance. This guideline focuses on a set of standardized maintenance requirements that have been guided for the common components of all types of bridge and culvert, and then expanded to specific structure such as culvert and bridge in subsequent Chapters. The general maintenance items are listed as:

- Deck Surface Maintenance
- Guardrail and Bridge Rail Maintenance
- Sub-structure and Super-structure Maintenance
- Waterway Maintenance
- Approach Roadway/Embankment Maintenance
- Other non-structural elements

3.1 Deck Surface Maintenance

The main objective is the provision of clean, even and free-draining roadway and walkway, surfaces. In order to achieve this objective, the following criteria must be satisfied:

3.1.1 Drainage

The bridge surface must drain properly in order that casual water cannot accumulate on the surface. Debris on the surface and in gutters and drains prevents free drainage of the surface water and increases the risk of vehicles hydroplaning (See Figure 3.1). Additionally, casual water can leak through the deck or joints and deteriorate the saturated super-structure and sub-structure components. Accordingly, drains must be kept clear by rodding or flushing with compressed air or water.



Figure 3.1: Blocked Drainage Inlets

3.1.2 Potholes

Potholes in the surface will similarly initiate dynamic load effects up to three-fold of the static axle mass resulting in accelerated deterioration of the bridge and the departure lane of the approach embankments with the attendant accelerated deterioration of both. Additionally, surface water will saturate the deck and potentially generate a larger failure (See Figure 3.2). Accordingly, pothole patches should be affected pending a pavement rehabilitation repair.



Figure 3.2: Potholes in wearing surface

3.1.3 Cracks

Cracks in the wearing surface or over buried deck joints allow water to saturate the deck and underlying components with attendant accelerated deterioration (See Figure 3.3). These defects should be filled with a hot-melt bitumen or other approved sealant pending a permanent repair.



Figure 3.3: Cracks in the wearing surface

3.1.4 Uneven Footways

Uneven footways pose a risk to pedestrians and any discontinuities must be feathered with a cold asphalt correction course pending a permanent repair.

3.1.5 Clean Deck Joints

Debris in the joints can also puncture seals/glands and allow leakage on underlying superstructure and substructure components.



Figure 3.4: Blocked expansion joint

3.1.6 Vegetation Control

Vegetation may grow in the debris that has accumulated in the channels, drainage components or approach of poorly maintained bridges (See Figure 3.5). This will compromise the drainage of surface water and will hold casual water on the structure. The roots of some plants or trees will damage drainage and structural components.



Figure 3.5: Growth of vegetation on the wearing surface

3.2 Guardrail and Bridge Rail Maintenance

The main objective is to maintain vehicle containment and pedestrian barriers to the original design standard. In order to achieve this objective, the following criteria must be satisfied:

- a) Accident damage should be identified and made safe until a permanent repair can be effected. See Figure 3.6.
- b) Scheduling inspection and/or repairs to accident damaged or defective guardrail and bridge rail. See Figure 3.6.
- c) Sounding barriers for any loose fasteners and tightening as required.
- d) When steel members corrode, they will lose strength thus it is important that areas of spot rust are touched up routinely. See Figure 3.7 (left).
- e) Painting guardrails, kerbs, and handrails. See Figure 3.7 (right).



Figure 3.6: Accident damage of guardrails



Figure 3.7: Corrosion of steel guardrails(left), Repainting guardrails (right)

3.3 Sub-structure and Super-structure Maintenance

The objective is to maintain serviceability and durability of primary support elements and ancillary components. In order to achieve this objective, the following criteria must be satisfied:

3.3.1 Removal of Flood Debris

Flood debris or surface water deposits associated with deck joint leakage will trap moisture on the supporting piers and the accelerated deterioration of the sub-structure components will ensue. Joints must be made watertight, and the debris accumulating near the piers must be removed to eliminate further deterioration. Larger pieces of flood debris may have damaged the structure or may have the potential to do so if it is dislodged.



Figure 3.8: Pier with flow obstruction

3.3.2 Scour Prevention

Flood flow may result in scouring damage to the bed or embankments and/or promote the collection of further debris creating a serious constriction of flow with consequential damage to structure or scour damage to the bed, banks and/or approaches.

3.3.3 Vegetation Removal

Vegetation is likely to germinate in poorly maintained sub-structures, and super-structure and this could cause severe damage if trees are allowed to mature. All vegetation must be removed and uprooted.



Figure 3.9: Vegetation growth that may damage the sub-structures and super-structure

Similarly, vegetation around the foundations must be cut back, as in addition to damaging sub-structure components this may promote the collection of flood debris and tree growth may result

in damage to both sub-structure and super-structure elements. Cleared areas around sub-structures also facilitate unobstructed inspections.

3.3.4 Bearing Damage

Poorly maintained bearings will seize and the resulting "locked-in" stresses will damage the components, fasteners and bearing shelves. Accordingly, bearings must be cleaned and lubricated. If defective bearings are detected, then an extensive inspection should be scheduled.

3.3.5 Steel Member Corrosion

When steel members corrode, they will lose strength and eventually become ineffective in load bearing. Thus, in the case of the substructure, it is very important that areas of spot rust are investigated regularly and touched up routinely.



Figure 3.10: Steel member corrosion

3.3.6 Foreign Material Removal

Material trapped between deck joints or the between the deck and the ballast walls at abutments will similarly compromise design movements and result in the transfer of loads to members not designed to accommodate them. All foreign material must be removed.

The super-structure maintenance is mainly the combined maintenance of deck wearing surface, guardrails, deck joints, drainage, and vegetation control. Since the individual items are explained separately, the superstructure maintenance activities are not described as a whole.

3.4 Waterway Maintenance

The objective is to maintain channel and approach embankment integrity and the designed waterway area. In order to achieve this objective, the following criteria must be satisfied:

3.4.1 Flood Debris Removal

Larger pieces of flood debris upstream of the structure, such as logs or dead trees, may have damaged the structure or may have the potential to do so if it is dislodged. Furthermore, if the material is not removed it may result in scouring damage to the bed or embankments and/or promote the collection of further debris creating a serious constriction of flow with consequential

damage to structure or scour damage to the bed, banks and/or approaches. Consequently, this material should be removed from the site and carted to a dump location clear of the channel.

3.4.2 Fence or Barrier Removal

Fences or any kind of barriers should not be constructed across the bridge opening as these promote the accumulation of debris mats across the bridge. This should be removed in order to ensure a clear opening of the channel under the bridge.

3.4.3 Vegetation Control Adjacent to the Structure

Vegetation under and adjacent to the structure must be cut back and (where feasible) poisoned as it could potentially damage the structure and may promote the collection of flood debris. A cleared area around substructures also facilitates the unobstructed flow of water and prevent the deposition of bed materials. Vegetation shall be cleared over the structure footprint and ten meters upstream and downstream over the projected length of the structure. Any trees with limbs overhanging the roadway should be removed or lopped to maintain the stated clearance area.



Figure 3.11: Vegetation removed from the adjacent area of the structure

3.4.4 Scouring of Bed Materials

Scouring of bed material adjacent to or under substructures will compromise the stability and load capacity of the structure. Where this is found to be the case, measurements shall be taken and details forwarded maintenance officials for analysis and repair recommendations. The creek bed should be checked for a distance of 150m upstream of the bridge scours to determine if there is any fresh disturbance to the banks or channel that may have initiated the scour event. Any such disturbance should be photographed and measured and the advice forwarded to Maintenance Division with the scour details.



Figure 3.12: Scouring of bed materials

3.4.5 Deposition of Bed Materials

Deposition of bed material may occur (aggradation) and this can adversely affect the waterway area. Advice from the higher officials should generally be sought to investigate the issue if the accumulation is in excess of 10% of the waterway area of the bridge. In extreme cases, more than 20% of the waterway area reduced, then the accumulated material should be removed such that at least 90% of the design waterway is restored.

3.5 Approach Roadway/Embankment Maintenance

The objective is to maintain embankment integrity and carriageway continuity across the approach/bridge interface. At least 50 m of approach roadway and embankment should be included in the routine bridge maintenance program, with all approach roadway and walkway surfaces clean and free-draining. In order to achieve these objectives, the following criteria must be satisfied:

3.5.1 Settlement of Approach Embankment

Poorly designed and/or constructed approach embankments will consolidate and/or settle behind the bridge abutments resulting in a discontinuity at the embankment/bridge interface. The approach embankment should be maintained routinely by looking for any potential erosion, settlement or water leakage. The settled down embankment should be constructed at the same level to ensure the traffic continuity. Additionally, the approach guardrail will have to be raised by the same depth as the overlay to maintain the required standard.

Additionally, casual water can leak through the wearing surface and saturate the pavement and underlying embankment resulting in pavement failures. Accordingly, the pavement surface should be cleared of debris and drains must be kept clear.



Figure 3.13: Settlement of approach embankment

3.5.2 Scouring Damage

Blockages in a drainage channel in approach embankments can cause catastrophic scour damage and the loss of the embankment and severe damage to the abutment and first span (See Figure 3.14). Any debris or vegetation in these areas should be removed. Collected material should be carted and dumped clear of the channel. Road drainage offsets, particularly adjacent to the abutments, should be cleared and any scour damage repaired.



Figure 3.14: Scouring near the abutment (left), Scour prevention through maintenance (right)

3.5.3 Slope protection

The slope of approach embankment should be protected from flood flow and any other erosion through permanent compaction, concrete paving or using sandbags. The most feasible option to protect the slope near the structure is to protect the toe of abutment and wing wall which are more prone to erosion and have to withstand the pressure of channel flow (See Figure 3.15).



Figure 3.15: Slope Protection

CHAPTER IV: CULVERT DEFECTS AND ITS MAINTENANCE

4.1 Culvert Defects

4.1.1 Debris

Debris inside culverts, as well as at inlet and outlet ends, can degrade the hydraulic capacity and performance of culverts. An example of debris and silt buildup in a corrugated culvert is shown in Figure 4.1. Accumulations of debris can re-direct the flow of water causing scour of the stream banks, road embankment, or stream bed. Debris can also cause standing water in culverts, or conditions where material stays in contact with the culvert keeping it wet, and accelerating deterioration of the culvert materials. This is of particular concern in applications with galvanized steel. The debris may be large such as trees, limbs, woody vegetation, tires, shopping carts, or naturally occurring objects such as cobbles and boulders that have been washed down stream into or around the culvert. The debris may also be smaller grained soils, such as gravel, sand or silt. Vegetation growing at the inlet may also restrict the waterway opening.



Figure 4.1: Debris and Silt in Corrugated Culvert

4.1.2 Siltation

Similar to debris, siltation inside culverts, as well as at inlet and outlet ends, can degrade the hydraulic capacity and performance of culverts, as shown in the examples in Figure 14.6. Accumulations of silt can re-direct the flow of water causing scour of the stream banks, road embankment, or stream bed. Silt can also cause standing water in culverts or conditions where material stays in contact with the culvert keeping it wet. These situations can accelerate deterioration of the culvert materials. This is of particular concern in applications with galvanized steel. The siltation may be caused, or at least exacerbated, by larger debris, such as trees, limbs, cobbles and boulders, or vegetation. Removal of larger debris should be done in conjunction with siltation removal to reduce the potential for deposition of more silt.

4.1.3 Scour

Scour is the lowering of the stream bed due to the removal and transportation of stream bed material by flowing water. Scour can occur at the inlet end, outlet end, or both ends of a culvert. Scour holes may form near the culvert inlet due to the flow striking against the upstream embankment prior to entering the culvert, or as the flow accelerates upon entering the culvert if the culvert is forming a constriction in the stream. Local scour at the outlet may occur when high velocity water exits the culvert.

The stream may also be subject to general scour. General scour is the gradual degrading or lowering of the stream bed over all, or part of its length. General scour can be a concern if the area of degradation is migrating upstream toward the outlet of the culvert. This can ultimately lead to the culvert being undermined. Conversely, general scour upstream of the culvert can subject the culvert to siltation and debris accumulation.

4.1.4 Leaking Joints

Joints serve several purposes, including:

- Maintaining the flow of water through the culvert
- Keeping soil, and water from infiltrating into the culvert
- Maintaining alignment
- Preventing the sections from pulling apart

Evidence of poor joint performance includes:

- Separated or misaligned joints
- Evidence of water infiltration or exfiltration
- Soil infiltration
- Seepage at joints
- Open joints
- Sink holes in the embankment/pavement surface over the joints

4.2 Corrugated Metal Culvert Defects

4.2.1 Damaged Coatings

Steel culverts are typically coated to protect the steel from corrosion. The coatings routinely used include galvanizing (zinc) or aluminizing (aluminum). In addition, galvanized pipe can be coated with polymer or asphalt to extend its life. In all cases, damage to the coatings will accelerate the onset of corrosion at the damaged coating location, which will reduce the lifespan of the culvert as corrosion advances.

Aluminum alloy culverts are typically uncoated, because aluminum alloys develop a protective oxide barrier layer when installed in environments where pH is between 4.0 and 9.0. For comparison purposes, distilled water has a pH of 7.0, baking soda has a pH of approximately 9, and tomatoes have a pH of around 4.

4.2.2 Dents and Localized Damage

Pipe wall damage such as dents, bulges, cracks, tears, and creases can occur during installation and backfill of culvert. If the damage is extensive the structural integrity of the culvert may be

compromised, requiring rehabilitation or replacement. In many cases, localized damaged is not critical to the long-term durability of the culvert.

4.2.3 Corrosion

Corrosion occurs in soils or waters that have corrosive properties (high or low pH) or an abrasive environment (where abrasion removes the protective coatings on the culvert increasing vulnerability to corrosion). Acidity or Alkalinity can occur from pollutants, such as acid rain, but more typically it is due to the underlying geology of the area. Corrosion can lead to section loss, perforation, and ultimately structural failure. Some agencies may choose to utilize culvert materials of heavier gauge, or with additional coatings to obtain the necessary design life given a more corrosive environment. However, when corrosion progresses to the point where the culvert is failing (leaks, perforations, structural deformations) repairs, rehabilitation, or replacement options should be explored. During selection of the repair option, the pH of the environment should be considered. Examples of culvert corrosion are shown in Figure 4.2 and Figure 4.3.



Figure 4.2: Corrosion in Plate Arch

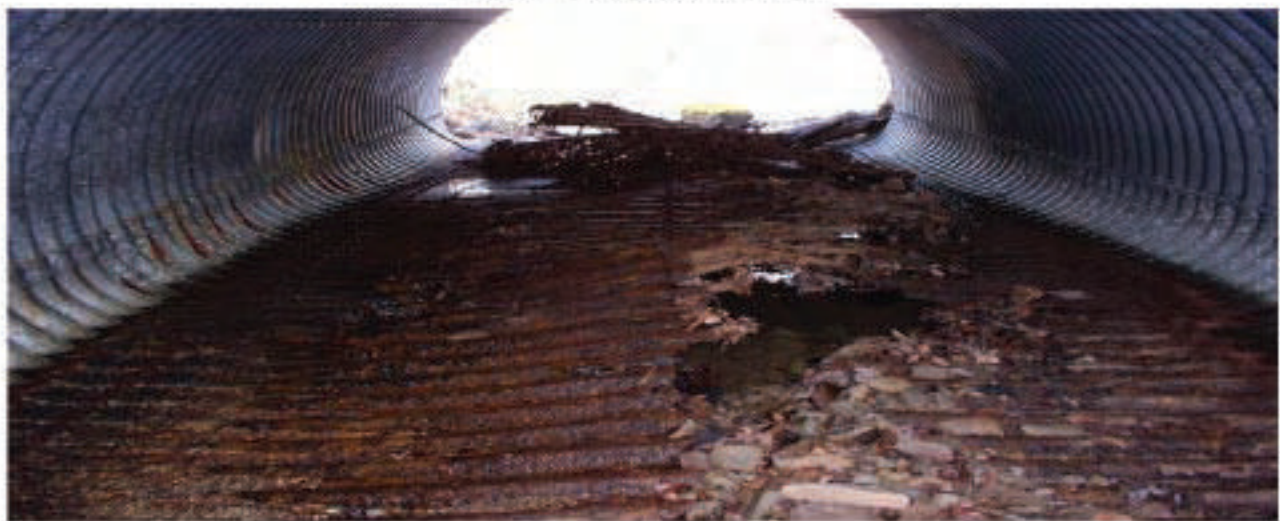


Figure 4.3: Corrosion Failure of Culvert Invert

4.3 Concrete Culvert Defects

4.3.1 Cracks

Cracks in concrete culverts may occur due to a number of factors. The contributing factors can often be identified by the type of crack in the concrete culvert. Types of cracks occurring in concrete culverts include:

- Circumferential cracks near joints of round concrete culverts. These cracks usually indicate that there has either been some movement or settlement of the pipe sections, the gaskets were improperly installed, or the pipe was damaged during installation.
- Longitudinal Cracking of round culverts at the invert, crown and spring line. These cracks are usually flexural cracks caused by loss of side support (Figure 4.4 left).
- Longitudinal Cracking of round or pipe arch culverts parallel to the invert. These cracks are usually shear cracks due to loss of support at the haunches (Figure 4.10 middle).
- An example of good side support is shown in Figure 4.4 (right).

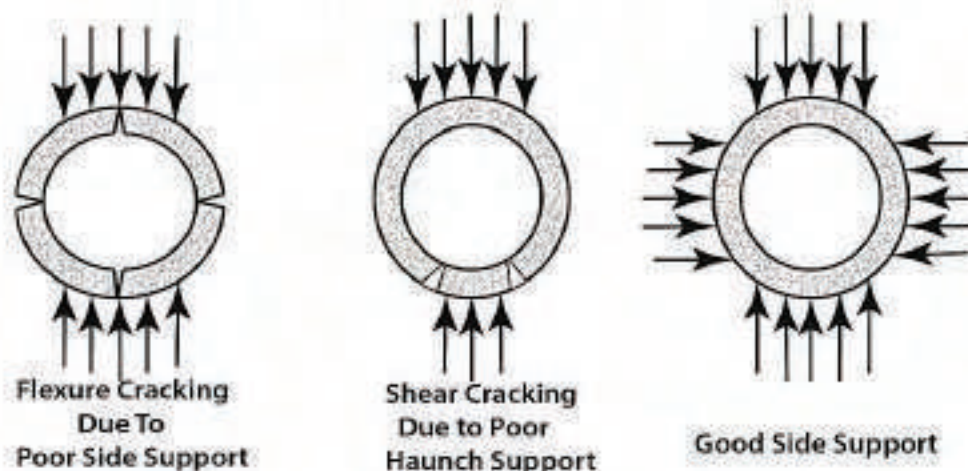


Figure 4.4: Longitudinal Cracks in Concrete Pipe

Transverse, or circumferential, cracks in round or pipe arch culverts often indicate that the pipe is in flexure. Cracks in the bottom of the pipe indicate a loss of support at midsection, whereas cracks in the top of the pipe indicate a point load was applied to the pipe at or near the location of the crack.

Cracks in box culvert and arch concrete can indicate differential settlement in the case of transverse cracks in the top, or invert and differential earth pressures in the case of the walls. Longitudinal cracks in box culverts that are not a result of reinforcing corrosion can often be attributed to flexural or shear distress in the concrete.

4.3.2 Spalls

A spall is a fracture of the concrete parallel to or inclined to the surface of the concrete. Spalls often occur along the edges of cracks, near joints and corners, and above reinforcing steel that has corroded.

4.4 Masonry Culvert Defects

4.4.1 Cracks

Individual bricks, stones, or blocks may be cracked, broken, crushed, or missing. Sometimes a seam or crack will follow a mortar line through a masonry unit, which then opens up a seam or crack in the individual unit. Cracks may also develop parallel to the surface of the masonry unit resulting in spalls or loss of section. Cracks may also be caused by thermal movement, or moisture related causes such as water infiltration. Cracks may also indicate that the culvert is subject to differential loading or stresses, or the loss of foundation support.

4.4.2 Loss of Mortar

In most masonry construction, mortar is used to cement the masonry units together. Loss of mortar may lead to displacement or loss of masonry units which can compromise the structural integrity of the culvert (Figure 4.5). Loss of mortar may be caused by moisture, thermal expansion and contraction, or loss of soil support. Loss of mortar may lead to water and soil infiltration.



Figure 4.5: Mortar Loss in Bottomless Masonry Arch

4.4.3 Soil and Water Infiltration

Cracking or loss of mortar may lead to water and soil infiltrating into the culvert. This can lead to the loss of soil covering the culvert, distresses in the surfaces above, and potential structural failure of the culvert from the loss of uniform soil cover. Loss of mortar may be caused by moisture, thermal expansion and contraction, or loss of soil support. In cold climates, water infiltration can also lead to freeze thaw damage to the mortar and masonry units.

4.4.4 Loss of Foundation Support

Like any other type of culvert construction, support from the surrounding soil is important to the proper structural performance of the culvert. Loss of soil support may be caused by weak soils, inadequate compaction of bedding or back fill, erosion or scour, moisture infiltration or frost action. Indications of foundation support loss include cracking, deformation, loss of mortar, settlement, or uplift.

4.5 Preventive and Basic Maintenance of Culverts

In this section, preventive maintenance refers to the on-going scheduled activities aimed at maintaining and extending the service life of culverts. Scheduled maintenance activities become important to proper culverts performance. Many agencies have a culvert inspection program, which assesses conditions and identifies maintenance and repair needs. The inspection cycle for culverts is based on the agency, but can range from 1 to 10 years. The level of detail can vary from inspection-to-inspection and can also vary within various agencies. Often culverts are checked for debris or scour issues annually, or after major storm events, whereas in-depth structural inspections happen less frequently.

4.5.1 Scour Repair

Local scour occurs at the inlet and outlet ends of culverts due to changes in water velocity as water enters and exits the culvert. The strategies for repair vary depending on the severity of the scour, the likelihood of continued scour, and the risk of damage caused by scour.

In the case of a scour hole at the outlet of a culvert which is not threatening the culvert, its appurtenances, or adjacent properties, no action may be warranted.

In the case of scour holes that could potentially threaten the culvert, filling the scour hole with riprap (i.e., stone fill) of the appropriate size based on stream flows and velocities is an appropriate option. An example drawing for scour repair using riprap is shown in Figure 4.6.

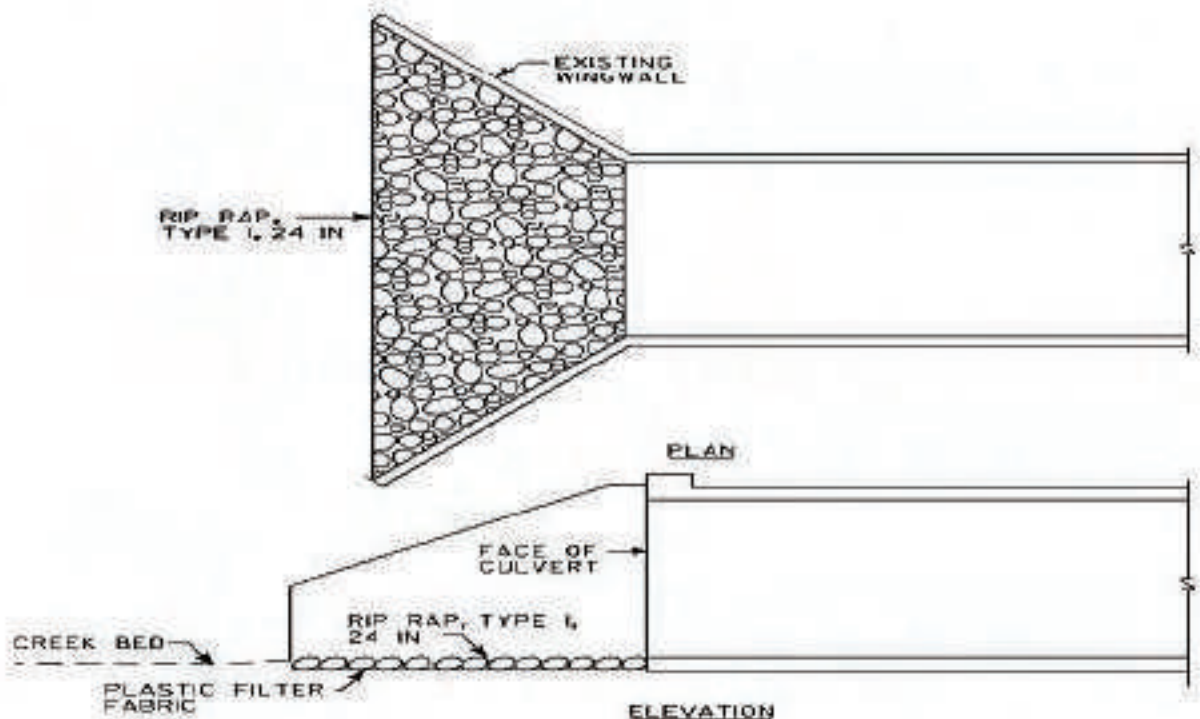


Figure 4.6: Riprap at Culvert Entrance/Exit for Scour Repair (Courtesy of GADOT)

In cases where scour has advanced underneath the culvert, a repair will not only need to be made to the scour hole outside the culvert, but also to the undermined area. One method of repairing the

undermining uses self-leveling concrete (flowable fill). An example of this technique is shown below in Figure 4.7, with cross section views shown (left), plan views shown (right), existing condition (top) and final condition (bottom).

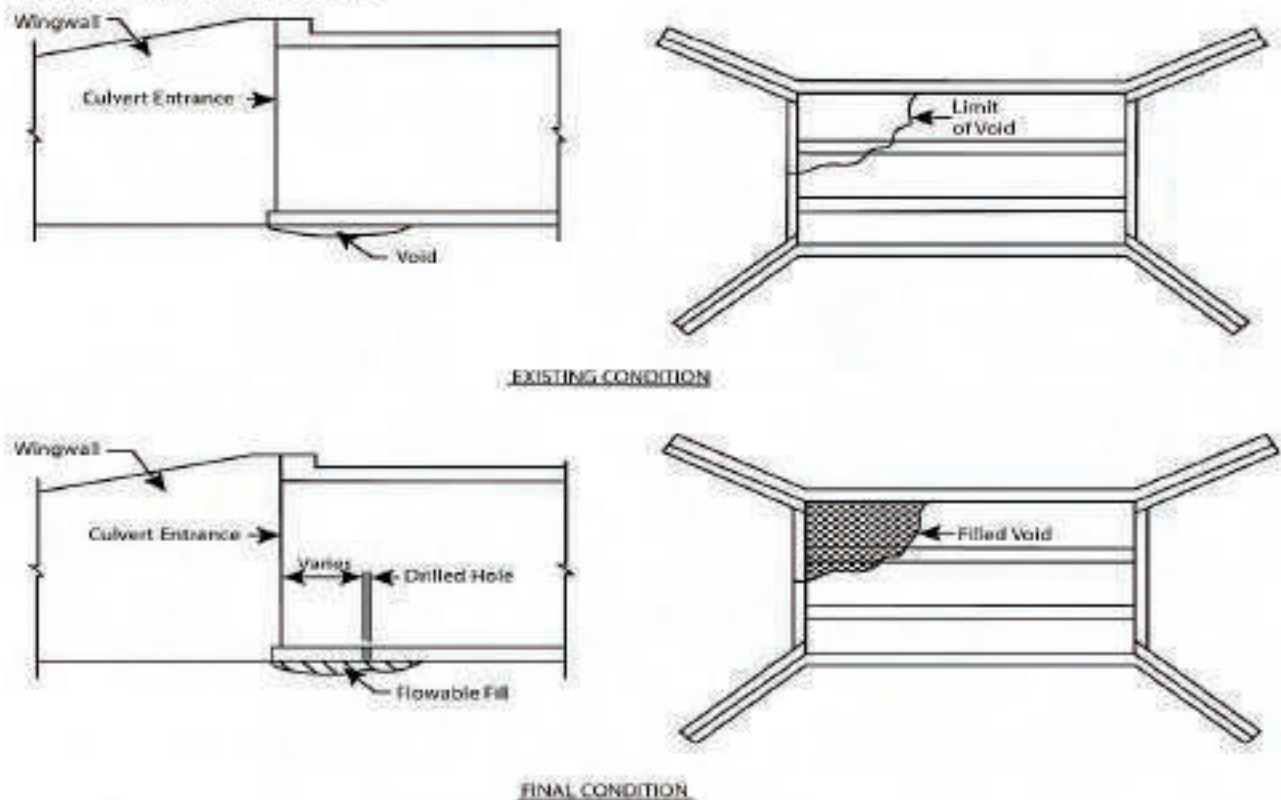


Figure 4.7: Void Repair

It may be necessary to perform this repair technique in conjunction with either the construction of a cutoff wall, the placement of riprap, or both.

Grout bags can also be used for foundation undermining repair.

4.6 Repair and Rehabilitation of Culverts

A bridge maintenance crew will often be called upon to repair and rehabilitate culverts. This will go beyond simply cleaning debris and patching damaged areas.

4.6.1 Repair of Damaged Invert

4.6.1.1 Concrete Paved Invert

When the invert of a culvert has deteriorated, and the culvert is large enough for workers to enter the culvert, a new concrete invert may be cast in the culvert. Prior to relining, or as part of the new invert installation, any voids beneath the invert should be filled with concrete. An example of a repair of a damaged invert by using a concrete paved invert is shown in Figure 4.8.



Figure 4.8: Concrete Paved Invert

4.6.1.2 Shotcrete/Spin cast Concrete Liner

A deteriorated culvert may be relined with pneumatically projected concrete (Shotcrete) or spincast concrete. Both methods provide a new concrete liner to the culvert which can be structural if designed correctly.

Pneumatically projected concrete is most often used in culverts that are large enough for workers to enter. It consists of a dry concrete mix that is pneumatically applied, typically 2 inches to 4 inches thick on the interior surfaces of the culvert. If necessary, reinforcing steel or shear studs can be placed. Alternatively, steel or polypropylene fibers can be included in the concrete for added strength.

Spin cast concrete is typically used for smaller diameter culverts that are not feasible for workers to enter. The spin casting process uses a centrifugal device to cast a cement/mortarmix against the walls of the culvert to form a new concrete liner inside the existing culvert.

4.6.2 Rehabilitating/Installing Toe Walls

In some cases, installation of toe walls can serve to create a permanent repair in areas where there is a tendency for scour holes to develop at either the inlet or outlet of a culvert. If sized correctly, toe walls prevent scour from undermining the culvert, but toe walls will not prevent scour holes from forming. The toe wall should extend to at least 6 inches below the depth of expected scour to protect the culvert from undermining. After the stream flow is diverted around the work area, the toe wall is formed and poured. The toe wall should be adequately anchored to the culvert with rebar, bolts, shear studs or other appropriate means. On completion of the wall, the scour hole is backfilled with either riprap, or native stream bed material to conform to the natural stream bed. An example of a toe wall and its associated detail are presented Figure 14.9 and Figure 14.10, respectively.

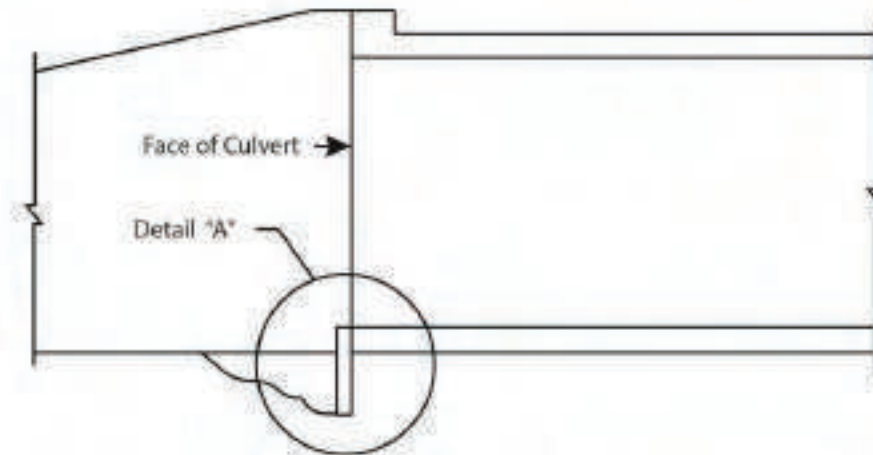


Figure 4.9: Toe Wall

Detail "A" Shown in Figure 4.10

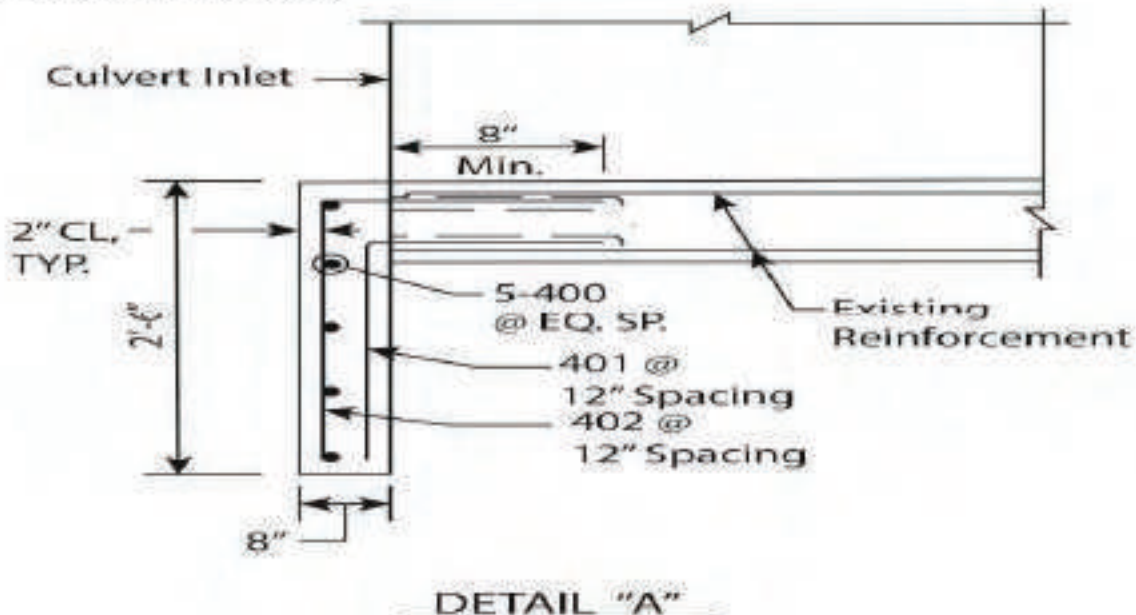


Figure 4.10: Toe Wall Detail at Concrete Box Culvert (Courtesy of GA DOT)

4.6.3 Repointing Masonry

Repointing of masonry is necessary when the mortar has deteriorated to the point that it is either missing or has loosened to the point where bonding and support are no longer provided.

Hand pointing is appropriate if the loss of mortar is less than the full depth of the masonry units, and the new mortar is primarily serving to protect and retain the original mortar. In the cases where the joint mortar is completely missing or deteriorated to the point where it is providing no support, pressure mortaring is a more appropriate technique. Pressure mortaring will provide full penetration of the joints to restore structural strength of the mortar.

Prior to repointing existing masonry, the existing mortar should be carefully evaluated, and a new mortar selected that will provide a similar strength to the existing mortar. A mixture of mortar

strengths in the culvert could cause cracking of existing mortar joints, or cracks to propagate through masonry units where they had previously been in mortar joints.

Because of the age of many masonry structures, they may be eligible for the National Register of Historic Places, either alone or as part of a grouping, district, or larger feature. If state or federal funds are being used for the work, the State Historic Preservation office should be consulted before initiating any work.

Repointing Masonry

1. The masonry should be carefully evaluated to determine the extent and types of repairs.
2. A grout placement plan that also identifies any reinforcement is developed.
3. Necessary water diversions/cofferdams are deployed to permit grouting and repointing to occur in a dry environment.
4. Joints to be repaired are cleaned with low pressure water.
5. Cleaned joints are blown clean with compressed air.
6. Any reinforcement required is installed (See Figure 4.11).
7. Masonry is pre-wet to Saturated Surface Dry (SSD) prior to grouting or hand pointing.
8. Pointing and grouting is performed maintaining the moisture and temperature necessary to provide adequate curing of mortar or grout.
9. Face of masonry units is cleaned prior to set up mortar or grout.



Figure 4.11: Reinforcing Steel Installed Prior to Grouting Stone Masonry

CHAPTER V: PREVENTATIVE AND BASIC MAINTENANCE OF MASONRY BRIDGES

5.1 Signs of Potential Distress

Efflorescence

Efflorescence, when present, is indicative of soluble salt migration into the masonry structure. The salt laden water typically seeps into the mortar joints first; therefore, water stain and efflorescence are most likely to be first visible in the mortar joints. However, if efflorescence is visibly emerging from the masonry units, the area should be monitored for future deterioration.

Split and spall

Splitting and spalling of the mortar joints is anticipated and can be repaired by repointing if required. If the porosity of the masonry unit exceeds that of the mortar, or if the mortar is harder than the adjacent masonry units, spalling of the masonry units can occur. Both conditions should be monitored and appropriate corrective action should be undertaken when required by the field condition.

Mortar Deterioration

Mortar deterioration is expected to occur over the service life of the bridge. The deterioration itself is not necessarily an issue. However, if the contact between the stones is significantly reduced, if water infiltration is occurring, or if other signs of distress are present, repair of the joint may be required. An example of mortar deterioration is shown in Figure 5.1.



Figure 5.1: Mortar Deterioration (indicated by arrow)

Cracking

Cracking in the mortar is common and is not necessarily a sign of distress for the masonry structure. The mortar is designed to fail in order to alleviate the pressure on the masonry itself. However, if there is cracking in the masonry, there is a condition that needs to be monitored. An example of a

crack in masonry is shown in Figure 5.2. Many cracks in the mortar and masonry occur either when the forms are removed or while the arch is being backfilled. These cracks should be noted, but if the condition of the crack (width or length) is not changing, there is most likely no cause for concern. However, if the crack monitoring is noting differences, the crack should be closely monitored and changes in condition should be assessed. Having an engineer review the crack and its effect on the load carrying capacity of the bridge is highly recommended. Masonry arches are typically load rated based on an assumed one-foot unit width. Any cracks that create segment of masonry smaller than one foot should be noted.

Any cracks in the masonry of a slab bridge should be monitored. Transverse cracks indicate that failure has already occurred. Transverse cracks require immediate attention and shoring or road closure over the cracked slab should be considered, at least until an engineer can make an assessment. Longitudinal cracking in the masonry of slab bridges is not as critical. Each individual slab acts as a unit and the load are distributed over the width of the slab unit. A longitudinal crack reduces the distribution width available and is therefore important to note, but not necessarily critical to the load carrying capacity of the bridge.



Figure 5.2: Transverse Crack in Stone Arch

Slipped Masonry Units

A slipped masonry unit is a stone or brick that is lower than the adjacent stones and one that has displaced below the interior of the arch barrel (or the intrados). Examples of a slipped masonry units are presented in Figure 5.3 and Figure 5.4. Many times, the stone slips during construction. This typically only occurs in un-mortared masonry arches and as brick arches are almost always mortared, it is prevalent mainly in stone arches. During a flood event, the fines and chinking stones (small stones used to fill voids between adjacent masonry units) can be washed away causing the stones to loosen in the arch ring. Therefore, monitoring of slipped stones is important after flooding. The slipped stone should be noted and monitored. An engineer should be notified if any change in the condition of the slipped stone is noted. However, the presence of a slipped stone is not typically critical to the load carrying capacity of the structure. Figure 5.3 shows multiple slipped stones, which resulted in arch deformation, this condition is more critical than a single slipped stone.



Figure 5.3: Multiple Slipped Stones



Figure 5.4: Single Slipped Stone

Missing Masonry Units

One modification historically made to masonry bridges is the removal of masonry units for the installation of utilities or drainage. Typically, the arch (or the soil in a slab bridge) is able to create an “arching effect” over the missing masonry units. The condition should be noted, but is not typically

critical to the load carrying capacity of the bridge. An example of missing stones at a drainage pipe is shown in Figure 5.5.



Figure 5.5: Missing Stones in a Masonry Arch

Masonry Displacement

Masonry displacement, or arch deformation, is an indicator that failure of the masonry arch has occurred. Typically, the deformation is localized, but the arch requires immediate attention. Barriers to keep traffic away from the deformed section or temporary shoring may allow sections of the arch to remain in service, but repair should be given priority. An example of masonry displacement is presented in Figure 5.6.



Figure 5.6: Arch Deformation caused by Masonry Displacement

Scour

One sign of potential distress, especially in arches, is scour. Scour is the erosion of the channel bed. Scour is critical if it occurs adjacent to the footings. If the soil behind the arch is scoured away, the integrity of the arch is compromised. The soil behind the arch is used to resist the thrust from the arch and prevent the arch footing from kicking out. As many masonry bridges do not have mortared

stones, scour holes can provide a weakness in the foundation that allows the foundation to displace, therefore causing a displacement in the arch or and slab bridge foundation. Undermining can also occur to the foundation elements if scour is allowed to go unchecked. Stream beds are dynamic and it is not uncommon for scour holes to form during high flow condition and then be filled back in by material suspended in the water settling back out during normal flow. If silt deposits are found during wading, probing and/or underwater bridge assessment, further investigation may be required to determine if a scour condition is present.

Scour should be closely monitored or repaired depending on its severity.

5.2 Repair and Rehabilitation of Masonry Bridges

The intent of repair and rehabilitation of masonry structures is to extend the service life of the bridge, correct a deficient condition, or to bring the bridge up to current standards (such as changing the crash barrier). As many of the structures are historic, it is important to match the existing structure or to repair it in a way that is sensitive to the historical significance. However, public safety is still the first priority.

5.2.1 Repointing of Mortar

Mortar is a sacrificial element of a masonry bridge, and it is anticipated that it will need to be repointed over time. However, keeping the mortar in good repair is an essential aspect of extending the life of a bridge.

Depending on the age of the structure and the preferences of the designer or mason, a variety of binders could have been used in the original construction. The type of binder significantly affects both the characteristics and the performance attributes of the mortar. For example, mortars made with lime binders tend to have a lower compressive strength, lower bond strength, and greater permeability than mortars made with Portland cement as the binder. Mortars with a high lime content also have the ability to re-seal small hairline cracks. This allows the mortar to provide greater resistance to rain penetration and allows the moisture to escape after it has penetrated. However, the high lime content also delays curing of the mortar and the time it will take for the mortar to reach full compressive strength.

When designing the mortar mix, it is very important that the new mortar have a lower compressive strength than the units that it will be connecting (as previously mentioned). The mix must have a greater porosity than the adjacent masonry units. As not all of the mortar is removed in the joints, it is also important that the proposed mortar be compatible with the existing mortar. It is recommended that the mortar be replaced only if there is evidence of deterioration. Therefore, it is also important that the proposed mortar have a similar cured appearance to the existing mortar, especially if aesthetics is a concern. Recommended repair method is as follows:

Mortar Repointing

1. It is recommended that 2 to 2-1/2 times the thickness of the mortar or a minimum of 3/4 inch (whichever is greater) be removed for repair. This will ensure the replacement mortar has adequate bond area. The mortar should be removed using hand tools or a small pneumatic chisel. The use of power tools such as grinder is not recommended because it is difficult to

control and can result in damage to the adjacent masonry units. Square cuts at the limits of the removal are recommended.

2. Debris should then be removed from the joint. This can be accomplished using a nylon brush, low pressure compressed air (40 to 60 psi) or low-pressure water (less than 400 psi). Care should be taken during the removal of the debris to ensure that additional mortar depth is not removed by the debris removal process.
3. Any loose stones should be carefully removed, cleaned, and re-set prior to repointing.
4. Mortar used for repointing should be stiff, as stiff mortar will shrink less as it cures. Mortar should not be allowed to dry out during use and should be reworked at intervals prior to being placed in the joints. Retempering (or adding additional water) to the mix is not allowed. Repointing should not be done if the temperature is below freezing. To help control suction and evaporation of the water from the new mortar, the existing stones and mortar should be wetted to Saturated Surface Dry (SSD) prior to the placement on any new mortar.
5. The mortar should be applied by hand and in $\frac{1}{4}$ inch lifts (especially in the last 1-1/2 inch to be repointed) as shown in Figure 5.7. Thicker lifts and use of pressure gun (or grout gun as shown in Figure 5.8) to apply the mortar (or grout) can be used behind the final 1-1/2 inch if required. All lifts should cure to thumbprint hardness prior to the application of the next layer.

Immediately after repointing, the masonry units should be cleaned of all excess mortar. Application of a bond breaking agent, such as paraffin wax, to the face of the masonry units can aid in the clean-up process. It is recommended that use of metal tools or chemical cleaning agents be avoided unless tests to determine their effect on the masonry units have been performed and no adverse effect is noted.

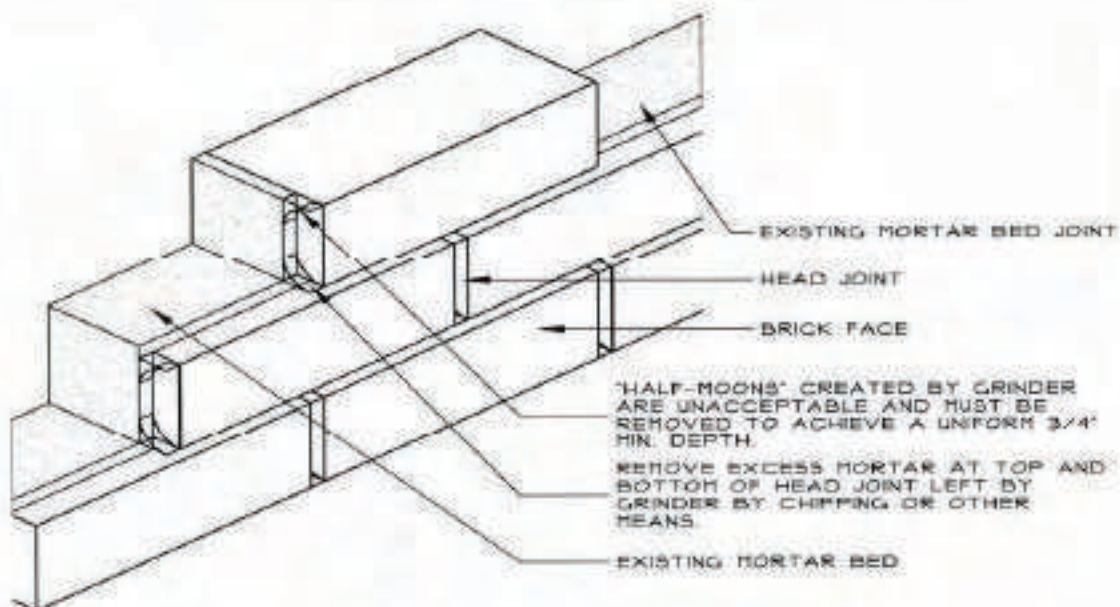


Figure 5.7: Preparation for Mortar Repointing

Cut out mortar in head and bed joints to a minimum uniform depth of 3/4 inch. Remove additional unsound Mortar. Remove dust and debris from the joint by blowing with air. Brick surfaces must be free of mortar or other residue before pointing.

Dampen joints to be tuckpointed. Pack mortar tightly into the joints in three layers (1/4 inch maximum). Each layer should become "thumbprint" hard before applying next layer. Where steel is present, pack mortar around steel.

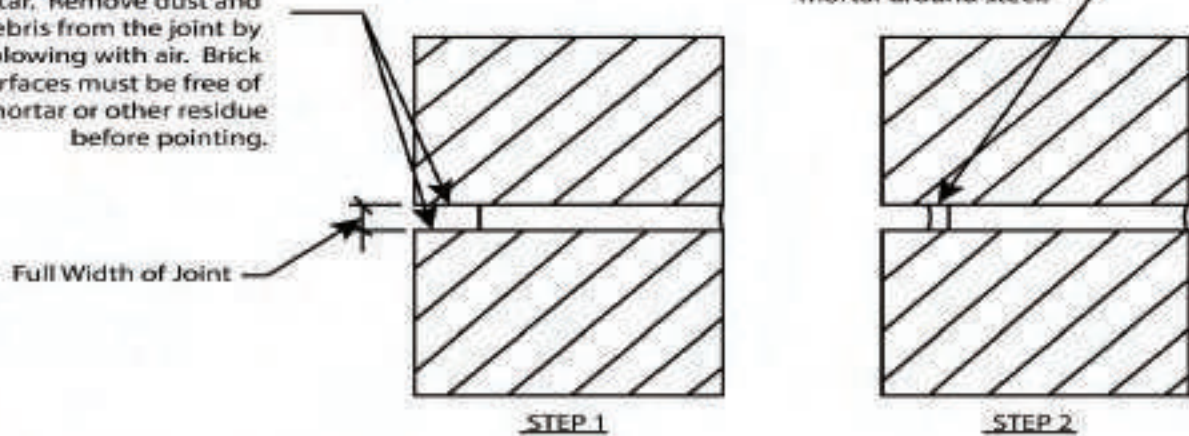


Figure 5.8: Mortar Repointing Schematic

5.2.2 Removal of Shotcrete Coating

The application of exterior coating, such as shotcrete, to masonry structures can trap moisture. The trapped moisture makes the structure more susceptible to frost damage. Frost can cause cracks in the coating (as shown in the example in Figure 5.9) and in the bridge mortar. The cracks then allow more moisture to enter, which then enlarges the crack during the freeze / thaw cycle. The coating can also trap moisture between the masonry units and the coating. If the coating does not allow the masonry units to dry out, spalling and delamination of the masonry units can occur. Additionally, this coating hides potential problems and signs of distress from the inspector, making visual inspection a much less effective tool.

It is recommended that unite or shotcrete coating be removed when signs of distress are found. Wetness along the joint or efflorescence is often the first sign that the coating has water trapped behind it. Removal of the coating should be achieved through the use of hand tools or small pneumatic hammers and care should be taken to remove the coating without damaging the masonry units. Any damage that occurs to the masonry joints should be repaired by repointing.

Care should be taken when the coating is removed. The surrounding surfaces should be continuously monitored to ensure that the coating is not delaminating and potentially detaching, a dangerous situation which could cause a sudden collapse of a large section of the coating and injury to the workers. If any masonry units fall out during the removal of the coating, they should be replaced (see missing stone repair procedure outlined below).

After the coating is removed, it is recommended that the surfaces be cleaned using a light abrasive blast cleaning (below 400 psi) to remove any remaining film and chisel marks. However, if damage to the masonry units is occurring, the process should be immediately stopped and another method of cleaning or a discontinuation of the cleaning should be performed. Low pressure water (preferably demineralized) should then be used to finish cleaning the surfaces.

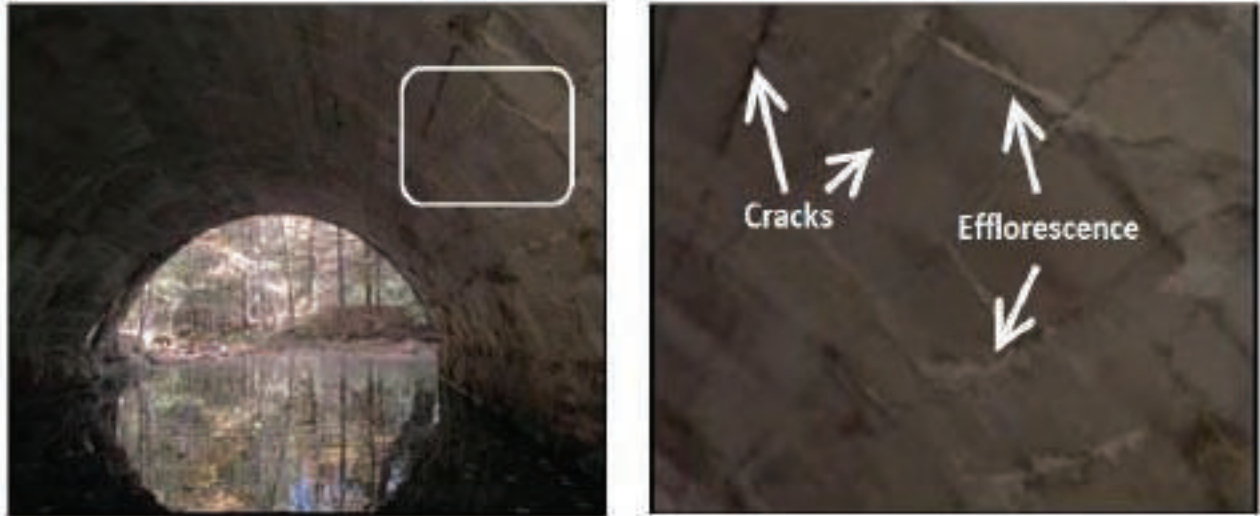


Figure 5.9: Evidence of Degraded Shotcrete Coating. Macro View (left) and Magnified View of Inset on Left Showing Cracks and Efflorescence (right).

5.2.3 Repair of Spalling or Delaminated Masonry Units

Spalling occurs when the exterior face of the masonry unit begins to flake or chip off. Delamination occurs when the masonry unit separates into constituent layers. If the spalling or delamination is extensive, replacement of the whole area may be required. However, if the damage occurs over a small area and there is no evidence of water infiltration, repair can be accomplished.

The recommended repair is to use mortar to patch the face. The repair is typically completed for aesthetic purposes only. If the spalling is widespread enough to be a structural concern, in the opinion of an engineer, the delaminated stones will most likely require replacement. The mortar mix should be designed to match the color of the adjacent stones and finished to match the texture. The face of the masonry unit to be repaired should be cleaned with a wire brush, low pressure compressed air, or low-pressure water prior to the mortar being applied. After the mortar has cured, the adjacent mortar joints should be repointed. At a minimum, mortar lines should be ascribed to represent the joints.

5.2.4 Repair of Missing Masonry Units

This treatment is intended to replace a single stone or small group of stones. If larger areas of stone are missing or loose, a more extensive rehabilitation or restoration of the masonry is required. Missing stones can be indicative of a serious problem with the bridge, such as water infiltration. Therefore, it is recommended that the underlying cause of the missing stones be discovered and addressed prior to replacing the stones.

If the original stones can be found, it is recommended to use the original stones in the replacement. If not, it is recommended to find stones that match the original stones. Clean the area where the stone will be replaced. Apply a bedding mortar (a layer of mortar that is applied below the masonry unit to be set) in the area of the stone to be replaced. Carefully reset the stones ensuring that they are well bedded. Take care to protect the adjacent masonry units. It may be required to shore the masonry unit in place until the bedding material and potentially the mortar used for repointing have

cured. After the bedding mortar has cured, repoint the masonry joints following the repointing procedure.

5.2.5 Repair of Slipped Masonry Units

It is recommended that loose slipped stones be removed and replaced (see missing stone procedure above). If the slipped stone is locked into its location and does not create a structure deficiency, it is recommended that it be left in place. The mortar adjacent to the slipped stone should be repointed (or pointed) to further help secure the slipped stone.

5.2.6 Repair of Cracked Masonry Units

Slab bridges that have transverse cracks cannot be repaired. Temporary shoring can be used until a permanent solution can be found, but the cracked slab requires replacement. Longitudinal cracks in slabs and cracks in arch barrels are not typically structural detriments and can be repaired.

- If there is water leaking through the crack, it must be addressed prior to repair of the crack (see water infiltration above). Only repair the crack after water no longer escapes through it. Mortar injection can be used to repair the crack without having to excavate over the bridge.
- The joint must be cleaned using hand tools, low pressure compressed air, low pressure water or vacuuming. Mortar injection ports should then be affixed into or onto the crack. Make sure that there are ports attached at the ends of the crack to ensure complete filling. For cracks less than 1/4 inch wide, space the injection ports no further than 2 feet apart. For cracks over 1/4 inch wide, the ports can be spaced up to 3 feet apart.
- After the ports have been installed, the crack can be sealed using a stone patching grout. The injection should begin at the wider end of a horizontal crack or at the lowest point of a sloping or vertical crack. Adjacent stones should be monitored during the pressure injection to ensure there is no movement. If any movement is found, immediately stop the pressure injection.
- The grout should be placed in the first port and the filling should continue until it starts leaking out of the next port. The leaking port should then be temporarily capped and the filling should continue until maximum pressure is reached (5 psi, as shown on a pressure gauge). Only then should the pressure injection be moved to the next lowest port and the first port sealed. After the grout has cured, carefully remove the injection ports and fill the holes with grout or mortar.

5.2.7 Arch Deformation Repair

If the deformation of the arch is not detrimental to the structural capacity of the arch, as determined by an engineer, then the stones can be repointed to prevent further displacement of the masonry units. However, if the structural capacity of the arch does not meet the safety requirements for the structure, more extensive rehabilitation is required. (Presented in detail in an ensuing section)

The recommended repair for the displaced stones is the application of mortar or grout with a pressure gun to penetrate as far as possible into the joints. The last 1-1/2 inch of the joints should be repointed by hand.

5.2.7.1 Relieving Slabs

One method of repairing an arch that had insufficient structural capacity is the installation of a reinforced concrete relieving slab, as shown in Figure 5.10. The purpose of the relieving slab is to help distribute the live load to the arch. The relieving slab distributes the live load over a wider area than either a directly applied point load (less than 2 feet of fill) or a pressure based on distribution through over 2 feet of fill.



Figure 5.10: Reinforced Concrete Relieving Slab (Under Construction)

5.2.7.2 Moment Slabs

A common problem with masonry bridges is the lack of a crash tested barrier system and the difficulty inherent in attaching one. One possible solution is the use of a moment slab, as shown by the example in Figure 5.11. The moment slab can be placed over a masonry bridge if there is sufficient fill, as shown by the example in Figure 5.12, or it can be placed around an arch if the geometry allows.



Figure 5.11: Reinforced Concrete Moment Slab (Under Construction)



Figure 5.12: Moment Slab (indicated by arrow)

5.2.7.3 Repairing an Arch if Fill is Removed

Repairing an arch by removing the fill should only be undertaken if all other options have already been exhausted. A Professional Engineer should oversee be involved in the design and construction of this work. Prior to any excavation, the use of formwork should be investigated to determine its necessity. The barrel may appear to be stable when construction is first started; however, as the fill is removed over the arch, it is likely that the stones will shift. It is extremely important that the fill be removed from both sides of the arch in equal lifts as the unbalanced loading is detrimental to masonry arches. Shoring the arch can help to alleviate some of these concerns.

If construction vehicles will be traveling over the arch when fill is removed, or if the depth of fill in the final condition (after the work is complete) is going to vary from the existing condition, it is extremely critical that the arch be analyzed for the revised depths of fill. Masonry arches are very sensitive to variations in the depth of fill and even small changes can be detrimental to the structure.

One possible method of repair is to install a concrete saddle over the extrados of the arch. The concrete saddle is concrete that is poured over the arch to strengthen the arch and lock the masonry units into place. The concrete saddle can either be used over a deteriorated portion of the arch or over the full structure. However, since it is necessary to remove the soil over the arch for the installation of a concrete saddle, the preferred method of strengthening a solid structure that is structurally deficient for current loading is to install a relieving slab to help distribute the load. The purpose of the saddle is to reinforce the area where the saddle is applied, as well as to help distribute the load around the damaged area.

Excavation over the arch can also be required for the installation of a relieving slab or to remedy drainage and water infiltration problems. In any case where fill is removed, it is recommended that well graded fill be used to replace the existing material. Use of a geotextile material for separation is also recommended.

5.2.8 Scour Protection

Substructure failure due to channel scour can cause catastrophic collapse of a masonry bridge. The substructure of a masonry bridge is typically comprised of unconnected masonry units. The small units are much less likely to be able to span over a scour or undermining hole than a reinforced

concrete footing. Masonry bridges are therefore much more susceptible to scour damage. Once the stones begin to settle into the undermined area, continuity is lost and an accelerated loss of stones is likely to occur. If unchecked, collapse of the arch barrel is likely.

Scour holes should be filled with large granular material (6 inch to 8 inch minimum stone size). The actual required stone size should be sized by a hydraulic engineer. Higher velocity streams require larger stone sizes. If undermining of the substructure has occurred, it must be repaired by placement of concrete under the undermined area or another acceptable repair. If undermining of the arch has occurred, there are several methods that can be used to fill the voids.

- Grout bags can be placed in the voids
- Tremie placed concrete can be placed in the voids
- Pressure injected grout can be used to fill the void
- Construct cofferdam, dewater, and pour sub-footing

Typically, a combination of the above methods will be required by the engineer. Dewatering is required.

5.3 Visual Deck Inspection

Visual inspection of reinforced concrete bridge decks provides the inspector with information that can be used to determine if more in-depth investigation or testing is warranted. Visual indicators of bridge deck deterioration or potential for deterioration include: cracking, spalling, abrasion, scaling, efflorescence, rust staining, and freeze thaw damage.

5.3.1 Deck Cracks

The severity of deck cracking is classified based on the frequency, width and length of the cracks. Deck cracks can be classified into several basic types generally based on the orientation of the cracking or the crack patterns produced. Typical crack types and causes are shown in Table 5.1.

Table 5.1: Typical crack types and causes

Crack Type	Cause	Orientation
Shrinkage cracks (Plastic and Drying Shrinkage)	Concrete drying too quickly	Well-defined widely spaced pattern with shallow and wide cracks
Stress Cracks	Stresses in the concrete exceeding tensile capacity	Pronounced on stress lines, typically, transverse or diagonal
Settlement Cracks	Settlement of hardened concrete	They can be of any orientation and width
Map Cracks	Reactions between the cement and aggregate (alkali-silica and alkali-carbonate reactions)	Map cracks are a closely spaced uniform pattern over large area

Corrosion Induced Cracks	Expansion of corroding reinforcement	Oriented along reinforcement lines
Restricted Movement Induced Cracks	Temperature expansion and contraction of concrete that is restricted	At acute corners of skewed bridges

Regardless of cause, reinforced concrete bridge deck cracking creates potential pathways for water and salts to penetrate the concrete matrix and cause deterioration of the concrete or reinforcing steel. Minimizing reinforced concrete deck cracking through proper mix design, placement and curing methods will improve the service life of the deck. When cracks occur, sealing the cracks or applying other preservation treatments that minimize the ingress of water and salts will extend the life of the bridge deck.

5.3.2 Deck Spalls

Decks spalls can be caused by pressure exerted by the corrosion of reinforcement and the resulting expansion that causes cracking beneath the surface of a bridge deck in a plane parallel to the roadway surface. The cracked concrete is pounded by traffic ultimately causing it to break off completely as a spall. The spalled area left behind is characterized by sharp edges on the sides and bottom of the area. Spalls can also be caused by vehicular forces on exposed concrete edges such as deck joints or construction joints. Additionally, spalling may be caused by overloading of the concrete in compression (commonly from restricted thermal expansion). This results in the breaking off of the concrete cover to the depth of the outer layer of reinforcement.

Spalls are visible during a deck inspection and are typically reported using the area spelled out and the depth of the spall. Spalls often occur in areas where delaminations are occurring. The area around a spall should be hammer tapped or chained to determine if there is any delaminated area outside the limits of the spall. Delaminated areas surrounding a spall should be chipped out and patched when the spall is repaired.

5.3.3 Abrasion and Rutting

Abrasion of the top surface of concrete bridge decks can occur from the wearing action of studded tires, snow chains and snow plows. Abrasion is most often found in the wheel lines of the lanes only. The abrasive action slowly chips away the top layer of cement in the concrete exposing the aggregate below the surface. If abrasion is allowed to progress over time, ruts can develop in the wheel lines of the deck. Rutting is characterized by shallow depressions in the wheel lines caused by the concrete being worn away from abrasion. If left untreated, abrasion and rutting tend to attract water and salts into the depression, and expose the concrete matrix which may accelerate the ingress of chlorides.

5.3.4 Scaling

Scaling is the deterioration of the upper quarter to half inch of the concrete deck surface and is characterized by the peeling or flaking away of the surface concrete. Scaling results from repeated freezing and thawing of concrete at high moisture levels. Freezing water held in the concrete expands and causes deterioration of the concrete. The effects of scaling are accentuated by the

presence of deicers and salts. Scaling decks will often have the coarse aggregate exposed and some of the aggregate may have been dislodged by the lack of sound concrete paste holding it in place.

5.3.5 Efflorescence

Efflorescence is a white crystal residue that results when salt laden water reaches the surface of the concrete, evaporates, and salt deposit is left behind. There are different types of salts, and efflorescence on bridges is most often caused by salts that were in the original concrete ingredients. Efflorescence is common on the underside of reinforced concrete bridge decks at cracks or other areas of high permeability. Although efflorescence is harmless, it is an indicator that water is moving through the deck at that location, and the water is likely to contribute to reinforcing steel corrosion and possibly freeze/thaw damage. Advanced efflorescence may result in crystallized deposit building up over time to form stalactites that protrude down from the underside of the bridge deck. A photographic example of efflorescence is shown in Figure 5.13.



Figure 5.13: Typical Soffit Crack with Efflorescence

5.3.6 Rust Staining

Rust staining visible on the top surface, or soffit, of the deck is a clear indication that corrosion is actively occurring within the reinforced concrete deck. Rust staining on the soffit is also an indication of areas where water is passing through the deck from above. The location of all rust staining should be noted during the deck investigation to support other evidence of reinforcing steel corrosion at that location.

5.3.7 Follow the Water

Water is a conveyance method for chlorides and can cause deterioration in the concrete on its own. When conducting a deck investigation, it is important to understand the flow of surface water. By following the water path, the investigator can quickly locate high deterioration risk areas on the deck. When in the field, take note of the cross slope of the deck. Does the water flow to one side or both? Note how the water flows along the length of the bridge. Are there deck drains at low points along the deck or does the water flow off the bridge? If there are construction joints that may be more permeable than the surrounding deck concrete, focus attention there particularly if the construction

joint is at a low point in the deck. The investigator should also look at the profile of the deck and rails to identify any potential low points that resulted from poor grade setting during construction. The low points will tend to have more moisture and chlorides than other areas of the deck, increasing chances of deterioration.

5.3.8 Determining Potential for Corrosion

Corrosion potential can be influenced by cracking, half-cell results, chloride content, pH levels in the concrete, and rebar cover depths. Although not an ideal situation, the potential for corrosion can be determined without knowing all the attribute information. Table 5.2 and Table 5.3 can be used to assess the risk of deterioration of reinforced concrete decks with uncoated and coated reinforcing. For decks with stainless steel or galvanized reinforcing steel, use Table 5.3.

It is possible that deck investigation results will have attributes that fall into multiple corrosion potential classifications. If this is the case, use the most aggressive corrosion classification.

Table 5.2: Uncoated Deck Reinforcing Corrosion Potential Classification

Test Result	Low Potential	Moderate Potential	High Potential
Chloride Levels (Pounds per Cubic Yard)	< 2	2.0 to 2.5	> 2.5
Half-cell Potential (Volts)	More positive than -0.2	-0.2 to -0.35	More negative than -0.35
Cover Concrete Depth (Inches)	> 1.9	1.0 to 1.9	< 1.0
Concrete pH	> 9.0	7.0 to 9.0	< 7.0

Table 5.3: Coated Deck Reinforcing Corrosion Potential Classification

Test Result	Low Potential	Moderate Potential	High Potential
Chloride Levels (Pounds per Cubic Yard)	< 10	10 to 15	> 15
Half-cell Potential (Volts)	More positive than -0.2	-0.2 to -0.35	More negative than -0.35
Cover Concrete Depth (Inches)	> 9.0	7.0 to 9.0	< 7.0

The corrosion potential classification results above can be used to select the appropriate action table (Table 5.4 through Table 5.6). The action tables should be used with the results of the visual deck assessment of cracking and spalls and the delamination survey. The visual assessment information will intersect on a recommended action class.

5.3.9 Reinforced Concrete Deck Action Classes

Having assessed the corrosion potential of the deck being investigated, select the appropriate action class table for low, moderate, or high corrosion potentials. Use investigation results for cracking, spalls and delaminations to identify the appropriate action class provided in Table 5.4 through Table 5.6.

The cracking attribute should reflect the predominate size and spacing over the entire deck or over the localized area of concern if the deck investigation or repair is limited to certain areas of the deck. In general, the potential for water and salts to enter the deck matrix is being evaluated with the crack information. Spalls and delaminations, on the other hand, are damage that has already occurred.

Table 5.4: Low Corrosion Potential Actions

	Percent Spalls and Delaminated Deck Area 0% < Distress < 2%	Percent Spalls and Delaminated Deck Area 2% < Distress < 5%	Percent Spalls and Delaminated Deck Area 5% < Distress < 10%	Percent Spalls and Delaminated Deck Area Distress > 10%
Deck Cracking Width < 0.02 inch, AND Spacing > 3 feet	Do Nothing OR Repair	Repair	Repair	Rehab OR Replace Deck
Deck Cracking Width < 0.02 inch, AND 1 foot < Spacing < 3 feet	Do Nothing OR Repair	Repair	Repair	Rehab OR Replace Deck
Deck Cracking Width ≥ 0.02 inch, AND 1 foot < Spacing < 3 feet	Do Nothing OR Repair	Repair AND Seal Deck	Repair AND Overlay	Rehab OR Replace Deck
Deck Cracking Width ≥ 0.02 inch, AND Spacing < 1 foot	Repair AND Seal Deck	Repair AND Overlay	Repair AND Overlay	Rehab OR Replace Deck

Table 5.5: Moderate Corrosion Potential Actions

	Percent Spalls and Delaminated Deck Area 0% < Distress < 2%	Percent Spalls and Delaminated Deck Area 2% < Distress < 5%	Percent Spalls and Delaminated Deck Area 5% < Distress < 10%	Percent Spalls and Delaminated Deck Area Distress > 10%
Deck Cracking Width < 0.02 inch AND Spacing > 3 feet	Repair	Repair	Repair AND Seal Deck	Rehab OR Replace Deck
Deck Cracking Width < 0.02 inch AND 1 foot < Spacing < 3 feet	Repair AND Seal Deck	Repair AND Seal Deck	Repair AND Seal Deck	Rehab OR Replace Deck
Deck Cracking Width ≥ 0.02	Repair AND	Repair	Repair	Rehab OR

inch AND 1 foot < Spacing < 3 feet	Seal Deck	AND Protect Deck	AND Overlay	Replace Deck
Deck Cracking Width \geq 0.02 inch AND Spacing < 1 foot	Repair AND Seal Deck	Repair AND Overlay	Repair AND Overlay	Rehab OR Replace Deck

Table 5.6: High Corrosion Potential Actions

	Percent Spalls and Delaminated Deck Area 0% < Distress < 2%	Percent Spalls and Delaminated Deck Area 2% < Distress < 5%	Percent Spalls and Delaminated Deck Area 5% < Distress < 10%	Percent Spalls and Delaminated Deck Area Distress > 10%
Deck Cracking Width < 0.02 inch AND Spacing > 3 feet	Repair AND Seal Deck	Repair AND Seal Deck	Repair AND Seal Deck	Rehab OR Replace Deck
Deck Cracking Width < 0.02 inch AND 1 foot < Spacing < 3 feet	Repair AND Seal Deck	Repair AND Seal Deck	Repair AND Overlay	Rehab OR Replace Deck
Deck Cracking Width \geq 0.02 inch AND 1 foot < Spacing < 3 feet	Repair AND Seal Deck	Repair AND Overlay	Repair AND Overlay	Rehab OR Replace Deck
Deck Cracking Width \geq 0.02 inch AND Spacing < 1 foot	Repair AND Overlay	Repair AND Overlay	Repair AND Overlay	Rehab OR Replace Deck

5.3.10 Selecting the Appropriate Deck Actions

Having determined the appropriate action class from Table 5.7 thru Table 5.9, use the following discussion to further refine the action class for determining the specific deck repair or treatment.

Do Nothing

- **Low Corrosion Potential** – The likelihood of reinforcement corrosion is minimal in these decks. For decks with minimal deck spall and delamination damage, it may be advisable to do nothing if the area of repairs is on the low end of the given range or if there is any uncertainty in the delamination determination. The cracking does not need to be treated until it increases in size or spacing is reduced. However, in areas where deicing chemicals are used, sealing the deck with a low permeability overlay is often a cost-effective long-term preventive maintenance strategy.
- **Moderate Corrosion Potential** – Doing nothing is not advised for moderate corrosion potential decks with identified delamination. The delamination may provide a pathway for accelerated deterioration.
- **High Corrosion Potential** – Doing nothing is not advised for high corrosion potential decks with identified delamination. The delamination may provide a pathway for accelerated deterioration.

Repair

This action class includes spall and delamination repair methods for full and partial depth repairs that are described in Chapter 6. Repairs may be done with a vast number of concrete patch products. Usually these are rapid setting concrete products with high bond strengths. For moderate and high corrosion potential decks, sacrificial anodes should be placed within the patch area to increase the life of the repair.

- **Low Corrosion Potential** decks may warrant spall and delamination repair without further action if delamination is less than 10% of the total deck area. The low probability of corrosion in these decks wouldn't typically warrant the expense of sealing or overlays. However, in areas where deicing chemicals are used, sealing the deck with a low permeability overlay is often a cost-effective long-term preventative maintenance strategy.
- **Moderate Corrosion Potential** decks may warrant repair of spalls and delamination without further action if combined area is less than 5% of the total deck area.
- **High Corrosion Potential** decks should always include an additional protection action (sealing overlay, sacrificial anode or cathodic protection) when delamination is repaired.

Repair and Seal

This action class involves repairing all spalls and delamination using techniques described in Chapter 6 and as described under the repair action above. In addition, the combination of cracking severity and spalls/delaminated area warrants sealing the deck from the ingress of water and salts. Sealing the deck is typically accomplished using a penetrating sealer or water proofing treatments, however low permeability overlays will also seal the deck.

Overlay

This action class includes a vast number of overlay products. The recommendation for an overlay is to span over and seal larger or densely spaced deck cracks and to provide a low permeability barrier that will limit the amount of water and salts that penetrate into the concrete matrix. Overlays are also effective treatments for rutting, abrasion, and scaling that may have been observed during the visual inspection. Common overlays that will seal include asphalt with a membrane seal, thin bonded polymer overlays, modified concrete overlays and polyester concretes. The selection between overlays that will provide the corrosion protection is typically governed by a variety of factors that may include load capacity, profile grade, joint type, rail height, agency experience and how quickly the overlay can handle traffic.

Rehabilitate

This action class is recommended when the spalled and delaminated deck area exceeds 10% of the total bridge deck area. The rehabilitate action class includes several methods to remove varying depths of concrete, removing rebar corrosion and patching the areas disturbed. Chapter 6 describes the saw cutting, chipping and sand blasting method commonly used for smaller scattered areas and hydro-demolition methods typically used for larger areas. If a deck has experienced this level of spalls and delamination, a complete deck overlay with a low permeability overlay should be considered to prevent future corrosion damage in the deck. An overlay after rehabilitation will also provide a smooth riding surface on the deck instead of a patchwork of repaired areas (as discussed in Section 20.4.5). Alternatively, an active cathodic protection system could be installed to prevent the future corrosion, although this will not address the ride quality.

Replace

This action class is for complete removal and replacement of the concrete deck. Replacement of the deck may be warranted as the percentage of spalls and delamination increases. Agencies typically use a spalling and delamination percentage of 30 - 50% of the total deck area as the triggering threshold for replacement in lieu of rehabilitation. The presence of exposed and corroded bottom mat reinforcing over significant areas also tips the decision toward deck replacement. Material defects such as ASR or other deterioration that causes a loss of strength in the concrete may also trigger replacement. If the delamination and spalls are extensive and very deep, the concrete removal process for rehabilitation may require shoring of the bridge during the repair. In these cases, replacement may be the preferred alternative because the cost of rehabilitation can approach replacement costs.

5.3.11 Common Deck Seals and Overlay Products

Many products are commercially available for sealing and overlaying bridge decks. The selection among the available deck sealing products may be governed by attributes such as viscosity, modulus of elasticity, gel times, time until traffic can ride on the treated deck, and temperature at the time of application. Low viscosity sealers can penetrate deeply into fine cracks and higher viscosity sealers are better suited for larger crack widths. High modulus sealers are typically very stiff, while low modulus sealers are flexible.

For overlays, the selection may be governed by cost, thickness of overlay, time required before traffic can use the overlay, and the permeability of the overlay. The additional dead load on the bridge, and the required minimum height of bridge rails, are considerations when determining the allowable thickness of overlays. An engineering review should be conducted prior to installing or increasing the thickness of overlays. Table 5.7 and Table 5.8 present the attributes of common deck seals and overlays, respectively.

Table 5.7: Common Deck Sealing Products

Seal Type	Viscosity (Centipoise)	Modulus of Elasticity	Expected Life (Years)	Time-to Traffic-Use (Hours)	Cost Range (Dollars per Square Foot)
Low Modulus Low Viscosity	≤ 50	Low	5 - 10	2 - 6	0.20 - 3
Low Modulus Super Low Viscosity	> 50	Low	5 - 10	2 - 6	0.20 - 3
High Modulus Low Viscosity	≤ 50	High	5 - 10	4 - 8	0.20 - 3
High Modulus Super Low Viscosity	> 50	High	5 - 10	2 - 6	0.20 - 3

Table 5.8: Common Deck Overlay Products

Overlay Type	Thickness (Inches)	Permeability	Expected Life (Years)	Time-to Traffic-	Relative Cost ⁷
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				Use (Hours) ⁶	
Asphalt w/o Membrane	2 – 12 ^{1,2}	Variable, but generally high	10 - 15	2 - 6	\$ - \$\$
Asphalt with Membrane	3 – 12 ^{1,2,3}	Zero ⁴	10 - 20 ⁵	2 - 6	\$\$
Thin Bonded Polymers	0.125 - 0.5	Zero ⁴	5 - 15	2 - 6	\$ - \$\$\$
Modified Concretes	1 - 12	Very Low	20 - 30	4	\$\$\$ - \$\$\$\$
Polyester Concretes	0.75 - 12	Very Low	20 - 30	2	\$\$\$ - \$\$\$\$

¹ Asphalt thicknesses over 4 inches are not recommended on bridges due to the weight of the asphalt. The live load capacity of a bridge is decreased by the weight of an overlay.

² Asphalt overlays provide a protective wearing surface, but tend to trap moisture and chlorides.

³ Asphalt overlays without waterproofing membranes are not recommended for bridge decks where deicing chemicals are used.

⁴ Three inch minimum is recommended to prevent disturbing membrane during asphalt resurfacing that includes roto milling and replacing the asphalt to the original thickness.

⁵ The sealing material is impermeable where there are no imperfections due to installation, aging, or wear.

⁶ Waterproofing membranes are known to last over 30 years when not disturbed. However, the asphalt typically needs resurfacing every 10 to 20 years to maintain the wearing surface and protect the membrane. It is important to not disturb the membrane during resurfacing operations.

⁷ Time-to-Traffic Use is the time after the last operation has been completed before traffic can be restored.

⁸ All costs are for deck preparation, furnishing of material and placement.

5.3.12 Strength Evaluation

Concrete deck strength should be tested during construction to verify that the bridge deck being built meets the design specifications. In rare cases, the deck concrete may lose strength over time due to material deterioration. The most common cause of strength loss of existing bridge decks is ASR. Table 5.9 provides a decision matrix for evaluating strength and ASR results for reinforced concrete bridge decks.

Table 5.9: Concrete Compressive Strength Evaluation

ASR Present?	Concrete Compressive	Concrete Compressive	Concrete Compressive
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	Strength \geq Design Strength	Strength $<$ Design Strength but Acceptable for Applied Loads	Strength $<$ Applied Loads (Unacceptable)
ASR Not Present	Do Nothing	Do Nothing	Replace Deck
ASR Potential in Aggregate	Treat ASR	Treat Deck	Replace Deck
ASR Active	Treat ASR	Treat ASR OR Replace Deck	Replace Deck

Do Nothing

If ASR cannot be confirmed by lab testing and the strength of the concrete is adequate for the loads it carries, it is appropriate to do nothing.

Treat ASR

The appropriate treatment could involve penetrating deck sealers, such as high molecular weight methacrylate or protection methods, that would typically involve the application of a low permeability or impervious overlay to limit the moisture flow into the concrete matrix. If the deck can be kept dry, the ASR will be denied the moisture necessary to further react.

Replace Deck

If the concrete strength is found to be inadequate, there are two possible corrective strategies: deck replacement or the construction of a reinforced deck on top of the existing deck.

6.1 Identification of Deteriorated Concrete

Concrete is a cost-effective building material, i.e., relatively cheap but strong (in compression) and fairly chemically resistant. But eventually concrete will crack and deteriorate. There are many reasons why deterioration occurs, such as:

- Adverse chemical reactions between the cement and the aggregate
- Improper mix design
- Lack of concrete cover
- Problems with delivery to the site
- Temperature extremes on the day of the pour
- Poor curing

While these issues are all important, even a quality mix design placed in perfect conditions can still have problems down the road.

The culprit in concrete degradation typically is not the concrete itself, but the steel reinforcement (rebar) that is embedded in the concrete. When water has access to the rebar, either through a crack in the concrete or through pores in the concrete itself, the rebar is then exposed to water, oxygen, and potentially chlorides. These ingredients, in physical contact with the steel rebar, will begin the corrosion process. Once steel starts to corrode, it expands to many times its original cross section due to the formation of corrosion product at the rebar surface. The corrosion product exerts pressure onto the concrete, a tension force, and since concrete is weak in tension, it will further crack. These new cracks will allow more water, oxygen and salt to further corrode the rebar in a vicious cycle. Soon the concrete will delaminate or de-bond from the steel, and a spall develops. See Figure 6.1 that depicts the typical concrete deterioration process.

There are many factors that can contribute to de-passivation:

- Structural deficiency - the result of inadequate design or poor detailing
- Unsatisfactory construction - poor workmanship (honeycombing, porous or permeable concrete), placement of concrete in high temperatures, inadequate curing (plastic and restrained shrinkage and settlement)
- Chemical attack - due to the effect of substances such as acids, sulphates, chlorides, soft water, grease, and oils, which increase the risk of corrosion
- Mechanical damage - abrasion or spalling of concrete cover, or erosion
- Mechanical damage - abrasion or impact on reinforcing steel during construction
- Insufficient curing - curing is inadequate or not carried out Insufficient early strength – loading applied to the concrete before adequate concrete strength has developed.

- Insufficient cover to the steel reinforcement - the concrete cover provides a physical barrier against the effect of aggressive agents such as chlorides, carbon dioxide, oxygen, and moisture. Under certain conditions, lack of a cover or damage to the cover reduces the effectiveness of the concrete in providing physical and chemical (e.g. highly alkaline environment) protection against corrosion of the steel.

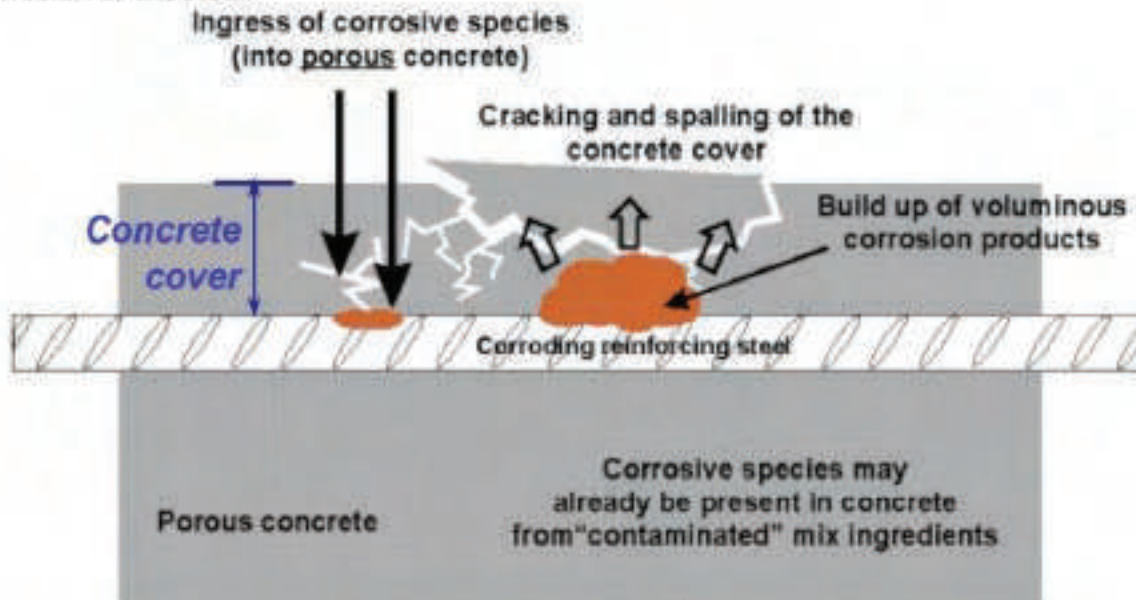


Figure 6.1: Concrete Deterioration Process

Table 6.1: Potential Solutions to Prevent Concrete Deterioration

Potential Solution	How is the Solution Working?
Protect the steel rebar by coating	This has been used with some success, epoxy, galvanized, stainless steel, stainlessclad rebars, etc. Caution needed when rebar coating is damaged, particularly during on site storage and placement during bridge construction (e.g., there are stories of trucks driving over the rebar).
Replace steel rebar altogether with smart materials	Rebar made of fiber reinforced composites (FRP) has been used in many bridges with promising results.
Make concrete denser	Low permeability concrete has helped deter the corrosion process by reducing the water access to the rebar.
Preventive Maintenance, fill cracks as soon as they appear, apply protectivemembranes	

Failure of concrete is a complex phenomenon because more than one mechanism of damage occurs at the same time, and it is difficult to determine which problem caused the initial damage. It is important to have an understanding of the basic underlying causes of damage in concrete and the maintenance problems that develop. The most common cause for failure of reinforced concrete is corrosion of the reinforcing steel, but concrete can deteriorate by other means as well.

Some common construction related defects in concrete include voids and inclusions (also known as honeycombing, due to incomplete vibration), poor surface finish or cracked surface (due to plastic shrinkage), damage and cracking from residual stresses (due to thermal effects), surface weakness, weak bonds between steel and concrete due to bleeding and cracking at cold joints.

In-place concrete can deteriorate from load-related causes, such as fatigue, impact, residual stresses due to overloading, and creep. Or, deterioration can be environment-related, such as corrosion, chemical attack (alkali-silica reaction), creep and shrinkage, carbonation, freezing and thawing, de-icing salts, and scaling. Improper maintenance or repair could also be classified as a service-induced damage.

6.2 Condition Assessment

Assessing the condition of concrete members is categorized into Non-Destructive Testing (NDT), semi-destructive testing, and destructive testing. NDT (also called Non-Destructive Evaluation or NDE) is a class of evaluation techniques that do not damage the existing concrete. Semi-destructive tests require removal of a piece of the material for evaluation. Destructive tests are performed to failure. Destructive tests, such as those done on concrete cores, result in some minor damage to the concrete, but the damage can be repaired. A brief summary of NDE is provided in this section, with a more comprehensive summary provided in Appendix 2 References.

The first step in performing a condition assessment is to get details of the structure with respect to its design, features, and past performance. An initial visual inspection of the structure can reveal useful information about areas that need a closer look. There are many causes for the deterioration of structures, so it is difficult to exactly pinpoint the type of damage that has led to the deterioration. However, all types of damage, whether they be load related, environment related, or earthquake related, lead to similar signs of concrete deterioration, cracking, spalls, delamination and corrosion.

Traditional methods for locating unsound or deteriorated concrete include visual assessments (including cameras), sounding (chain drag and hammer sounding), and half-cell potential measurements (copper-copper-sulfate). Specialized equipment can also be brought in to help determine the specific mode of deterioration.

6.3 Deterioration of Internal Origin

Shrinkage or swelling that is caused by the internal chemical reaction can initiate cracks on the surface of the concrete or within the concrete. The effect on deterioration of the constituents of concrete are summarised as follows:

- a) Cement and other cementitious materials; cracking may result from higher percentage of expansive cement, heat generation from hydration of concrete, or excessive drying of fresh concrete.
- b) Aggregates: certain aggregates react with the alkali present in cement to form a gel that swells by absorbing moisture, resulting in pop outs or cracks leading to efflorescence that may destroy the structure.

6.4 Deterioration of External Origin

The physical properties of concrete are a major influence in the rate of deterioration of concrete from external origins. Factors that affect deterioration of external origin include permeability,

compressive strength, density, aggregate size and distribution, cement content, water-to-cement ratio and mix design generally. Permeability is controlled by the mix constituents (i.e. cement content, water-to-cementitious material (W/C) ratio, concrete cover, and the standard of compaction and curing. High strength concrete is more resistant to penetration of corrosive agents. High-density concrete is more impermeable. Better compaction contributes to higher density and strength. A well graded and uniformly distributed aggregate within the concrete assists with providing a well compacted, dense and impermeable concrete. Low cement content results in higher permeability, lower alkalinity and lower strength. Higher water-to-cementitious material (W/C) ratio results in higher permeability, higher porosity, and greater drying shrinkage. Use of water reducing admixtures, super plasticizers, fly ash, slag, silica fume, and lower sand content facilitate the reduction of water content.

6.5 Types of Reinforced Concrete Deterioration and Detection

The main categories of reinforced concrete deterioration are shown below:

6.5.1 Cracking

Cracking can be found in almost all concrete bridge elements. The cracks may be vertically oriented such as the column cracks shown in Figure 5.2, horizontally oriented following the rebar spacing, and they can be full depth or partial depth such as deck cracks due to concrete plastic shrinkage or restraint. Cracking can also be random based on the material deterioration as described below.

6.5.2 Delamination

Detection of delaminations (or the debonding of concrete from the reinforcing steel) includes detection of predominantly horizontally-oriented cracks, although substructure members may exhibit delamination on vertical surfaces. Delamination occurs as a result of rebar corrosion, overloading, or other type of deterioration. Delaminated concrete needs to be detected and restored to original conditions. Delaminations are detected by sounding.

6.5.3 Spalls

Spalls (concrete pieces that are flaking or breaking away from the surface) are detected during a visual review.

6.5.4 Corrosion

Detection of rebar corrosion is directed towards two main objectives: detection and evaluation of intensity of active corrosion, and detection and evaluation of the severity of existing corrosion.



Figure 6.2: Vertical Crack on a Pier Column

6.5.5 Concrete Material Deterioration

Detection of concrete deterioration involves measurement of a change or variability of material properties regardless of what is the cause of concrete deterioration: corrosion, alkali-silica reaction (ASR), carbonation, and mechanical or thermal stresses.

6.5.6 Alkali Silica Reactivity

ASR is a widespread problem, affecting concrete structures in almost every area of the U.S. It occurs when silica in aggregate and alkali in cement react in the presence of water. This reaction creates a gel in the hardened concrete. Over time this gel expands causing cracks and disintegration of the bond between the concrete ingredients.



Figure 6.3: ASR Cracks in a Concrete Column

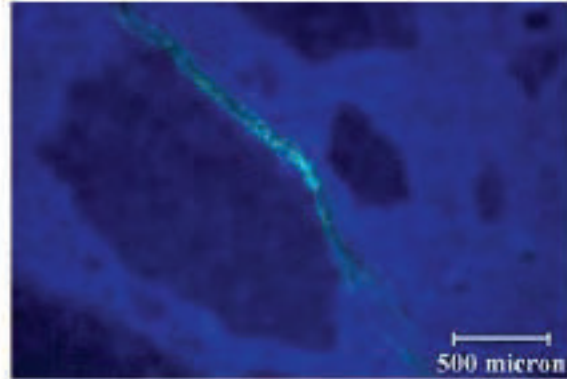


Figure 6.4: Microscopic Image of ASR Crack

6.6 Non-Destructive Evaluation (NDE) Techniques

Non-destructive evaluation techniques range in sophistication from simple methods where the quality of sound obtained by striking the surface of the material with a hammer to complicated techniques where the ultrasonic signals traveling through the material are analyzed mathematically.

Good NDE methods possess the following qualities:

- Sensitivity to small flaws
- Reliability
- Simplicity
- Cheap
- Portable
- Easy to use

NDE methods provide information but are not 100 percent accurate. Some factors that affect the reliability of NDE techniques are:

- The location and the orientation of subsurface cracks
- Difficult access for the equipment
- Selection of the proper technique
- Correct application – proper training of technician and proper calibration of equipment
- Environmental factors – weather, live traffic, and material property
- Human factors – the location of the test, operator fatigue, time constraints, bias, and experience

6.7 Visual Assessment

Visual assessment is the most commonly used non-destructive evaluation method for determining concrete condition. A detailed visual inspection makes it possible to focus on the critical areas in a structure that may benefit from further investigation using more sophisticated techniques. Areas that show cracking, discoloration, rust stains, etc. should be investigated closely with the help of visual aids such as magnifying lens and crack comparators. Visual inspections can identify areas of deterioration that could be investigated further using more sophisticated techniques. Concrete

scaling, such as that shown in Figure 7.6, is one of the forms of deterioration readily identifiable during a visual inspection.

Visual inspections can be aided by a number of tools such as:

- Telescopes / Binoculars
- Camera
- Magnifying lenses
- Real-time video
- Ruler, measuring tape
- Crack width gauge comparator
- Light hammer, chipping / scraping tools



Figure 6.5: Scaling on a Bridge Deck

Table 6.2 provides a visual observation of concrete deterioration and corresponding probable cause.

Table 6.2: Likely Causes of Various Types of Deterioration

Visual Observation	Deterioration Type and Probable Cause
Rust staining, cracks run in straight parallel lines at uniform intervals as per the reinforcement position. Spalling of concrete cover	Reinforcement corrosion: Exposure to normal atmospheric conditions, Cyclic wetting and drying
Cracks mostly on horizontal surfaces, parallel to each other, 3 to 6 feet apart, relatively shallow 1/2 to 1 inch in depth and vary in length from 1 inch to 10 feet (see Figure 7.6)	Plastic shrinkage: Caused by surface tension forces, environmental effects of temperature (concrete and ambient), wind velocity in excess of 5 mph and low relative humidity while curing
Cracks characterized by their fineness and absence of any indication of movement, shallow (a few inches) in depth, typically orthogonal or blocky	Drying shrinkage and creep: Placement of a footing on a rough foundation, or chemical bonding of new concrete to earlier placements; the combination of shrinkage and restraints cause tensile stresses that can ultimately lead to cracking

Visual Observation	Deterioration Type and Probable Cause
Cracks are regularly spaced (restrained contraction) and perpendicular to larger dimensions of concrete, spalling (restrained expansion), shallow and isolated (internal restraint), extend to full depth (external restraint), surface discoloration (fire damage) (see Figure 7.7)	Thermal effects: Induced by exothermal chemical reaction in mass concretes. If volume change is restrained during cooling of the mass, by the foundation, the previously placed concrete, or exterior surfaces, sufficient tensile strain can develop to cause cracking
Spalling and scaling of the surface, exposing of aggregate which is un-cracked, surface parallel cracking and gaps around aggregate (see Figure 7.5)	Freeze-thaw deterioration: Alternate cycles of freezing and thawing, use of deicing chemicals
Absence of calcium hydroxide in cement paste and surface dissolution of cement paste exposing aggregates	Acid attack: Acid smoke, rain, exhaust gases
Rough surface, presence of sand grains (resembles a coarse sand paper)	Aggressive water attack: Causes serious effects in hydraulic structures due to a continuous water supply resulting in the washing away of aggregate particles due to leaching of cement paste
Map or pattern cracking, general appearance of swelling of concrete (see Figure 6)	Alkali-carbonate reaction: Chemical reactions between alkali in cement with certain dolomitic aggregates, Expansion due to dedolomitisation and subsequent crystallization of brucite
Map or pattern cracking, expands freely, silica gel leaches from cracks, calcium hydroxide depleted paste.	Alkali-silica reaction: Chemical reactions between alkali ions (Na ⁺ and K ⁺) in cement with silica in aggregates
Map and pattern cracking, general disintegration of concrete	Sulphate attack: Formation of gypsum, thaumasite and ettringite which have higher volumes than the reactants
Single or multiple long diagonal cracks (usually larger than 0.25 inch in width) accompanying misalignment and displacements	Structural damage: Induced by improper construction and maintenance throughout the lifetime of a structure.
Spalling or cracking of concrete, complete collapse of structure	Accidental loadings: Generates stresses higher than strength of concrete resulting in localized or complete failure of the structure
Honey combing, bug holes (Small holes less than about 0.25 inch in diameter), cold joints, cracking in concrete	Construction errors: Improper mix design, consolidation, curing etc., inexperienced labor work, incorrect position of reinforcement.
Surface is generally smooth with localized depressions, long shallow grooves, spalling along monolith joints (abrasion). Severely pitted and extremely rough surface.	Erosion: Rolling and grinding of debris (abrasion), sub atmospheric pressure, turbulent flow and impact energy (cavitation)
Cracking or spalling of concrete, complete deterioration of the structure. (see Figure 7.9)	Design errors: Abrupt changes in design, insufficient reinforcement, inadequate provision for deflection and drainage

The following forms of deterioration may be observed by visual inspection and apply to concrete construction:

- Crack type, classified as:
 - Hairline – barely visible
 - Fine – 1/32 inch to 1/16 inch
 - Medium – 1/16 inch to 1/8 inch
 - Wide – Greater than 1/8 inch
- Patterns, location, and orientation of cracks (whether stress related or not)
- Scaling, classified as:
 - Light – does not expose coarse aggregate
 - Medium – involves loss of surface mortar to 1/8 inch to 3/8 inch in depth and exposure of coarse aggregate
 - Severe – involves loss of surface mortar to 1/8 inch to 3/8 inch in depth with some loss of mortar surrounding aggregate particles 3/8 inch to 5/8 inch in depth
 - Very Severe – involves loss of coarse aggregate particles as well as mortar generally to a depth greater than 5/8 inch
- Spalling, classified as:
 - Small – involves a roughly circular depression not greater than 5/8 inch in depth and 6 inch in any dimension
 - Large – may be roughly circular or oval or in some cases elongate; more than 5/8 inch in depth and 6 inches in greatest dimension
- Exposed reinforcement

Other general defects that can be identified by visual observation are:

- Surface distress: Disintegration of the surface, surface honeycombing.
- Water leakage: Surface dampness, efflorescence, seepage or leakage through joints or cracks.
- Movements: Deflection, heaving, settlement.
- Metal corrosion: Rust staining, exposed post-tension cable strands.
- Miscellaneous: Blistering membranes and coatings, ponding of water, discoloration.



Figure 7.6: Plastic Shrinkage



Figure 7.7: Cracking from Thermal Effects



Figure 7.8: Map Cracking



Figure 7.9: Spalls and Cracking

6.7.1 Concrete Cracks

Cracks in reinforced concrete do not necessarily mean the presence of unsound concrete. Regardless of the crack type, water seeps in and causes the reinforcement to corrode. The corroded reinforcement expands and exerts pressure on the concrete. This pressure can cause delaminations and spalls. The presence of rust stains or efflorescence are evidence of possible reinforcement corrosion and section loss.

Exercise care in distinguishing between non-structural cracks and structural cracks. An example of spalled concrete and efflorescence is shown in Figure 6.10.

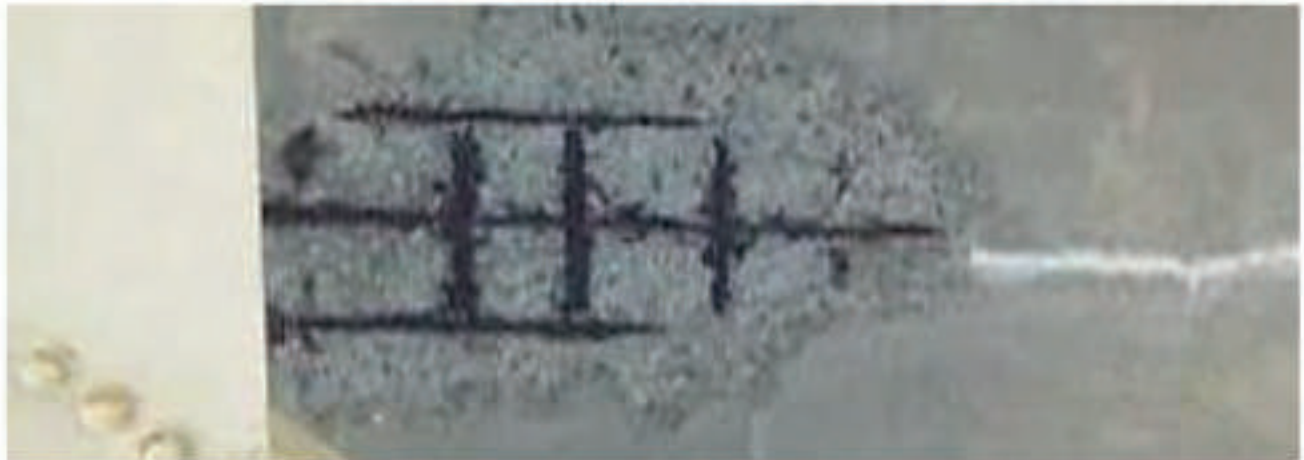


Figure 6.10: Spalled Concrete and Efflorescence on the Underside of a Bridge Deck

Crack widths in reinforced concrete bridges exceeding 0.006 inches to 0.012 inches reflect the lower limit of moderate cracking. The American Concrete Institute Committee Report 224R-01 presents guidance for what could be considered reasonable or tolerable crack widths at the tensile face of reinforced concrete structures for typical conditions. These range from 0.006 inches for marine or seawater spray environments to 0.007 inches for structures exposed to de-icing chemicals, to 0.012 inches for structures in a humid, moist environment. A crack comparator used for determining crack widths is shown in Figure 6.11.

In prestressed concrete bridge structural elements, cracking is generally more problematic than cracking in reinforced concrete elements. This is because prestressing is designed to control or prevent cracking in the member while normal steel reinforcement is intended to structurally bridge cracks. Thus, cracking in prestressed elements may indicate a design or fabrication flaw or that an overload of the structural element has occurred. A determination of the exact cause of the cracking, as well as an assessment of possible structural damage to the element and/or its prestressing steel, is needed before arriving at a repair method.



Figure

6.11: Crack Comparator for Determining Crack Width

6.8 Sounding

6.8.1 Hammer Sounding

A good evaluation of concrete can be easily obtained by just sounding it (i.e., tapping it) with a hammer. When the hammer is struck on good concrete, a ringing sound is created. Where delaminations occur, the striking of the hammer produces a drum-like, or thud. A schematic is presented in the left side of Figure 6.12. Because sounding is a manual method, several factors can affect accuracy of the investigation. Exposure to constant sounding may lead to temporary or permanent operator desensitization to high-pitched tones. It can also be difficult to hear when sounding near live traffic. The feel of the rebound of the hammer will help an experienced technician determine the condition of the concrete. The hammer will seem to bounce off sound concrete but tend to “lay-into” punky concrete. The limitation of sounding is that it cannot detect defects that exist deep in the member. Also, defects lying under overlays are also difficult to find. Operator fatigue can also be a problem.

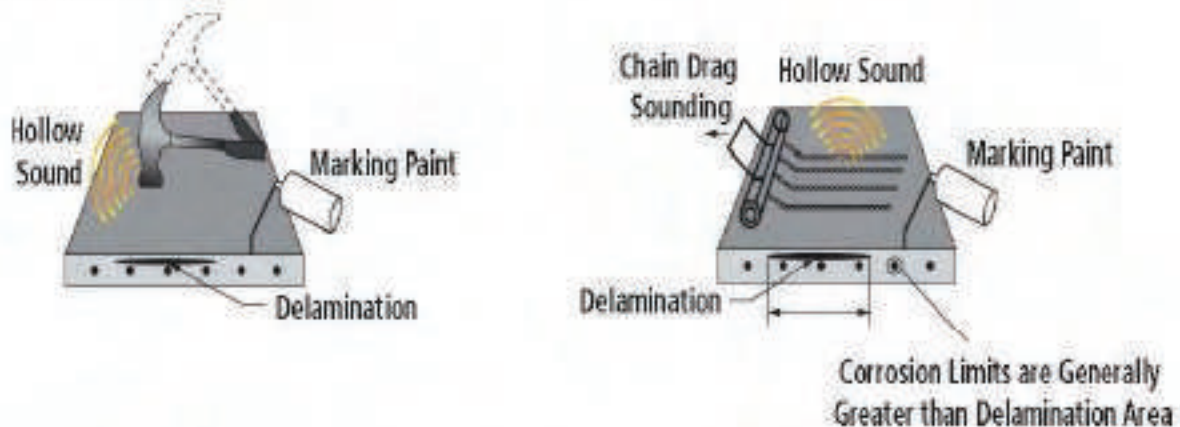


Figure 6.12: Sounding Tests: Hammer (left) and Chain Drag (right)

6.8.2 Chain Drag Sounding

Chain drag is another way of finding delaminations and voids. Compared to sounding with a hammer, chain drag can cover more area in a given time. In this method, the operator passes a heavy chain on the surface of the concrete. The quality of sound generated is picked up using microphones and can be characterized by wavelengths. A schematic of chain drag was shown in the right side of Figure 6.12.

To conduct a sounding test, a grid is first laid on the concrete surface. Then the surface is sounded; the areas where delaminations are heard are recorded. A delamination map is then drawn and the amount of delamination is computed as a percentage of the total surface area. Spalls are not included. For bridge decks, 5-foot grid may be used. For substructure and/or superstructure members, the grid size may be 2.5-foot.

Suggested Procedure for Sound Testing:

1. A grid is first laid on the concrete surface.
2. Then Surface is sounded; the areas delaminations are heard are recorded

3. A delamination map is then drawn and the amount of delamination is compound as a percentage of the total surface area. Spalls are not included. For Bridge decks, 5-foot grid may be used. For substructure and/or superstructure members, the grid size may be 2.5 foot.

When the method is used on decks overlaid with cement concrete mixtures, it will detect both debonding of the overlay and delamination of the underlying concrete; but the method cannot distinguish debonding from delamination. A supplementary corrosion detection test may be used to delineate debonding from delamination. Note that delamination will be associated with bar corrosion, whereas debonding will not. The sounding method is considerably less accurate when used on decks overlaid with asphalt concrete and should not be used on decks overlaid with asphalt concrete. For more information, see *ASTM D4580 Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding*, 2012.

6.9 Corrosion Survey

6.9.1 Corrosion Potential

This procedure determines the potential for the existence of reinforcing steel corrosion through measuring the electrical potential of the reinforcing steel. The electrical measurements are made by placing a half-cell probe on the deck surface at predetermined grid points. The electrode may be attached to a hand-held voltmeter or it may be in the wheel of a rolling device that stores data for transfer to a laptop computer. Either way, the probe is attached to a rebar in the deck, which is in contact with the rebar mat being tested. A schematic of half-cell equipment for measuring corrosion potential is shown in Figure 6.13.

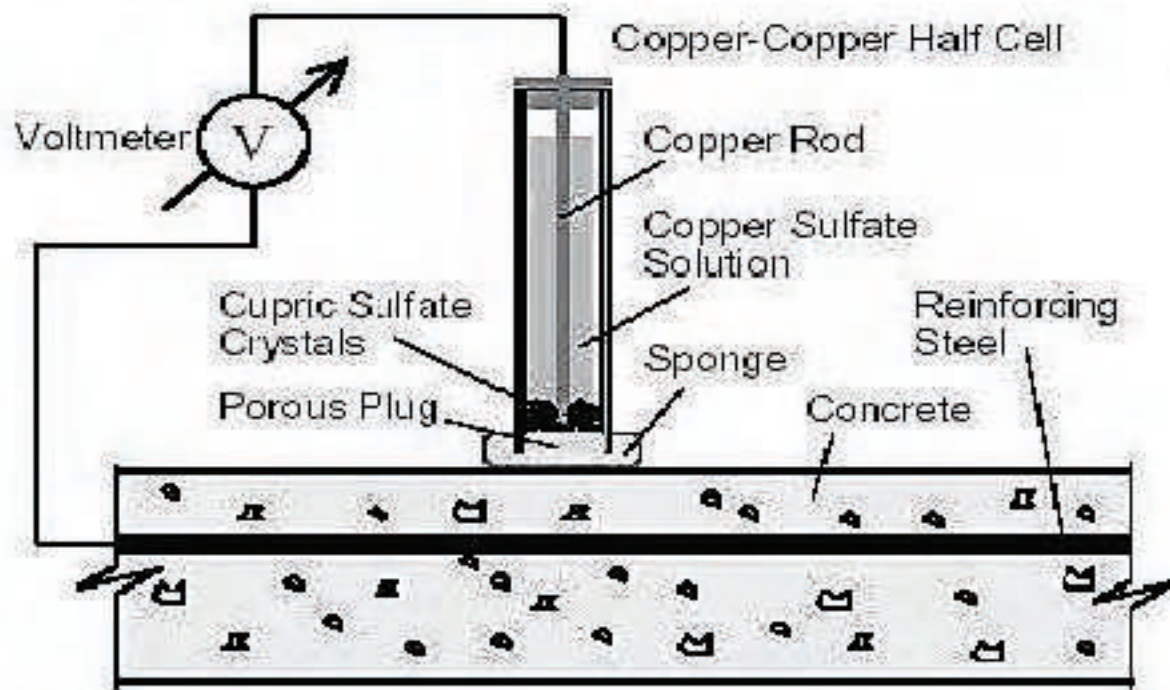


Figure 6.13: Schematic of Half-Cell Potential Test

An example of half-cell test equipment applied on a bridge deck is shown in Figure 6.14.



Figure 6.14: Example of Half-Cell Potential Test

The magnitude of the voltage measured determines the state of the steel with respect to corrosion. Laboratory tests have found that steel is corroding if the electrical potential is numerically greater than 0.35 Volts (relative to copper/copper sulfate reference cell) and steel is not corroding if the potential is numerically less than 0.20 Volts. For potential in the range of 0.20 to 0.35 Volts, the corrosion activity of steel was found to be uncertain.

Electrical potential test results may be used for repair of reinforced concrete bridge elements. To do this, the deteriorated areas as well as all areas with a corrosion potential numerically more than 0.25 Volts (relative to copper/copper sulfate half-cell) are removed and replaced.

The half-cell corrosion detection test cannot be conducted when a dielectric material, such as a waterproofing membrane or polymer, is applied on the concrete surface, unless the material is punctured at the test location. Also, the test is not recommended for epoxy-coated reinforcement. This is because epoxy-coated bars are electrically insulated from each other. Older decks in which the epoxy coating is more deteriorated may be an exception; in that situation, half-cell testing may provide value.

6.9.2 Depth of Cover Meter (Pachometer)

The less concrete over the rebar, the less protection there is against salt penetration, since moisture can access the rebar more quickly. Cover depth may be measured by a cover meter (pachometer), which is a magnetic flux device. This device uses a magnetic field to detect reinforcing steel within concrete. It can determine the location, orientation, size, and depth of the bar. For a meaningful evaluation, a number of locations should be tested. An example of an appropriate number of tests is the greater of (1) 40 locations per member or (2) 40 locations per every 5,000 square feet of deck. Examples of a cover meter (pachometer) are shown in Figure 6.15 and Figure 6.16.



Figure 6.15: Pachometer in Use



Figure 6.16: Examples of Pachometer Equipment

The accuracy of the device in measuring the depth of cover decreases as the depth of cover increases. Magnetic particles in the concrete can influence the measurements. Thus, a correction factor should be obtained by exposing the bar at one location and measuring the actual depth. Usually, the correction factor is obtained at a location which can later serve as the half-cell test bar ground connection.

If the bar size is not known, due to equipment limitations using magnetic field measurements, the depth cannot be read directly from the scale. Therefore, the following procedure is suggested:

Suggested Procedure of using a Pachometer:

1. Locate the bar in the test area.
2. Place a two-by-four, or other non-metallic spacer, between the probe and the concrete surface
3. Record possible bar size and depth combinations
4. Correct readings for the thickness of the spacer by subtracting its thickness.
5. Place the probe directly on the concrete surface and record possible reading

The bar size for which the same cover depth is obtained in steps 4 and 5 is correct result.

Cover depth may also be determined using Ground Penetrating Radar (GPR).

6.9.3 Corrosion Rate Measurement

Since concrete deck deterioration from salt contamination is one of the most expensive bridge maintenance problems, there are ongoing advancements in detecting and predicting the problem. Examples of newer, and perhaps more precise, methods of predicting corrosion damage are devices that measure the rate of corrosion and the permeability of concrete.

The corrosion rate device determines the rate of corrosion of reinforcing steel, or stated differently, the thickness of the rebar section loss every year. Corrosion current is expressed in terms of milliamperes per square foot of reinforcing steel area, which can be converted to section loss per year. These types of measurements may be performed using a field machine, together with

appropriate probes and electrodes. An example field machine is shown in Figure 6.17. Also shown is a schematic of a typical probe and electrode set-up. Wire cable connections between field corrosion rate device and electrodes are not shown. Similar to a half-cell potential test, corrosion rate tests cannot be carried out where epoxy-coated or galvanized reinforcement is used or where there is a dielectric overlay or coating on the concrete.

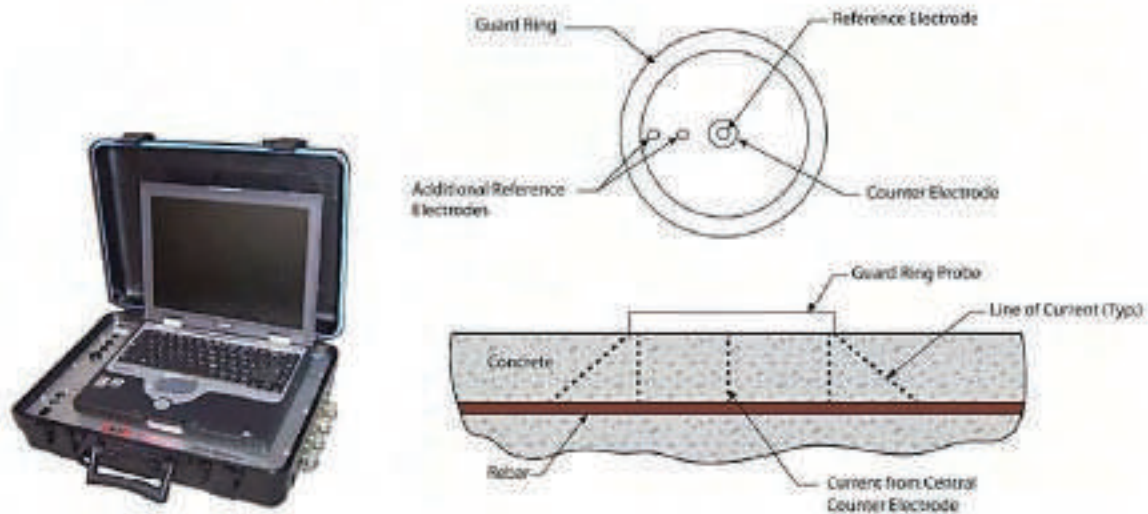


Figure 6.17: Example of a Corrosion Rate Device and Associated Electrode Schematic

6.10 Swiss Hammer for Approximate Compressive Strength

A hand held, spring loaded, device known as a Swiss Hammer (also called a Schmidt Hammer) can be used to gauge concrete compressive strength. Striking the spring tip against a concrete surface produces a gauge reading on the hammer that indicates the compressive strength. It is best to try this several times and take an average reading. The hammer is less accurate than laboratory compressive testing, but may be useful as a triage device in deciding whether to take cores and send them to a lab. A photo of a Swiss Hammer, or Schmidt Hammer device, is presented in Figure 6.18.



Figure 6.18: Swiss (Schmidt) Hammer

6.11 Portable Surface-Air Flow Device for Relative Permeability

Strategic Highway Research Program (SHRP) has developed a test procedure to measure relative concrete permeability non-destructively in the field. Using a portable surface-air flow device, the permeability measuring method creates a vacuum and measures the rate of air flow through the top half an inch of concrete. The higher the air flow rate, the higher will be the permeability. The SHRP method saves time by making on-site, non-destructive testing possible. The SHRP test method, *Method for Field Measurement of Concrete Permeability*, is documented in SHRP Report No. SHRP-S/FR-92-109, Strategic Highway Research Program, National Research Council, 1992.

6.12 Advanced NDT Techniques

Unlike a visual or hammer sounding where the assessment is a subjective determination, these techniques use data recorders and sensors to generate numbers.

Some of the more specialized methods appropriate for the evaluation of highway structures include: Impact echo (IE), Ultrasonic pulse echo (UPE), Ultrasonic surface waves (USW), Ground penetrating radar (GPR) and Infrared thermography (IT).

Non-destructive testing (NDT) of concrete is complicated because concrete is composed of aggregates, cement, rebar, and small air voids. It becomes difficult to distinguish between defects and an indication (a defect within tolerance is called an indication) in normally occurring situations within the concrete. In addition, a uniform pass/fail defect criteria for good concrete is not available.

6.12.1 Impact Echo (IE)

The IE method is based on the reflection measurement of mechanical stress waves, i.e., the time it takes for the stress wave generated by an impact to be received by the receiver. This method was originally developed to determine the thickness of concrete slabs and to detect defects such as delamination and void within concrete structures and has been standardized under ASTM C1383. The method has also been used for detection of debonding beneath Asphalt Concrete (AC) and Portland Cement Concrete (PCC) overlays. An example of the IE method is shown in Figure 6.19.



Figure 6.19: Impact Echo Testing

Recent advances in IE technology include the development of scanning systems and advanced signal processing approaches to speed up field testing and facilitate the interpretation of IE data. The IE method, with both point-by-point and scanning devices, can be used for characterizing delamination, overlay debonding, and for evaluation of vertical cracks.

6.12.2 Ultrasonic Pulse Echo (UPE)

A UPE test involves measuring the time it takes a high frequency ultrasonic wave to travel through a material between two or more points. Based on the time or velocity, this technique can also be used to detect the presence of internal flaws such as cracks, voids, delamination and other damage. Examples of ultrasonic pulse echo equipment are shown in Figure 7.20 and Figure 7.21.



Figure 6.20: Ultrasonic Echo Control Unit Mounted on Automated Scanning Device



Figure 6.21: Ultrasonic Echo Control Unit and Sensor Array

6.12.3 Ultrasonic Surface Wave (USW)

With suitable equipment and processing software, an USW test can be rapidly performed to assess the properties of concrete including the changes in modulus caused by mix design, compaction, curing, stress-related cracking and concrete deterioration. For concrete deck deteriorations such as debonding and delamination, the USW method can play a supplemental role. Examples of USW equipment are shown in Figure 6.22 and Figure 6.23.



Figure 6.22: USW on Bridge Deck

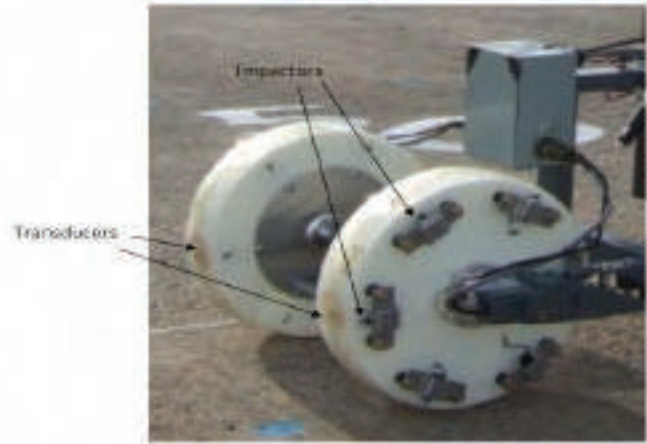


Figure 6.23: Bridge Deck Scanner Device has Two Identical Transducer Wheels

6.13 Destructive Testing

6.13.1 Concrete Cores

Cores are taken to provide a sample of the hardened concrete in place. Cores are used to perform microscopic analysis and other laboratory procedures that identify problems related to aggregate quality, air content, materials incompatibility or chemical contamination. Cores can also be tested to measure the compressive strength of the concrete. However, most concrete problems on existing bridges are related to durability rather than strength, so cores are rarely needed for strength.

The core sample technique requires drill equipment and subsequent laboratory testing. The method is destructive in that it causes micro-cracks and leaves holes in the base material, as shown in Figure 5.24. Since coring is moderately expensive and destructive, cores are typically taken only when other evidence indicates that further investigation is warranted. All core holes should be filled upon completion of coring with a suitable patching material. If the cores are taken on bridge decks, the material should gain at least 2000 psi strength or minimum mandated by the highway agency which owns the bridge before being exposed to traffic. Additional coring photos are included in Figure 6.25 and Figure 6.26.

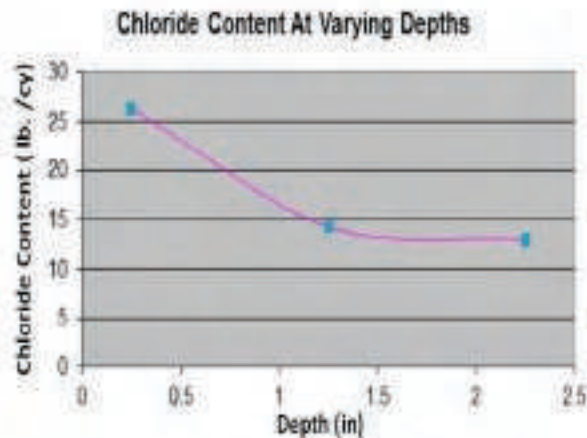


Figure 6.27: Example Graph of Chloride Content in Concrete Core



Figure 6.28: Chloride Profile Sample Collection

6.14 Additional concrete breakout

Where the breakout indicates that the exposed reinforcement is further corroded or the surrounding concrete is not sound, an enlarged area shall be agreed to be broken to the satisfaction of the Engineer-in-Charge.

The Contractor shall test the concrete for extent of carbonation at the reinforcement depth at his own expense. The depth of breakout in clearly defined areas can be increased based on written instructions from the Engineer-in-Charge, in order to remove all carbonated concrete. The additional concrete breakout shall not extend more than 20 mm behind the bottom layer main reinforcement. During breakout, utmost care shall be undertaken to minimize damage to the existing reinforcement.

6.15 Preparation of Deteriorated Concrete for Repair

6.15.1 Removal of Deteriorated Concrete

Concrete can fail for a variety of reasons, such as from exposure conditions, excessive loads, infiltration of deleterious chemicals (de-icing salts, improper placement, and improper mix design). Lack of quality control and quality assurance procedures, inadequate training, limited knowledge of the materials, and poor aggregate are additional factors. An example of a severely deteriorated concrete pier is presented in Figure 6.29.



Figure 6.29: Severely Deteriorated Concrete Pier

The extent of deteriorated concrete removal operations on a given bridge component can vary from large regular areas covering most of the component to small irregular areas in random locations. Bridge decks present large surface areas that are exposed almost uniformly to deicing chemicals and water. This means that deteriorated areas and contaminated areas with potential for deterioration are more likely to occur on decks with almost a systematic pattern. Other bridge elements normally present smaller and irregular deteriorated surface areas. For example, concrete pier caps below leaking joints are also frequently deteriorated by drainage from above. Additionally, deterioration of bridge pier columns located near roadways is typically more prominent on the side subjected to salt laden spray from passing vehicles.

The depth of concrete removed has a profound effect on the method and cost of concrete removal. The necessary depth of concrete removal is tied to the extent of deterioration. Well bonded but stained concrete or concrete preparation to receive a sealer are situations that require only surface removal, such as sand or shot blasting. In contrast, spalled or delaminated concrete needs to be removed to a deeper level. This typically would require concrete removal from around the total perimeter of the rebar to obtain a good bond between the replacement concrete and reinforcement steel. If the rebars are corroded, then the rebars must be sandblasted from behind to arrest the corrosion.

6.15.2 Surface Abrasion and Cleaning

Often it is necessary to remove concrete surface contaminants, such as dirt, oil, and rubber on bridge decks, to provide a durable bond between the concrete surface and certain types of protective layer, such as sealers and coatings, applied on the surface. The objective is to clean and provide a rough texture for bonding rather than to remove material. The two most frequently used methods for concrete surface removal and cleaning are sandblasting (abrasive blast cleaning) and shotblasting. Other options, such as sponge blasting and water blasting (either with abrasives or without abrasives in the water) are also effective in abrading concrete surfaces.

6.15.3 Abrasive Blast Cleaning

Abrasive blast cleaning uses compressed air to propel particles at high velocity onto the surface. The impact of particles produces a very abrasive action which cleans and roughens the exposed concrete. Sand and coal slag (e.g., Black Beauty) are commonly used to abrade the concrete surface. As the blast particles break down contaminants are released into the air and can be inhaled by workers and residents in the area. Recycled glass and very hard aggregates such as garnet do not break down as completely and are safer to use as abrasives, are more environmentally friendly, and perform as well or better than sand or coal slag. Examples of abrasive blast cleaning are shown in Figure 6.30 and Figure 6.31.



Figure 6.30: Abrasive Blast Cleaning Along the Curb Line



Figure 6.31: Abrasive Blast Cleaning a Concrete Barrier

As the concrete surface is abrasive blast cleaned, dust is created. The dust contains silica and very small particles that can cause respiratory and other health related problems. Workers should always wear proper respiratory protection.

6.15.4 Shotblasting

Shotblasters propel steel shot at high velocity against the concrete surface. The impact is capable of removing concrete to depths up to 1/4 inch. The rate at which the machine moves across the surface effects the amount of concrete removed. Machines vary from ride-on types to walk-behind and vary in widths of cut. Shotblasting can also be performed manually. Shotblasting is typically used to prepare the bridge deck surface for a rigid concrete or polymer overlay. An example of shotblasting is shown in Figure 6.32. A collection chamber collects debris and concrete dust and steel shot. The shot is separated from the debris and recycled. The debris should be disposed of as an industrial waste.



Figure 6.32: Shotblasting a Bridge Deck

6.15.5 Cover Removal

Certain types of protective layer, such as bridge deck concrete overlays, require substantial roughening of the surfaces for bond. This and the fact that these systems add to the deadload substantially require partial removal of concrete cover to reduce the dead load and maintain the live load carrying capacity of the structure at the pre-repair level. The work involves removal to a depth less than the cover depth of the steel and thus no work between, around, or under the reinforcing mat is included. Cover removal is normally done by a milling machine, also known as a scarifier, or hydrodemolition, or a combination of both. The machines remove concrete by the impact of numerous teeth mounted on a rotating drum (milling) or pressurized water (hydrodemolition). These techniques are typically used for concrete removal on horizontal surfaces.

6.15.6 Saw Cutting

The recommended procedure to prepare the patch area is to saw cut the perimeter of the patch approximately 1/2 inch to 1 inch deep. Make square cuts. If a concrete overlay is to be applied monolithic with the repair, saw cutting is unnecessary and the edge of the areas of removal should be chipped at about 45 degrees to prevent pockets of entrapped air when placing the overlay. Feather-edging the patching material should always be avoided. Sharp edges, at least 1 inch deep should be formed by jackhammers (not preferred) or, preferably, by saw cutting. When saw cutting, it is advantageous to tilt the saw blade to key in the patch by making it wider at the bottom than at the deck surface. This can be done by running one wheel of the saw on a plank placed on the deck. This angled portion would be made below the 1/2 inch to 1 inch deep vertical surface saw cut. An example of saw cutting and removal of deck cover concrete is shown in Figure 6.33.



Figure 6.33: Saw cut and Removal of Deck Cover Concrete

6.15.7 Removal around Reinforcing Steel

When spalls, delaminations, and contamination have progressed into the concrete that surrounds the reinforcing steel, it is necessary to remove concrete from between and below the reinforcing steel in addition to removing the cover. An example of saw cut removal around reinforcing steel is shown in Figure 6.6. The work must be done such that the steel is not damaged. Also, in the adjacent areas where the concrete is not removed, the bond between steel and concrete must not be destroyed, and the substrate concrete must not be cracked.



Figure 6.34: Saw cut Around Reinforcing Steel with Concrete Removed on an Angle

The two common methods to remove concrete without damaging the rebar are pneumatic breakers (jack-hammers) and Hydrodemolition (a form of water blasting). Both of these systems can be used to remove concrete cover and concrete from between and below reinforcing steel. They could also

be used to remove concrete from between and below reinforcing steel after a higher production method, such as milling, has been used to remove the cover concrete. If concrete has an asphalt overlay, it should be milled prior to Hydrodemolition. Hydrodemolition can be used without milling if delamination or spalls are present.

Jack hammers are suited for small jobs. However, improper handling of the hammer can destroy good concrete. Hydrodemolition is better for large jobs. It also does not cause any of the damage discussed above. Estimating the quantity of concrete to be removed prior to repairs is not an easy task, especially if it is intended that only unsound concrete be removed. Substantial overruns have not been uncommon. Estimating errors can be minimized by a thorough concrete condition survey as close as possible to the time the work is executed.

When the survey is done years prior to the repair, it is common to increase the estimated quantities by an arbitrary amount, usually 25 percent, to account for continued deterioration. Before concrete removal is started on a structural member, the member should be analyzed to determine whether shoring or formwork is required. After the concrete is removed, the remaining section must support its weight, any superimposed dead load, live load if the bridge is to be repaired under traffic, formwork, equipment, and the weight of plastic concrete. On a flexural member the final dead load deflection must be compatible with the other members in the unit.

6.15.8 Pneumatic Breakers (Jack-Hammers)

The pneumatic breaker (commonly known as a jack-hammer) is the most prevalent method for removing concrete from between and below reinforcing steel. An example of a worker jack hammering concrete is shown in Figure 8.7. It is a hand-held tool and powered by compressed air to deliver a series of high frequency blows that fracture concrete in a small, controlled area. The production depends on the size of the hammer and the skill of the operator.

It is common practice to restrict the size of hammers used in concrete removal to prevent damage to otherwise sound concrete. A typical restriction is the use of a 30-pound maximum size jackhammer above the top reinforcing steel and a 15-pound maximum size chipping hammer between and below the reinforcing steel. The existing concrete is removed to sound concrete. Pneumatic hammers should be worked at an angle of 45 to 60 degrees to the plane of the surface being removed.

Some states require removal to below the top mat of reinforcing steel to ensure the new concrete is "locked-in" by the rebar, but practices vary widely. The concrete directly below the top mat (i.e., the "shadow area") may be in good condition and care must be taken not to hit the rebar with the hammer as this will damage the rebar and may cause cracking in the concrete that is to remain. Figure 6.35 shows the top mat of reinforcement exposed and ready to be patched.



Figure 6.35: Hammering Concrete



Figure 6.36: Extensive Patching on a Bridge Deck

Where it is necessary to chip below the bar, a clear space of 1/4 inch plus the maximum size of the aggregate to be used in the repair concrete must be provided. This space is typically 1 inch beyond the reinforcing bar. It is also usual to expose bars that are heavily rusted or where there are heavy rust deposits in the concrete adjacent to the bar. The final step in preparing the existing concrete surface is blast cleaning of the concrete and the exposed steel. If the cause of deterioration is chloride-induced corrosion of the reinforcing steel, it is essential to remove all salt and chlorides from the reinforcement. Abrasive blast cleaning is typically done to remove rust and abrade the surface for the new concrete. After completion of this operation, the concrete should be carefully inspected and aggregate particles that have been cracked or fractured during chipping should be removed to a sound surface.

In areas of very badly deteriorated concrete decks, full-depth removal may be necessary. In such cases, forms must be attached to the soffit of the deck.

6.15.9 Hydrodemolition

Hydrodemolition is a high-production method for concrete removal. An extremely high- pressure water jet is used to destroy the cement matrix and liberate aggregate. The hydrodemolition equipment is expensive and complex. This method of concrete removal needs special skills to produce the desired results. Although hydrodemolition work is mainly used to remove concrete from between and below the reinforcing steel, the equipment can be calibrated to remove concrete to almost any depth. The hydrodemolition process is such that the system self-adjusts the depth of concrete removal depending on the soundness of the material encountered. An example of an area treated by hydrodemolition is shown in Figure 6.37.

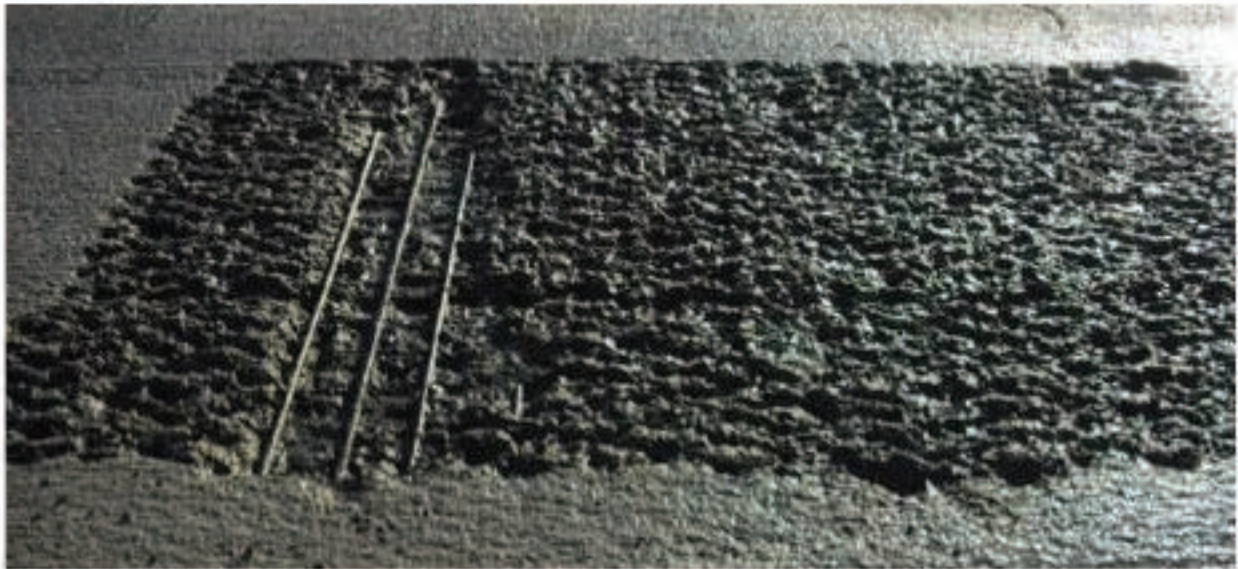


Figure 6.37: Hydrodemolition Test Section

Hydrodemolition is mainly used on horizontal surfaces such as bridge decks and for large-scale concrete removal to minimize cost. However, the method can be used on inclined, vertical, and overhead surfaces but cost effectiveness is reduced by the excessive cost of the specialized equipment needed to safely direct the jet and contain the debris.

Hydrodemolition works well on decks with uniform concrete. Decks that have a variety of patches of differing strength or hardness of concrete may not yield uniform removal depths when hydrodemolition is used.

6.15.10 Concrete Removal on Pier Caps and Columns

Because of leaky joints, bridge piers may have significant amounts of concrete deterioration which would require extensive concrete removal. It is imperative that the structural integrity of the piers be maintained during the repair process. Guidelines limiting the amount of concrete removal that can be completed at any one time on the pier/bent cap, beams and columns should be adhered. Techniques for concrete removal on pier caps and columns are very similar to those discussed above for bridge decks. The single bottom mat of reinforcing steel in the pier caps provide the majority of the tensile strength in the pier caps between the columns. In the columns, the reinforcing steel provides resistance to axial loads and bending and shear stability for the concrete.

6.16 Replacement of Deteriorated Concrete

Once the unsound concrete is removed, the replacement concrete can be placed. A term used in bridge maintenance to replace unsound concrete in an existing structure is to 'patch' the concrete. A patch is often thought of as a temporary fix, but that can be subjective and dependent on the quality of the patch. If most of the procedures in this manual are followed, many owners consider the patch as a permanent repair. Examples of a temporary patch include ones where saw cutting or hammering was not used, or asphalt or a concrete of non-similar quality of the original material was used. Some owners consider any localized patch as temporary, but a larger repair like a deck overlay as permanent.

6.16.1 Storage

Concrete repair materials should be stored in a dry location at a temperature near 70 °F degrees, but never below 40 °F or above 90 °F. Concrete repair materials that become damp or otherwise defective should not be used and should be removed from the job site. Concrete repair materials that are beyond the manufacturer expiration date should not be used unless the manufacturer has requalified them by actual laboratory testing.

6.16.2 Bonding Coating to Concrete and Rebar

Epoxy bonding coats are applied to dry and clean concrete surfaces in order to bond firmly. Specially formulated resins are also available for damp surfaces. Apply the selected bonding coat to steel bars as shown in Figure 6.38; with a brush working vigorously to ensure that they are evenly covered all around.



Figure 6.38: Epoxy bonding coat

6.16.3 Preparation of Reinforcement

Prior to replacing concrete, exposed reinforcing steel in the removal areas needs to be prepared for the repair. It is important that loose bars be tied at each intersection point, as shown Figure 6.39. Tying the reinforcing bars prevents relative movement of the bars and the concrete in the repair under the action of traffic in adjacent lanes during the curing period.



Figure 6.39: Reinforcement Bars Should be Tied at Intersection Points

If corrosion has reduced the cross-sectional area of the reinforcement by more than 20 percent, extra reinforcement is usually added. The typical method of adding reinforcement is to lap the weakened bars with additional bars to restore the cross-sectional area to its original value. The lap lengths over the undamaged parts of the bar should be the same as those required for new construction (typically 30 times the bar diameter). The lapped bars should be tied at both ends, as a minimum. If lapping is not possible, holes are drilled in the existing concrete and dowels or anchors installed. Mechanical splicers may also be used. The preparation of plain (not epoxy coated) reinforcing steel prior to replacement may include a calcium nitrate rust inhibitor coating of the reinforcement or use of sacrificial anodes (see Figure 9.3) to prevent further corrosion. The preparation of reinforcing steel prior to placement of repair material should include abrasive blast cleaning.



Figure 6.40: Sacrificial Anode Secured to Rebar

6.16.4 Additional or Replacement Rebar

The Contractor shall report to the Engineer any rebar which has 10% or more section loss as a result of corrosion. Additional or replacement rebar shall be provided as instructed by the Engineer. Replacement rebar shall be cleaned to the same standard as the existing rebar. This replacement rebar shall be lapped on the side of the existing bars and be spot welded on one side. It shall be fixed along its length at suitable intervals to prevent sagging. The corroded rebars shall be cleaned and applied with zinc rich primer to prevent further corrosion. The Contractor shall obtain Engineer's approval for the rebar coating prior to proceeding with Maintenance mortar application.

6.16.5 Types of Concrete Replacement

Areas of concrete removal can be repaired in several different ways, including:

- Form and Pour (Recasting) with concrete
- Prepacking dry aggregates and grouting
- Patching with cementitious, or polymer-based materials
- Shotcrete

On overview of each of these four methods will be presented in this chapter. Additionally, there are numerous American Concrete Institute (ACI) publications on repair techniques.

- RAP-1 Structural Crack Repair by Epoxy Injection
- RAP-2 Crack Repair by Gravity Feed with Resin
- RAP-3 Spall Repair by Low-Pressure Spraying
- RAP-4 Surface Repair Using Form-and-Pour Techniques
- RAP-5 Surface Repair Using Form-and-Pump Techniques
- RAP-6 Vertical and Overhead Spall Repair by Hand Application
- RAP-7 Spall Repair of Horizontal Concrete Surfaces
- RAP-8 Installation of Embedded Galvanic Anodes
- RAP-9 Spall Repair by the Preplaced Aggregate Method
- RAP-10 Leveling and Reproiling of Vertical and Overhead Surfaces
- RAP-11 Slab jacking
- RAP-12 Concrete Repair by Shotcrete Application
- RAP-13 Methacrylate Flood Coat
- RAP-14 Concrete Removal Using Hydrodemolition

6.16.6 Preparing Old Concrete for Repair

with Cementitious Materials To ensure an adequate bond and to reduce shrinkage of the new cementitious concrete, the existing concrete needs to be prepared. The first step, after the deteriorated concrete is removed, is to remove all loose particles, oil, grease, laitance, or any other material that would prevent the bonding to the concrete surface. The existing surface should be cleaned by light sand-blasting or other surface abrasion methods.

After cleaning, the concrete surface should be saturated with water spray, for 24 hours, if possible, to create a saturated surface-dry condition. If a 24-hour saturation is not practical, the concrete should receive a water spray for as long as possible. A quick "blessing" of sprinkled water just before the pour will not pre-soak the existing concrete sufficiently. The mix water in the new concrete will

be soaked up by the existing concrete. The result will lead to increased shrinkage cracks and poor bond.

Prior to the pour, any water remaining in the hole will need to be removed by waiting out or air blasting. If standing water remains, the quality of the new concrete will be affected. The compressive strength and durability of the repair will be compromised.

Next, a bonding agent (usually a cement grout or epoxy bonding agent) is well worked onto the concrete surface just before the installation of formwork. It is very important that the bonding agent does not dry out when the repair concrete is recast. Bonding agents are not always required because if the agent dries out, it can actually become a bond breaker.

6.16.7 Form and Pour with Concrete (Recasting)

The form and pour method use forms to repair bridge elements such as beams, columns, and wingwalls (see Figure 6.41 through Figure 6.44). If the formwork installation is lengthy, the use of bonding grout may not be possible, since it will dry out before the concrete is cast. In this case, specially formulated polymer bonding agents may be used which allow more working time.



Figure 6.41: Forming for a Wingwall Repair



Figure 6.42: Manufactured Forms for Columns



Figure 6.43: Forming for a Pier Cap Repair



Figure 6.44: Forming for Deck Repair

Formwork for recasting should be rigid enough to prevent new concrete from sagging away from the existing concrete under the weight of new concrete. The formwork should with stand forces from pumping of the concrete, if pumping is used to transport the concrete. Also, it should

withstand the forces from the external vibrators, if such vibrators are used to consolidate the concrete.

Steel forms have the advantage of being rigid, but they are heavy and not easily handled. Another advantage of steel forms is that they allow heat dissipation of the concrete during the hydration. This will minimize thermal shrinkage and subsequent cracking of the new concrete. Forms are either supported by props or fixed to the structure itself. To repair a vertical patch, a “birds’ mouth” is built into the forms to allow the placement of concrete, as shown in Figure 6.45. Prior to placing the concrete, the forms should be cleaned, sprayed with a form release agent and wetted to prevent absorption of the water used in the concrete. Concrete forrecasting should easily flow and fill all the voids in the form. Since space is limited, use of internal vibrators is usually restricted. Ideally, the mix should be very workable; however, use of water is limited to keep drying shrinkage to a minimum.



Figure 6.45: Birds Mouth Examples

Superplasticizers may be used to provide workability without resorting to water. Good compaction is achieved by placing the concrete in small amounts and vibrating effectively as the work proceeds. External vibrators are often used. When recasting deck soffit, sometimes it is possible to drill through the deck and place the concrete from the above. For difficult areas to access or in locations with considerable rebar, self-consolidating concrete could be used.

6.16.8 Consolidation

In some types of construction, the concrete is placed in forms, and then consolidated. Consolidation compacts fresh concrete to mold it within the forms and around embedded items and reinforcement and to eliminate stone pockets, honeycomb, and entrapped air. It should not remove significant amounts of intentionally entrained air. Vibration, either internal or external, is the most widely used method for consolidating concrete (see example in Figure 6.46). When concrete is vibrated, the internal friction between the aggregate particles is temporarily destroyed and the concrete behaves like a liquid; it settles in the forms under the action of gravity and the large entrapped air voids rise more easily to the surface. Internal friction is reestablished as soon as vibration stops.

An option to consolidation for these types of repairs has been self-consolidating concrete which was discussed earlier in this chapter. This type of concrete is very flowable and needs no vibration. Excess use of a vibrator can actually damage the composition of the concrete.



Figure 6.46: Pumping with Elephant Trunk and Consolidating with Vibrator

6.16.9 Prepacking Dry Aggregates and Grouting

The surface preparation and formwork for this technique is basically the same as those for the form and pour (recasting) method. The only difference is that dry aggregate is packed in the space behind the form so that it fills the space completely. Subsequently, grout is pumped from the lowest to the highest point to fill the space between the aggregate. Typically, a uniform size aggregate is used. An advantage of prepacking dry aggregate and grouting is that the overall shrinkage of the repair is eliminated.

6.16.10 Horizontal Patching

Horizontal patching does not require formwork unless the patch is a full depth type patch, for example on a bridge deck. These larger type patches are covered in Chapter 9. Small concrete areas up to 2 inches in depth can be patched with trowel-applied cement mortar regardless of the type of surface. To begin, clean the surface of the existing concrete by abrasive blast cleaning and bring it to a saturated surface dry condition. Subsequently, a conventional bonding agent may be used with caution as it can dry out (or prematurely set-up). Since no formwork is required, patching can be done immediately after the application of the bonding agent, thus preventing the bonding agent from drying out. The patch mortar should be firmly applied into place so that entrapped air is excluded as much as possible. The closer the physical properties of the patch material are to the existing material, the better. It is also important to minimize the shrinkage of the patching material. This can be done by using the exact amount of mixing water and ensuring the cement and water are well mixed. The inclusion of latex in a concrete mortar will help reduce the amount of water required for workability and also reduce the permeability of the patch. Other performance-proven additives can be used to reduce setting time and increase strength. The aggregate can be adjusted in the mix design for thin patches.

Proper curing is important for all cement concrete. Thin patches present a particularly difficult problem because they dry out or cure quickly. The existing concrete will tend to absorb the moisture in the patching material. If exposed to the sun or wind, the moisture evaporates. It is

recommended that thin patches be cured by covering with moist burlap and polyethylene sheet on top. Without the polyethylene sheet on top, the burlap will dry out faster, doing more harm than good.

To protect the rebar from additional corrosion a corrosion inhibitor can be brushed onto the steel. Furthermore, galvanic anodes that contain zinc can be tied onto the rebar, as shown in Figure 6.47. The zinc contained within the anode will protect the steel from further corrosion.

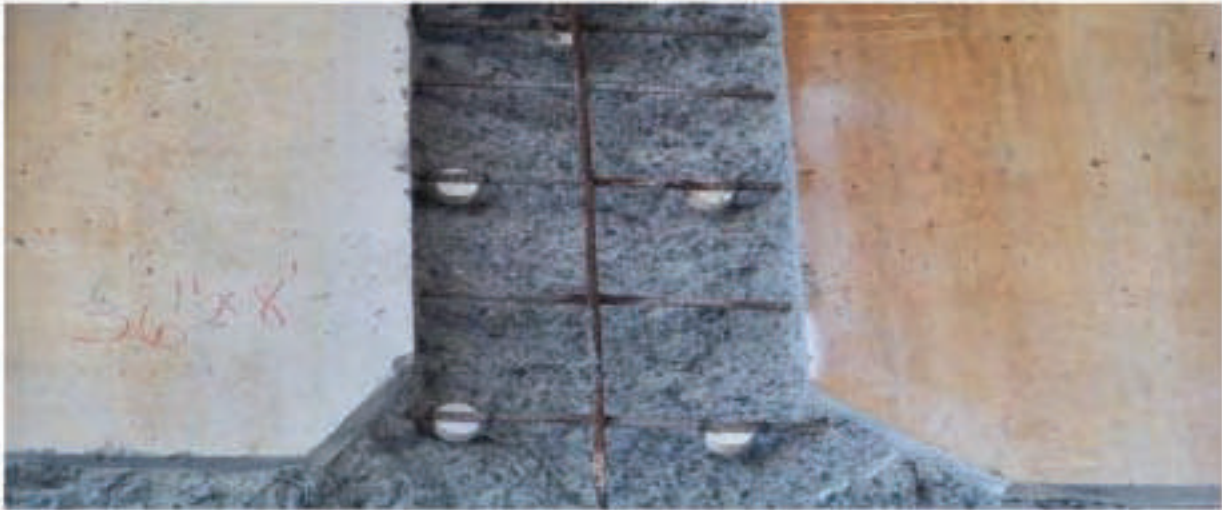


Figure 6.47: Galvanic Anodes Placed into a Patch of a Pier Cap

6.16.11 Non-Portland Based Concretes

In addition to conventional cement-based material, there are two types of materials that are used for patching concrete: quick-setting hydraulic cements and polymer materials.

There are a variety of proprietary brands of quick-set hydraulic and polymer material that are designed to gain strength rapidly. Each proprietary material has a number of unique properties favorable for bridge maintenance work. For some materials, the patch may be opened to bridge deck traffic after about one hour. Quick-setting hydraulic cements most commonly used are formed by the reaction between magnesium-oxide and a soluble phosphate. These materials have material properties that exceed those of Portland-based cements, but they are more expensive and require additional care during the mixing and installation. "Mag-Phosphates", as they are commonly called, can reach a compressive strength of 2,100 psi in less than an hour. Ultimate strengths can reach 10,000 psi. These materials also have good adhesion properties and are marketed for vertical and overhead applications as well.

Care must be taken during the mixing not to deviate from the recommended amount of mixing water. Even a small deviation will severely impact the durability of the cement. An example of a failed rapid-setting mix is shown in Figure 6.48. Aggregates used to make the concrete should not be calcareous (containing limestone) as these aggregates react with the cement to form a gel that deteriorates the concrete in presence of water over time. The rapid hardening process creates considerably more heat during hydration than a Portland-based reaction. Failure to control the heat of hydration will cause severe shrinkage cracking. In order to limit the amount of heat generated,

Mag-Phosphate cements should not be placed in lifts greater than 4". Modified formulations have been developed for repairs done in hot weather.



Figure 6.48: Failure of Rapid-Setting Mix

6.16.12 Polymer Concrete

Several types of polymers, such as epoxy, polyesters, methyl acrylates, and ureathanes, have been developed for the repair of concrete. Polymers consist of a resin and a hardener. These two components are proportioned and mixed at the site. Aggregate is added to improve the compressive strength and because polymers are expensive, reduce the amount of material used. The mix solidifies upon a rapid chemical reaction. Priming the existing surface with the polymer material (without aggregate) provides good bond. Polymer materials liberate heat, set, and gain strength very rapidly. Their pot life is very short, especially when they are used in large quantities or in hot weather. This is because more heat is liberated and this in turn expedites setting of the polymer. On the other hand, their strength gain usually slows down substantially when the air temperature is 65 °F or below, and they may not gain sufficient strength.

Each type of polymer has material characteristics that make them unique from cement. Two of the more commonly discussed are viscosity and elongation. Viscosity refers the ability of the material to flow. A very viscous material like molasses has a high viscosity number. Water is a low viscosity material. Polymer materials can be made fluid enough to flow into place by gravity so inaccessible places can be filled with them without compaction.

Advantages and Disadvantages of Polymer Concrete

Advantages of Polymer Concrete	Disadvantages of Polymer Concrete
<ul style="list-style-type: none"> • Rapid cure times. Polymer concrete has a liquid working temperature between 60 °F and 85 °F. The maximum workable temperature of this type of concrete is 150 °F. However, at higher temperatures, these materials can set in minutes and be hardened in the mixer. • High tensile and flexural strengths. These materials can bend and twist more than traditional concretes. • Good adhesion to most surfaces. Polymers are sticky and will develop strong bonds to properly prepared surfaces, such as concrete and steel. • Good long-term durability with respect to freeze-thaw cycles. The material is very dense so it does not allow water into the pours as it can with Portland-based materials. • Polymer concrete does not need to be sealed. • Low permeability to water and aggressive solutions such as de-icing salts. • Good resistance against corrosion. 	<ul style="list-style-type: none"> • Cost. Polymer concrete is expensive. Too expensive for use in large placements. • Lower compressive strength. Though polymers can be formulated to 12,000 pounds of pressure per square inch, or psi, the elastomeric concretes used in maintenance repairs are typically of lower strengths, typically 1,500 to 2,500 psi. • Polymer concretes are sticky and hard to manipulate with conventional tools shovels and trowels. • Not compatible with water. Most polymers are negatively affected by water. Both the substrate and the aggregate should be free of any water. Some polymers are affected by high humidity. • Safety. Some polymers are relatively safe to use, others can create hazardous conditions if mixed improperly. • Training. Crews should receive specific training and review the Material Safety Data Sheet (MSDS) before mixing and placing polymer materials. • Temperature limitations. Most, but not all polymers, cannot be used in below freezing temperatures. Also, at high temperatures, the material may set too quickly. • Surface Preparation. Proper surface preparation is critical for quality repair

6.16.12.1 Aggregates and Cements for Polymer Concrete

Aggregates - Polymer materials expand and shrink much greater than concrete when ambient temperature fluctuates. This unfavorable characteristic of polymer materials will cause debonding and separation of the patch from the old concrete in time. The addition of aggregate helps to alleviate this problem. Aggregates must be dry and free of fine particles. Moisture negatively affects the polymer reaction and additional polymer would be necessary if too many fines are included. A hard durable 3/8-inch sized aggregate is used in extending polymer concrete. A more angular aggregate produces a stronger matrix so that less polymer is needed.

Polymer Cement - Polymer cement mortar shall conform to the requirements of the specifications shown in Table 6.1 or equivalent ASTM Specifications.

Table 6.3: Specifications of Polymer Cement

Property	Test Method	Unit	Specification
Thermal Expansion	ASTM C531	mm/mm C	2.0 x 10 ⁻⁵
Slant Shear Bond to Concrete	ASTM C882	N/mm ²	1.5 /above
Compressive Strength (7days x 20)	ASTM C579	N/mm ²	20 /above

6.16.12.2 Material Properties of Polymer Concrete

Compression Strength - Strength is an important characteristic of concrete, as it is often used as a structural foundation material. The compressive strength of elastomeric polymer concrete can range from 1,500 to 12,000 psi. Exact compression strength varies depending upon the brand of polymer concrete and what type of material is used as a binder. Compressive strength increases over time. Polymer Methyl Methacrylate Concrete can gain compressive strengths of 8,000 psi after one hour of setting and 12,000 psi after seven days of setting.

Corrosion Resistance - A main advantage of polymer concrete is resistance to a wide range of solvents and oils; it is also resistant to acids (pH level ranging from 0 to 7). Polymer concrete is resistant to a variety of acids that can eat away concrete over time, de-icing salts, hydrochloric acid, formic acid, and ammonium sulfate. Polymer concrete does not corrode in the presence of hydrocarbons, such as acetone, gasoline, ethane, methanol, petroleum, diesel and vegetable oils. This type of concrete offers excellent resistance against water.

Flexibility - Traditional concrete is vulnerable to shearing forces that can cause cracks and a decline in structural integrity. Resistance to these forces is known as flexural and tensile splitting strengths. Polymer concrete offers greater resistance to cracking. Polymer concrete has a flexural strength of around 3,500 psi after eight hours compared to less than 1,000 psi for normal concrete. Tensile strength is around 2,000 psi after the material has been allowed to set for seven days, whereas normal concrete typically has just a few hundred psi. Alternatively, for patching, there are numerous proprietary brands of quick-setting cement. These can be used for patching, but a normal requirement is that they be placed in relatively thin layers because of the heat they generate. The manufacturer instructions should be followed.

6.16.12.3 Mixing Polymers

Each of the two parts, the resin and the hardener, should each be stirred separately for about 3 minutes or as per manufacturer recommendation prior to mixing together. This is necessary to re-mix any settled particles. Polymers are mixed in a ratio, typically one part resin to one part hardener, though some polymers may be formulated differently. Care should be taken to ensure the entire amount of resin and hardener are removed from each container. These combined parts should be mixed again for 5 minutes before adding the aggregate. Add the aggregates to the polymer. This helps to ensure the aggregates are coated with the polymer and thereby a more durable patch. Mixing should continue for another 5 minutes. Be sure to closely follow the manufacturer instructions. Examples of mixing for smaller patches and for larger patches are presented in Figure 6.12 and Figure 6.13, respectively.



Figure 6.49: Five Gallon Bucket Mixing Method for Small Patches



Figure 6.50: Power Mixers for Larger Patches

6.16.12.4 Placing Polymers

All surfaces should be dry and free of any contaminants. It is critical the surfaces are dry. Adding a primer, as shown in Figure 6.14, will allow the polymer to bond better to the existing concrete. Unlike cementitious-based repair materials, the bond capability of polymer-based materials is significantly reduced in the presence of water. The surfaces should be clean of loose materials, rust, and oil free. The use of an electric leaf blower to air clean surfaces is recommended, as shown in Figure 6.15. Unlike combustible motors driven air compressor that rely on water and oil filters to provide clean air, the air from electric leaf blowers will not contain these contaminants.



Figure 6.51: Priming a Patch with Polymer

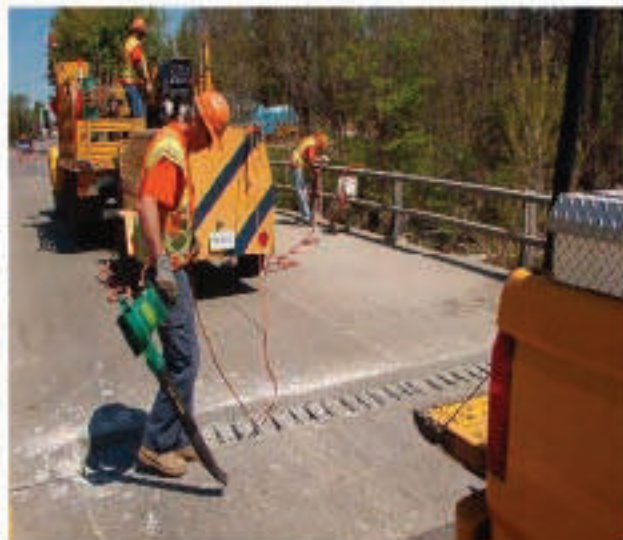


Figure 6.52: Using a Leaf Blower to Clean Surfaces

For a neat looking repair, duct tape is used to protect the perimeter concrete from spilled polymer that becomes difficult to remove once hardened. An example is shown in Figure 7.16.



Figure 6.53: Taping the Perimeter for a Clean Look

As polymers are generally not an electrically conductive material, the use of sacrificial anodes to protect the rebar is ineffective.

6.16.13 Polymer Mortar

Hand applied Mortar (Patching) Maintenance is performed to restore small areas where sound concrete is damaged by spalling, scaling and/or impact. This method of Maintenance is generally applied using trowel and requires minimum or formworks. The patch thickness is limited to a maximum of 100 mm depth of hollow surface.

Hand applied Mortar Maintenance is classified into two types as shown in Figure 6.17, considering defective area and surface. Hand applied Mortar (A) is for defects without exposed rebars while Hand applied Mortar(B) is applied to surfaces with exposed rebars.

Hand applied Mortar Maintenances may be composed of Polymer cement mortars or non-shrinkage cement mortars depending on the type of patching, location and extent of damage.

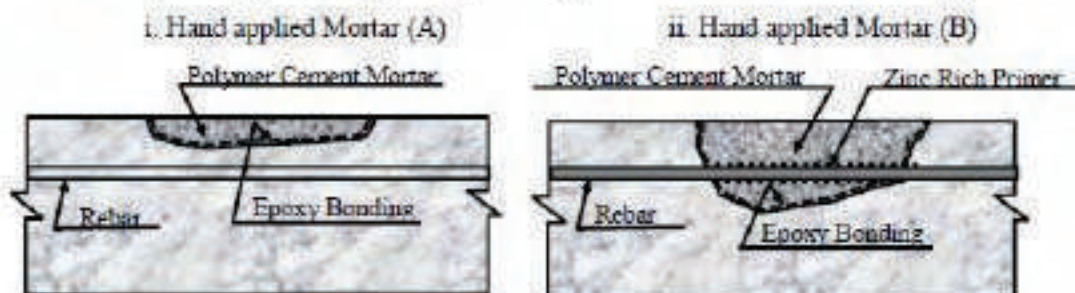


Figure 6.54: Hand applied Mortar

Hand applied Mortar (A) is applicable to surfaces without exposed rebars, having defective widths of up to 300mm and depths of up to 50 mm.

Hand applied Mortar (B) is for surfaces with exposed rebars, with defective widths between 300 mm and 600 mm, and up to 100 mm depths.

To keep good bond ability to the existing concrete, Polymer cement mortar is applied in Hand applied Mortar (A) and Mortar (B).

6.16.13.1 Filling up Maintenance Material

The mortar should be placed in layers of about 20 mm thickness. Compact each layer thoroughly over the entire surface using a wooden trowel or hammer. Generally, there should be no time delays between the placing and compacting of layers. The patching to the surrounding concrete is performed using a form material, and then hammered using a mallet, wood float or steel trowel. The mortar shall be mixed using equipment (normally a force action mixer) approved by the Engineer.

The mixing liquid shall be added to the dry components and thoroughly mixed to achieve a uniform consistency, unless otherwise approved by the Engineer. The mortar shall then be applied to the bonding agent using hand packing and trowel to the satisfaction of the Engineer-in-Charge. The textured finish of the final Maintenance mortar layer shall match the finish on the existing interior surface.

The Maintenance mortar application shall be built up to the original surface profile in layers not exceeding 20 mm and the final layer shall not exceed 15 mm, unless otherwise commended by the manufacturer and approved by the Engineer-in-Charge. The Engineer may approve Maintenance mortar application thickness of up to 50 mm for lightweight mortars, provided the Maintenance mortar manufacturer can furnish a technical data to justify a layer thickness of greater than 20 mm.



Figure 6.55: Filling up in Patching

6.16.14 Shotcrete

Shotcrete, or pneumatic applied mortar, is used to repair and restore the surface of concrete bridge elements. It is conveyed through a hose and nozzle and pneumatically projected at high velocity onto a surface. It contains the same cement, aggregate and water as concrete except that there are no coarse aggregates in the mix. The mix has high cement content and low water/cement ratio. Addition of silica fume, fly ash and/or slag enhances performance of shotcrete. Shotcrete can contain silica

fume at a minimum of 7 percent by weight of cement, flyash at a maximum of 20 percent and slag at a maximum 50 percent. Steel or synthetic fibers have also been used to increase compressive strength and decrease potential for cracking. When properly applied, mortar is dense, durable and strong. It is also reported to have superior bonding characteristics. A typical shotcrete machine is shown in Figure 6.56.



Figure 6.56: Typical Shotcrete Machine

The applicator controls the quality of the work. The applicator controls the amount of mix water that is added to the dry mix and the amount of material applied to the surface, as shown in Figure 7.20. A field verification jig is commonly used to assess the skill level of the applicator prior to performing the work. Qualification of applicators and test panel should be required.



Figure 6.57: Mix Water Added at Nozzle

On vertical and overhead surfaces, since forming is not required, pneumatic mortar can be a useful alternative in bridge restoration. Large surface areas can be repaired in relatively short periods of time. Examples are shown in Figure 6.21 and Figure 6.22.



Figure 6.58: Shotcrete Repair of an Invert of a Corrugated Metal Culvert



Figure 6.59: Shotcrete Repair of a Pier

Preparation of the existing surface is an important part of the repair. All deteriorated concrete should be removed, as previously described in this manual, with the edges sloped about 45° to prevent entrapment of rebound material. Existing concrete and reinforcing steel should be cleaned by abrasive blast cleaning. Reinforcement of the patch with welded wire fabric or small diameter wire mesh is advised for depths exceeding 3 inches. The limits of the existing concrete face should be exceeded, if necessary, to obtain a minimum 2 inches cover on existing reinforcement.

Mortar should not be used in thicknesses of less than 1 inch. The existing surface is wetted as needed so it will not absorb water from the pneumatic mortar. The natural handgun finish is preferred from bond and durability standpoints. Scraping or cutting may be used to remove high points and material that has exceeded the limits of the repair after the mortar has become still enough to withstand the pull of the cutting device. Troweling or other finishing is discouraged as it has a tendency to disturb the bond. Curing is very important for the rich mixes and thin sections used with pneumatic mortar. Seven days of water curing is generally advisable to promote good hydration of the cement, keep the mortar cool in hot weather, and prevent early shrinkage that may disturb the bond. Any defects in the shotcrete, such as the "Bug-holes" shown in Figure 6.60, should be repaired prior to curing.



Figure 6.60: Repair "Bug-Holes" after Placing Shotcrete

6.16.15 Small Volume Pneumatic Concrete

Hand-held machines have been developed for small repairs. These machines are effective in re-pointing laid up stone, re-mortaring granite curbs, and re-mortaring bridge rail postfoundations. An example of a small volume shotcrete tool is shown in Figure 6.24.



Figure 6.61: Small Volume Shotcrete Tool

A concrete mix with fine aggregates (mortar sand for instance) is added to a hopper on the tool. Water is added to the dry mix at the nozzle. The success of this application is also dependent on the skill of the applicator. The application, if done properly, provides for a dense concrete with good adhesion. An example of a small volume shotcrete tool application is shown in Figure 6.62.



Figure 6.62: Re-Pointing a Laid-Up Stone Parapet with a Small Volume Shotcrete Tool

Some manufacturers offer polymer modified mixes that can be pneumatically applied. These mixes offer the advantages of polymer concretes along with the benefits gained from pneumatically applying concrete to a vertical or overhead surface.

6.17 Finishing

Concrete that will be visible, such as approach slabs, bridge decks, and sidewalks, often needs finishing. Concrete slabs can be finished in many ways depending on the intended service use. Some surfaces may require only strike off and screeding (see Figure 6.26) to proper contour and elevation, while for other surfaces a broomed, floated, or troweled finish may be specified (see Figure 6.27). In slab construction, screeding or strike off is the process of cutting off excess concrete to bring the top surface of the slab to proper grade. A straight edge is moved across the concrete with a sawing motion and advanced forward a short distance with each movement. A challenge for concrete finishers can be access.



Figure 6.63: Use of a Screed to Finish Elevation on a Sidewalk



Figure 6.64: Trowel Finish on a Deck Patch

Jointing is required to eliminate unsightly random cracks. Contraction joints are made with a hand groover or by inserting strips of plastic, wood, metal, or preformed joint material into the unhardened concrete. Saw cut joints can be made after the concrete is sufficiently hard or strong enough to prevent raveling. After the concrete has been jointed, it should be floated with a wood or metal hand float or with a finishing machine using float blades. This embeds aggregate particles just beneath the surface; removes slight imperfections, humps, and voids; and compacts the mortar at the surface in preparation for additional finishing operations. Where a smooth, hard, dense surface is desired, floating should be followed by steel troweling. Troweling should not be done on a surface that has not been floated; troweling after only bull floating is not an adequate finish procedure. A slip-resistant surface can be produced by brooming before the concrete has thoroughly hardened, but it should be sufficiently hard to retain the scoring impression.

6.18 Curing

Curing plays an important role on strength development and durability of concrete. Curing takes place immediately after concrete placing and finishing, and involves maintenance of desired moisture and temperature conditions, both at depth and near the surface, for extended periods of time. An example of a wet cure for a small deck patch is shown in Figure 6.28. Pre-soaked burlap covered with polyethylene sheets is used to keep the concrete wet. Properly cured concrete has an adequate amount of moisture for continued hydration and development of strength, volume stability, resistance to freezing and thawing, and abrasion and scaling resistance.



Figure 6.65: Wet Cure of a Small Deck Patch Using Pre-Soaked Burlap

The length of adequate curing time is dependent on the following factors:

- The type of cementitious materials used
- Mixture proportions
- Specified strength
- Size and shape of concrete member
- Ambient weather conditions and
- Future exposure conditions

Slabs on ground (e.g. pavements, sidewalks, parking lots, driveways, floors, canal linings) and structural concrete (e.g. bridge decks, piers, columns, beams, slabs, small footings, cast-in-place walls, retaining walls) require a minimum curing period of seven days for ambient temperatures above 40 °F).

The American Concrete Institute (ACI) Committee 301 recommends a minimum curing period corresponding to concrete attaining 70 percent of the specified compressive strength. The often specified 7-day curing commonly corresponds to approximately 70 percent of the specified compressive strengths.

70 percent strength level can be reached sooner when concrete cures at higher temperatures or when certain cement/admixture combinations are used. Similarly, longer time may be needed for different material combinations and/or lower curing temperatures. For this reason, ACI Committee 308 recommends the following minimum curing periods:

- ASTM C 150 Type I cement: 7 days
- ASTM C 150 Type II cement: 10 days
- ASTM C 150 Type III cement: 3 days
- ASTM C 150 Type IV or V cement: 14 days

There are three main functions of curing:

1. To maintain mixing water in concrete during the early hardening process, which is accomplished by:
 - **Spraying and fogging:** Spraying and fogging are used when the ambient temperatures are well above freezing and the humidity is low. Fogging can minimize plastic shrinkage cracking until the concrete attains final set.
 - **Saturated wet coverings:** Wet coverings saturated with water should be used after concrete has hardened enough to prevent surface damage. They should be kept constantly wet.
 - **Left in Place Forms:** Left in place forms usually provide satisfactory protection against moisture loss for formed concrete surfaces. The forms are usually left in place as long as the construction schedule allows. If the forms are made of wood, they should be kept moist, especially during hot, dry weather.
2. To reduce the loss of mixing water from the surface of the concrete:
 - **Cover concrete with impervious paper or plastic sheets:** Impervious paper and plastic sheets can be applied on thoroughly wetted concrete. The concrete surface should be hard enough to prevent surface damage from placement activities.
 - **Apply membrane-forming curing compounds:** Membrane-forming curing compounds are used to retard or reduce evaporation of moisture from concrete. They can be clear or translucent and white pigmented. White-pigmented compounds are recommended for hot and sunny weather conditions to reflect solar radiation. Curing compounds should be applied immediately after final finishing.
3. Accelerating strength gain using heat and additional moisture:
 - **Live steam:** Live steam at atmospheric pressure and high-pressure steam in autoclaves are the two methods of steam curing. Steam temperature for live steam at atmospheric pressure should be kept at about 140 °F or less until the desired concrete strength is achieved.
 - **Heating coils:** Heating coils are usually used as embedded elements near the surface of concrete elements. Their purpose is to protect concrete from freezing during cold weather concreting.
 - **Electrical heated forms or pads:** Electrical heated forms or pads are primarily used by the precast concrete producers.
 - **Concrete blankets:** Concrete insulation blankets are used to cover and insulate concrete surfaces subjected to freezing temperatures during the curing period. The concrete should be hard enough to prevent surface damage when covering with concrete blankets.

Curing in either cold or hot weather requires additional attention. In cold weather, some of the procedures include heated enclosures, evaporation reducers, curing compounds, and insulating blankets. The temperature of fresh concrete shall be above 50 °F. The curing period for cold weather concrete is longer than the standard period due to reduced rate of strength gain. Compressive

strength of concrete cured and maintained at 50 °F is expected to gain strength half as quickly as concrete cured at 73 °F. In hot weather, curing and protection are critical due to rapid moisture loss from fresh concrete. The curing actually starts before concrete is placed by wetting substrate surfaces with water. Sunscreens, windscreens, fogging, and evaporation retardants can be used for hot weather concrete placements. Since concrete strength gain in hot weather is faster, curing period may be reduced.

Concrete bridges require a high standard of curing to achieve the low permeability required for protection of steel reinforcement. Standard recommendations for curing bridge decks is moist curing for a minimum of 7 days for concrete mixtures containing only Portland cement and as long as 14 days when supplementary cementing materials are included in the concrete mixture. Some states also require the application of curing compound upon removal of the moist curing methods.

Curing can be problematic for bridge maintenance staff when patching existing concrete decks. Traffic typically needs to be restored in less than 7 days. For this very reason there are specific deck patching products that cure quickly, typically 2 to 4 hours. These patch materials contain special additives or polymers to help in curing. A product popular with many bridge owners allows curing and restored bridge traffic in 45 minutes.

Typical moist curing for bridge decks requires the application of adequate quality water saturated burlap or other approved absorptive material covered with minimum 6 mil plastic covering (see example shown in Figure 6.29). The temperature of the saturated materials should be within 20 °F of the temperature of the in-place concrete. White plastic is used to reflect solar radiation, reducing the temperature rise beneath the plastic, while cold temperatures (less than 50 °F) may allow the use of black plastic to add heat to the system.



Figure 6.66: Wet Cure with Pre-Soaked Burlap for a Large Deck Repair

CHAPTER VII: CRACKS AND OTHER REPAIR OF CONCRETE

7.1 Crack Repair

The cause of cracking should be determined before repairs are undertaken. Structural defects should be remedied before filling. A crack caused by settlement, flexural or diagonal tension will recur if the problem is not corrected.

Cracks can be repaired one at a time or done by a flood coat (also called: topical application). Small cracks occurring over large areas can be sealed with a crack filler applied to the entire surface. These materials have very low viscosity and are able to flow deep into the cracks to seal out moisture. If the cracks are full-depth, it will be necessary to seal off the bottom surface. As these cracks cannot be cleaned out, the bond to the existing concrete may be compromised. 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.

Large, open cracks that are dormant or not “working” may be treated by injection. Typically, an epoxy or urethane-based product is used. The injection method is laborious but is effective in treating vertical cracks. Cracks on horizontal surfaces can be packed with mortar sand, and then a low viscosity polymer is fed into the sand. A lower viscosity product, such as a urethane, should be used for this type of repair.

If cracks are open and appear to be “working”, or if a crack has been sealed but has cracked again, a flexible sealant, such as asphalt crack sealer, should be used. This usually requires routing a groove into the crack to act as a reservoir to hold the proper amount of sealant. A backer rod may also be necessary.

Repair of cracks in concrete reduce the rate of deterioration due corrosion of reinforcement and the resulting volumetric change associated with the corrosion product which may lead to further cracking and spalling. Selection of the repair method should be based on an understanding of the causes of the cracking. It is important to identify, photograph and record details of cracks during the visual inspection (e.g., location, extent and orientation). A crack gauge, feeler gauge or machinists steel ruler can be used to measure the width and a piano wire can be used to measure the depth of the crack.

Figure 7.1 illustrates a variety of crack types and their cause. If cracks are minor (i.e., less than 0.2 mm wide), autogenous healing can sometimes close the crack. Autogenous healing normally takes place within the first few days of the life of the structure and can be improved by keeping the structure moist, particularly when it is kept under compression.

For precast pre-stressed concrete elements, the crack width at the concrete surface should not exceed 0.1 mm.

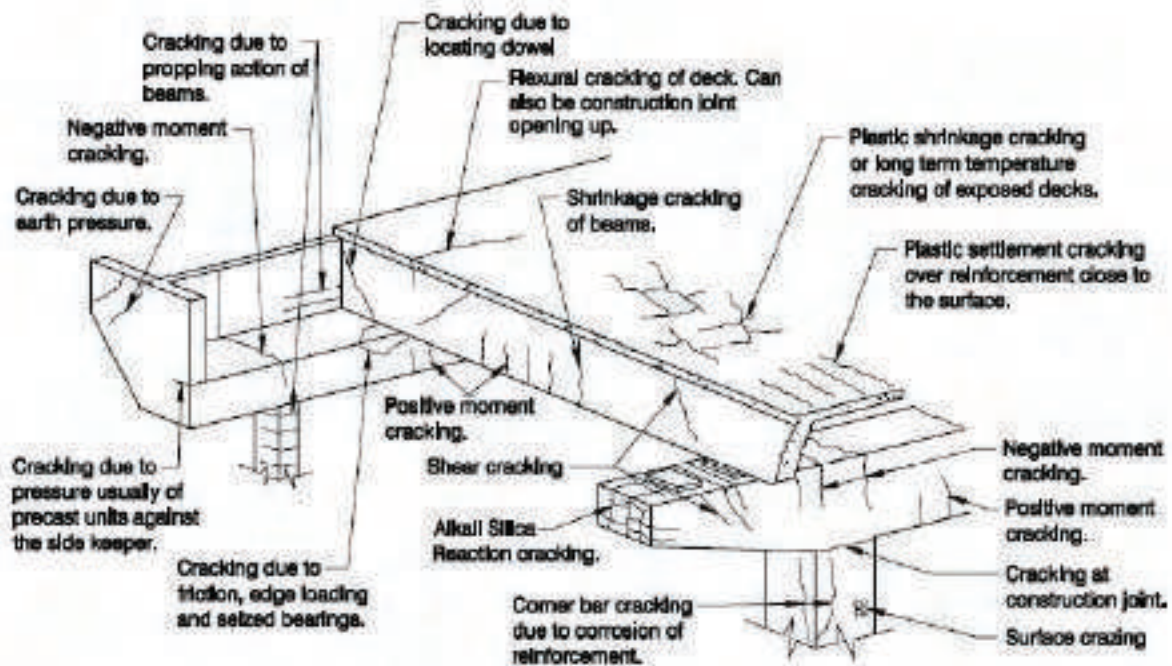


Figure 7.1: Various types of cracks in bridges

7.2 Typical Crack Repair Methods and Applications are Described Below:

7.2.1 Resin Injection of Cracks

Low-Pressure Injection: Proprietary injection kits are used with firm and sustained hand pressure until no further resin can be injected into the crack.

Multicomponent Injection: Procedures are based on the results of testing to confirm the discharge pressure and ratio of adhesive components to be used.

7.2.2 Routing and Sealing Cracks

Flexible sealing can repair "working" cracks. The technique consists of: 1) Routing the crack (see example routers in Figure 7.2 and Figure 7.3), 2) Cleaning and drying the crack, and 3) Filling the crack with a suitable field-molded flexible sealant (see Figure 7.4).



Figure 7.2: Crack Router for Preparing Horizontal Cracks



Figure 7.3: Routing Longitudinal Cracks

Routing and sealing may also repair both narrow and wide cracks that are dormant (non-working) cracks. The method involves enlarging crack along its exposed face and sealing it with a suitable joint sealant (see Figure 7.4). This is the simplest and most common technique for crack repair. It can be executed with relatively untrained labor. 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 tools. A minimum surface width of 1/4 inch is desirable. Repairing narrow grooves is difficult. The surface of the routed joint should be cleaned with an air jet and permitted to dry before placing the sealant.



Figure 7.4: Placing Crack Sealing into "Working Cracks"

7.2.3 Flooding Application of Crack Sealers

Relatively wide, dormant cracks in bridge decks are effectively repaired through gravity fill polymers such as high-molecular-weight methacrylates, ureathanes, 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. Usually, sand is spread onto the wet, uncured polymer to provide adequate skid resistance (shown in Figure 7.5). Failure to do so will cause slippery conditions when wet. The material remaining on the deck will abrade under traffic. Though the deck may be unsightly during the process, no harm is done to the concrete.



Figure 7.5: Spreading Mortar Sand onto Topical Crack Sealer

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

7.2.4 Sealing Concrete Cracks by Epoxy Injection

Sealing cracks by a process of injecting epoxy into the crack is very effective in bridge maintenance.

The process of determining the feasibility of epoxy injection of cracks includes:

- Consulting with engineer to determine if feasible. For example, if there are so many cracks that the structural integrity of the element is too compromised for this type of repair.
- Determine the size and depth of the cracks.
- Determine if the cracks are active.

Examples of epoxy injection are presented in Figure 7.6 and Figure 7.7. Before a method is identified for repairing a crack, it must be determined if the crack is working (i.e., active or moving) or not. If the crack is working it should be filled with flexible material. If it is passive a bonding material such as an epoxy can be injected into the crack. It is also necessary to determine if the crack is full depth. This may be difficult if the crack is in an abutment.

Narrow cracks that are dormant (i.e., non-working) may be effectively sealed by epoxy injection. The procedure can be applied to both horizontal and vertical surfaces. Cracks as narrow as 0.002 inch can be sealed and bonded by the epoxy injection. The procedure has potential to provide structural repair (increase stiffness and strength) in addition to sealing the crack.



Figure 7.6: Epoxy Injection Repair (Horizontal Surface)

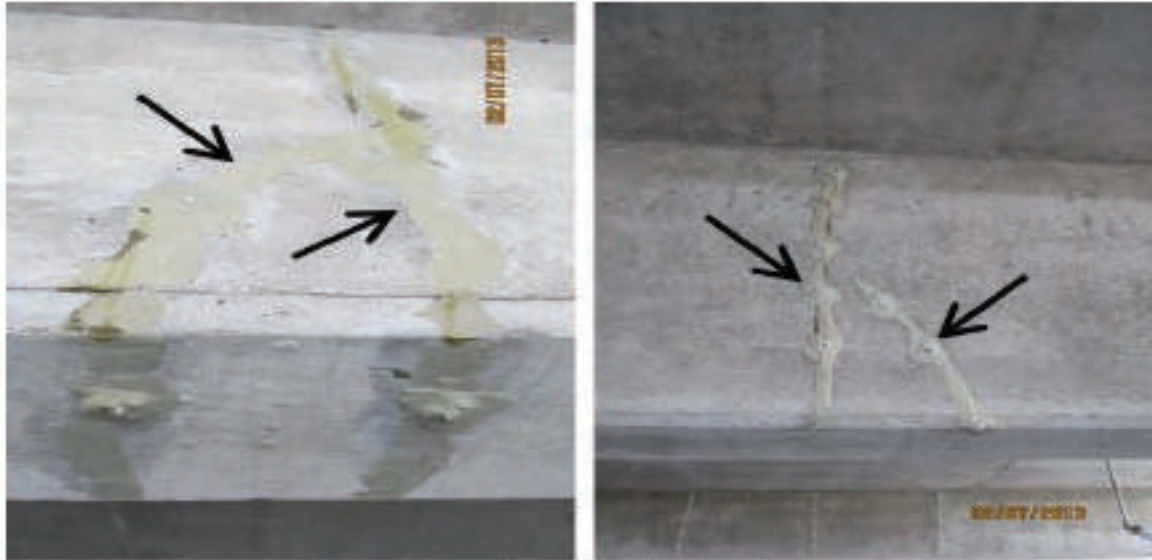


Figure 7.7: Epoxy Injected Cracks (Horizontal and Vertical Surfaces)

The technique requires:

1. Cleaning the cracks
2. Sealing of the crack surface
3. Installation of entry and venting ports
4. Preparation of epoxy
5. Injecting the epoxy
6. Removal of surface seal

These procedures are discussed in greater detail below.

2. Cleaning the cracks: Before injection, the interior of the crack should be cleared of all dust, dirt, oil, grease, or fine particles of concrete that could prevent epoxy penetration and bonding. Harsh chemicals or detergents should not be used to clean the cracks because they may compromise the ability of the epoxy to bond to the concrete.
3. Seal the crack surfaces: The exterior of the cracks should be sealed and allowed to harden to prevent the injected epoxy from leaking out of the crack. Cracks can be sealed by applying an epoxy, polyester, or other appropriate sealing material to the surface of the crack. For cracks that extend through the entire member section, the opposite side of the injection should be sealed as well. If the cracks on each side do not connect, epoxy injection should be performed on each side individually. If extremely high injection pressures are needed, the crack can be cut out to a depth of 1/2 inch and width of 3/4 inch in a V-shape, filled with epoxy and struck off flush with the surface.
5. Install the entry and venting ports: There are two general methods that can be used to install the entry and venting ports; surface mounted and socket mounted. Entry ports are typically tube devices that allow the pressurized epoxy resin to be pumped into the crack (see Figure 8.8). The entry port spacing is typically at 8 inches on center but can be increased for wider cracks. Port spacing depends on the crack width and the amount of pressure applied, however the spacing should be limited to the thickness of the repaired member if the cracks pass all the way through. Surface-mounted entry ports are normally adequate

for most cracks, but socket-mounted ports are used when cracks are blocked, such as when calcified. In some cases, it may be necessary to drill holes approximately 3/4 inch in diameter and 1/2 to 1 inch below the surface of the crack to place the entry or exhaust port.

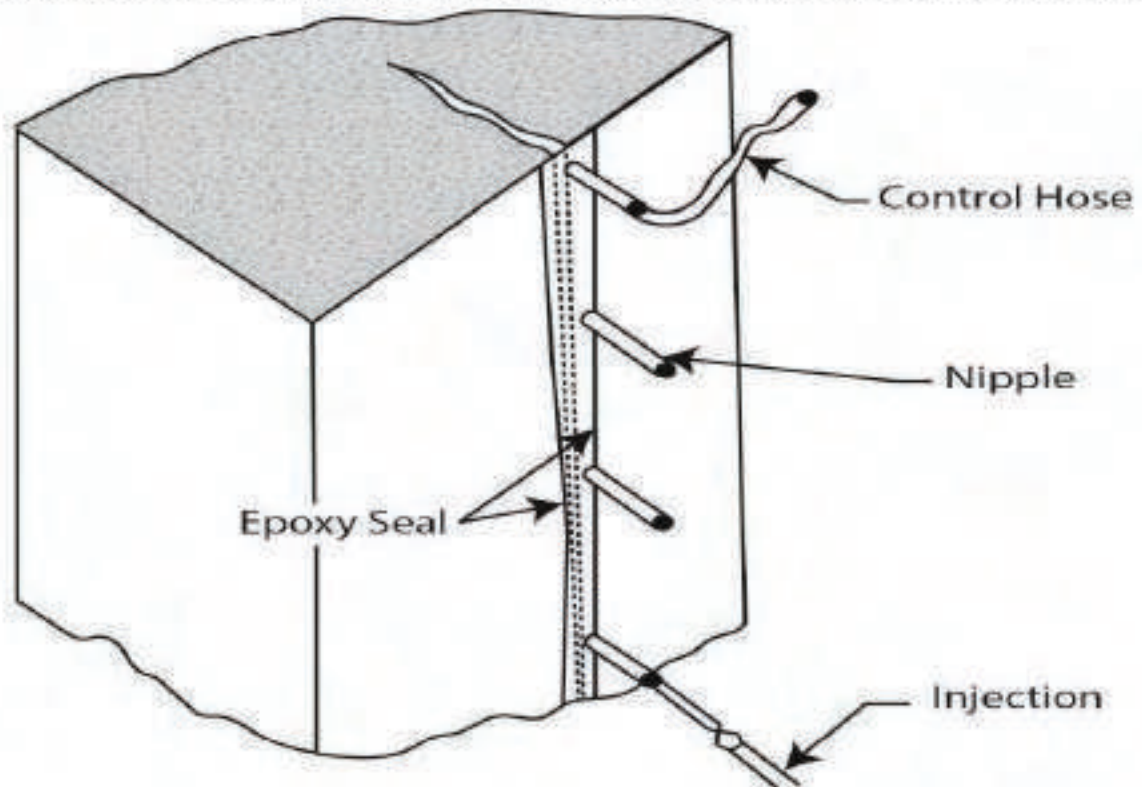


Figure 7.8: Epoxy Injection Schematic (Vertical Surface)

6. Inject Material: The injection progresses from port to port, normally starting at the lowest point, and continuing until epoxy is extruded from the next port. The distance between the ports should not exceed the expected penetration depth. 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. This technique may also be used to fill isolated voids or delamination in concrete. In this case, injection pressure must not be too high. This is not, however, considered as permanent as removing and replacing the concrete.

7.2.5 Repair of Inactive Cracks

Inactive or dormant cracks are cracks, which are not opening and closing or extending with time. A protective coating system can be used to repair inactive non-structural cracks with a maximum width of 0.2 mm, provided such cracks are not associated with earth or water retaining concrete components

For inactive cracks with minimum 0.3 mm width and up to 1mm, resin injection is the most common repair method (Figure 7.9). This treatment should effectively restore the structural adequacy of affected members by strong bonding of elements.



Figure 7.9: Injected cracks with epoxy resin through ports

It may be necessary to seal the reverse face of the cracked structure to prevent the resin flowing out. Although thixotropic resins can be used, they cease to flow when the injection pressure is released (i.e. when resin reaches the reverse side of the crack). Crack injection is particularly useful in cases such as retaining walls or ground slabs where sealing of cracks on the reverse side is impractical.

Injection pressure should be maintained so that the resin penetrates the crack. The most effective epoxy injection is carried out with pressures at the injection ports ranging from near zero (i.e. 0 to 300 kPa) to about 1000 kPa and certainly not exceeding 2068 kPa (300 psi).

The appropriate injection pressure depends on the equipment used (i.e. proprietary injection kits, grease guns) and the success of the outcome is dependent on the experience and knowledge of the operator. Excessive pressure is likely to force the resin along the path of least resistance, thus leaving voids and it can result in additional crack formation, particularly in the case of transverse cracks. The main feature of the method is that the ultra-low viscosity epoxy resin is injected under a relatively low constant pressure (i.e. in the order of 300 kPa) over several hours.

In the case of wide cracks, specially formulated injection materials with no shrinkage properties are commercially available which ensure that no loss of adhesion occurs from the bridging of wide cracks.

Other specially formulated epoxies such as filled liquid epoxy systems or epoxy paste are also available which can be applied manually to effectively seal wide cracks in both horizontal and vertical surfaces. Such materials are particularly suitable for much broader cracks or joints as well.

Cracks on horizontal surfaces can be repaired by simpler methods as follows:

- A dam is formed on each side of the crack with a mastic sealant

- Premixed liquid epoxy or sealant is poured into the dam and allowed to penetrate the cracks by gravity. The dam increases the size of the reservoir so that topping up is less frequent.
- Finally, surface appearance should be restored.

When the volume of repair material is significant, it may be economical to open out the fracture and repair the area with cementations mortar followed by appropriate curing to prevent premature drying out and subsequent cracking.

7.2.5.1 Repair Method

1. Complete surface preparation in accordance with [RPM01] Surface Preparation for Crack Repair.
2. Holes are drilled at 150 to 200 mm centres for insertion of injection ports.
3. Injection ports are placed over cracks and fixed in position using a suitable surface sealant and locating pins to ensure correct positioning. If the structure is cracked full thickness injection ports are fitted to both sides of the structure, with those at the back positioned midway between those in the front.
4. Seal the surface of all cracks between injection ports using a suitable surface sealant. Seal both sides of any cracks all over the structure if possible (i.e. back face of concrete against fill). Avoid blocking the port holes with the surface sealant.
5. Allow the surface sealant to harden.
6. Mix the low viscosity epoxy resin in accordance with manufacturer's instructions and commence injection from lowest port, particularly treating vertical cracks.
7. Inject epoxy resin into the lowest port until it begins to flow out of the adjacent port.
8. Detach the connecting tube and close off the first injection port.
9. Inject epoxy resin through the adjacent second port until it flows out of the third.
10. Follow the same procedure until all injection ports are injected. Allow the injected system to harden.
11. Remove injection ports and fill any holes or voids with surface sealant and allow hardening.
12. After hardening, remove surface sealant by grinding to restore the appearance of the surface (apply surface treatment if it is part of the repair process or if warranted).
13. Perform quality assurance for resin injection procedure including verification of epoxy penetration depth, component ratio and injection pressure.



(a)



(b)



(c)

Figure 7.10: Surface preparation for crack repair

7.2.6 Repair of Active Cracks

Cracks for which the width fluctuates with changing loads or temperatures are called active or live cracks and should be treated in the same way as movement joints. Such cracks should not be filled with rigid materials, as the treatment is unlikely to be successful and may generate new defects either in the repair material or elsewhere in the surrounding concrete substrate.

The advice of a structural engineer should be sought in deciding the appropriate treatment of active cracks. When deciding repair material and procedure for active cracks, it is important to appreciate the cause of the cracking.

To prepare for the repair a recess is cut along the line of the crack using a power hammer, fitted with a sharp chisel or crack cutter. Concrete that is carbonated or contaminated with other detrimental substances must be broken out and replaced with suitable cementitious or epoxy mortar, and the crack prepared as a flexible sealed movement joint.

Once the crack is prepared it is filled with a suitable joint sealant that will bond to the surfaces each side of the recess, but debonded from the bottom of the recess by using a bond breaking medium. Debonding the bottom of the recess prevents the crack propagating into the sealant and the subsequent failure of the sealant as a result of continuing crack width fluctuation.

The appropriate width and depth of the recess is dependent on both the total amount of crack movement (i.e. joint substrate characteristics) and the cyclic movement capacity of the joint sealant material. Expansion and contraction due to temperature changes in the concrete substrate is a common cause of crack width fluctuation. The capacity of the sealant to accommodate movement is dependent on its ultimate elongation capacity and its ability to recover when subjected to tensile or compressive strain.

The joint repair should be designed so that the movement due to factors such as concrete shrinkage and thermal changes does not exceed the cyclic movement capacity of the sealant. The joint width is a function of the magnitude of the movement.



Figure 7.11: Active or live cracks sealed with an appropriate flexible sealant

7.2.6.1 Repair Method

1. Complete surface preparation in accordance with Surface Preparation for Crack Repair.
2. Widen the crack to the required depth and width.
3. Insert a bond breaking medium into the base of the widened crack to serve as backing for the sealant (eg a proprietary backing rod).
4. Seal the crack with an appropriate flexible sealant.

7.2.7 Plastic Shrinkage and Plastic Settlement Cracks

Plastic shrinkage cracks are caused by rapid drying of the concrete during construction as a result of moisture loss from the surface prior to hardening of the concrete. Plastic settlement cracks typically form above reinforcing bars while the concrete is still plastic and has not hardened. These cracks can be treated as inactive cracks.

Plastic settlement cracks are a common cause of reinforcement corrosion. They should be sealed as soon as possible and before contaminants enter the cracks. Fine cracks can be treated by a suitable concrete coating.

7.2.7.1 Repair Method

1. Brush off the cracked surface with cement grout or non-shrink polymer modified grout to improve adhesion between the crack surfaces and reduce shrinkage of the grout.
2. Depending on the situation and the subsequent construction considerations, complete surface sealing using elastomeric protective coatings and waterproof membranes as required.

3. Prior to undertaking the coating works the surface should be high pressure water cleaned and allowed to dry before application of protective coating.
4. If cracks are greater than 0.2mm wide a surface coating or sealing will not be suitable and repairs should be implemented in accordance with [RPM 02] Repair of Inactive Cracks.

7.2.8 Shear and Transverse Cracks

Shear and transverse cracks can be either active or inactive and are caused by physical, thermal or chemically applied stresses. These cracks should be repaired to prevent further deterioration.

The advice of a structural; engineer should be sought in deciding the appropriate treatment. The cause of the shear crack needs to be determined and treated prior to the repair. If the shear cracks are caused by normal traffic loading, repairing the cracks may simply result in formation of new cracks.

However, if the crack is due to the passage of a single heavy load or some other single event, then the member can be repaired as outlined below. Refer to Standard Section.

7.2.8.1 Repair Method

1. The inactive crack can be repaired by epoxy injection in accordance with [RPM 02] Repair of Inactive Cracks.
2. The injection port spacing should be reduced to ensure full integrity has been restored to component. Once the crack has been repaired, it would be prudent to monitor the repair for a period to ensure the cause of the crack had been correctly determined and that the cracking does not recur. If cracking does recur, the member should be strengthened.

7.2.9 Cracks Caused by Rusting

Cracks which form directly over steel reinforcement are commonly caused by corrosion of the metal. These cracks result from the bursting forces exerted by the build-up of corrosion product and can lead to spalling and complete loss of concrete cover.

When concrete damage occurs as a result of corrosion of the reinforcement, the reinforcement must be treated to prevent rusting and the surrounding concrete replaced. If the steel reinforcement is showing only moderate corrosion, the cracks should be repaired in accordance with Reinforcement Corrosion Repair.

7.2.10 Quality Control of Epoxy Injection of Cracks

To ensure that the injection has been successful, quality assurance measures may include test cores or Non-Destructive Evaluation (NDE).

1. Test cores:
 - Core locations should be chosen to avoid cutting reinforcing steel, drilling cores in areas of high stress, or creating core holes below the waterline. The engineer should determine core locations when these types of conditions exist.
 - Be sure the epoxy has set before extracting a core.
 - Take cores (normally 2-inch diameter) to check that the penetration of the epoxy is

adequate.

- Inspect the core visually to determine the penetration of the epoxy into the crack; Cores can be further tested for compressive and split tensile strength per ASTM C 42.
- Subsequently, patch the removed-core area (after proper surface preparation) with an expansive cementitious or epoxy grout compatible with the existing substrate concrete and the surrounding environment.

2. Methods for nondestructive evaluation:

- Impact Echo (IE)
- Ultrasonic Pulse Velocity (UPV)

7.2.11 Underwater Epoxy Injection of Cracks

Pressure injection of cracks underwater can be used for underwater applications also. When cracks expose the reinforcing steel to moisture, the stage is set for the corrosion process to begin. In salt water environments corrosion can occur very fast. With the proper selection of a water-compatible adhesive, normally an epoxy resin, dormant (non-moving) cracks saturated with water can be repaired. The procedure can also repair other small voids such as delaminations or honeycombed areas near the surface of the concrete. Pressure injection can be used, within limits, against a hydraulic head provided the injection pressure is adjusted upward to counteract the pressure of the hydraulic head. The material must displace the water as it is injected into the crack to ensure that the crack is properly sealed resulting in a watertight monolithic structural bond.

Epoxies must have certain characteristics to cure and bond the cracked concrete together. Many adverse elements are present inside the concrete crack such as water, contaminants carried by water, dissolved mineral salts, and debris from the rusting reinforcing steel. The typical low surface temperature of concrete underwater eliminates many products due to their inability to properly cure. The epoxy injection resins for cracks are formulated in low viscosity, and they do not shrink appreciably. The surface wettability of epoxy resin is of major importance, because the resin should displace all water in the crack and adhere to a wet surface and then cure in that environment.

The procedure involves cleaning of the crack by a high-pressure water jet system and shaping the surface of the concrete directly above the crack so that it can be sealed with a grout. Injection ports are installed in holes drilled to intersect the crack by a hydraulic or pneumatic drill. Subsequently, the surface of the crack is sealed with a grout material suitable for underwater use such as the cementitious or epoxy mortar described earlier. The purpose of the grout is to retain the adhesive as it is pumped into the crack.

The adhesive is pressure-injected into the crack through the ports that are embedded in the grout at regular intervals. The injection sequence begins at the bottom and advances upward. The injection moves up when the adhesive reaches and begins to flow from that port. Epoxy resin is mixed either before or after pumping. Cracks varying in width from 0.002 inch to 0.25 inch may be injected successfully. Epoxy pressure injection has gained widespread acceptance as a cost-effective method to bond together and seal cracked structural concrete members.

The following precautions should be noted:

- Contaminants 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.

- Time and patience is required for the successful injection project.
- Injection is labor-intensive. (As the temperature drops below 50°F, it becomes more difficult to pump the epoxies into fine cracks).

Experience on the part of the diver in injection and the formulation of the epoxy for injection are very important.

7.3 Other Concrete Repairs

7.3.1 Alkali Aggregate Reaction (AAR) in Concrete

A reaction can occur between the reactive form of silica or silicate in the aggregate and the alkalis in the cement paste to produce a gel called ettringite which is highly expansive in the presence of moisture and exerts tensile stress. The result is map cracking or directional cracking (pre-stressed members) in the structure. Other visible signs of damage may include an aggregate pop out and discoloration.

It can take many years before AAR reaction gives rise to cracking and the cracking will decrease with time as the quantity of silica is depleted.



Figure 7.12: Cracks due to AAR

7.3.1.1 Repair Method

1. Identify and remove the source of water causing the chemical reaction (i.e. fix expansion joint, redirecting scuppers and etc.). This will slow the reaction and cracking.
2. Testing of concrete cores will show the presence of the gel and advice should be sought from structural engineer in deciding the extent of further possible cracking.

7.3.2 Damage to Concrete Protective Coating

In some cases, the concrete surface may appear sound but may be allowing excessive chloride or moisture penetration into the steel reinforcing. The surface needs to be coated to prevent the

penetration of these substances, but the coating should be capable of allowing the concrete to dry out by passing water vapour through the outer pores.

7.3.2.1 Repair Method

1. Thoroughly clean the outer surface of the concrete using a high-pressure water blast.
2. Inspect the concrete surface to identify any crack wider than 0.2mm or concrete spalls requiring crack repair.
3. Complete surface preparation in accordance with [RPM01] Surface Preparation for Crack Repair.
4. Complete spall and scale repairs in accordance with [RPM09] Spalling and Scaling Repairs.
5. The surface can then be sprayed with two coats of silane, silane/ siloxane or moisture cured polyurethane to block the pores in the concrete to a depth of 2mm to 3mm. These products allow vapour passage but not droplet penetration of the coating.
6. Recoating may be required in approximately 15 years as the coating begins to fail. Cementitious fairing coats can be effective in slowing down the ingress of chlorides but are not cost effective for large areas where the thin sprayed coatings can be easily applied.

7.3.3 Spalling and Scaling Repairs

Spalling may commence with local weakening of concrete and lead to more extensive deterioration of the concrete structure. It is a detachment of a fragment from concrete, usually in the shape of a flake. A spall may be due to the action of the weather, pressure by overstressing, expansion of the surface concrete as a result of corrosion of steel reinforcement or impact damage (Figure 8.13). Cracks associated with the spalls are usually wide, long, and deep enough to reach the steel reinforcement or prestressing steel.



Figure 7.13: Spalling Concrete

Corrosion of the underlying reinforcing steel or deteriorated aggregate is the most common cause. If the spall is about 25 mm or less in depth or about 15 cm in diameter or less, it is considered to be a small spall.

Large spalls may significantly affect structural capacity. A hollow sound when the area is struck with a hammer or steel bar or when it is swept with a drag chain can indicate unsound or defective concrete. When spalling or delamination is evident, the entire concrete component should be surveyed to determine the extent of spalling and delamination before beginning repairs.

A delamination survey might involve rod sounding, hammer sounding, drag chain sounding, or ultrasonic delamination detecting. The reinforcing cover survey uses a magnetic field detector to estimate the depth of concrete over the reinforcing steel. The chloride content survey involves analyzing the samples of concrete powder produced from drilling holes in the concrete.

The potential corrosion survey consists of electrical resistivity measurements using a halfcell probe. This will not be an effective procedure when the steel reinforcement contains epoxy-coated reinforcing steel or contains galvanized coated steel or if the deck surface was treated with a non-conducting material.

Once the extent and severity of the spalled area has been determined the appropriate corrective action can be determined. If a concrete bridge deck is scheduled for overlay, a low slump dense concrete should be considered to improve resistance to moisture penetration.

Light scaling of concrete does not expose coarse aggregate, and the maximum depth is about 6mm. Medium scaling involves loss of surface concrete to a depth of 5mm to 10mm. Severe scaling involves loss of surface concrete to a depth of 5mm to 10mm and exposure of coarse aggregate (10mm to 20mm). Very severe scaling involves loss of coarse aggregate as well as surface concrete generally to a depth greater than 20 mm.

For any condition exceeding a light scaling, concrete repair procedures similar to those applicable to concrete patch should be applied. When removing the deteriorated concrete, care should be exercised to avoid damage to the reinforcing steel. If the scaling results from chlorides penetrating the deck wearing surface, the deck should be monitored for further deterioration.

However, if the scaling is due to poor concrete, scaling may progress downward resulting in deeper deterioration of the concrete which requires more extensive repairs.

7.3.3.1 Repair Method

1. Undertake delamination survey and mark out area to be repaired having regard for the repair methods available and skill of available personnel. The repairable concrete area should be twice the length and width of the original spall surface area.
2. Break out marked out areas with hand held percussion equipment such as hammer drill. Check the prepared area by sounding with a hammer to confirm all defective concrete has been removed. If steel reinforcement is exposed, the concrete should be broken out to a depth of 20mm behind the steel reinforcement. With the reinforcement fully exposed the repair material can lock around the bar for support and strength. The fine nature of the ingredients ensures very small pore sizes and hence improved protection against moisture and chloride ingress.

3. Clean existing reinforcement if there is light corrosion or replace steel if it has lost more than 25 percent of its cross-sectional area in accordance with [RPM12] Reinforcement Corrosion Repair.
4. Sawcut to 15mm depth around the perimeter of the patch areas to avoid having feathered edges which could easily break off and provide a path for the ingress of moisture.
5. The prepared concrete surfaces and any exposed steel reinforcement should be cleaned by wash down or by blowing down with oil free compressed air to ensure removal of all residual contamination. The concrete surfaces should be thoroughly pre-wetted with clean fresh water and the surface dried prior to application of repair material.
6. The repair material should have similar properties to and be well bonded to the base material. It should not shrink away from the base surface and should offer better protection than the base material against ingress of aggressive agents where the cover depth is less than desirable. For these reasons, a cementitious polymer modified repair mortar or concrete that has shrinkage-compensating agents is preferred.
7. The manufacturer's mixing recommendations should be followed. The repair mortar may appear to be too dry, however the impulse to add extra water should be resisted. These high build mortars may be applied by trowel, or by hand to ensure proper compaction around the steel and good adhesion to the base concrete.
8. The repair must be built up in layers in accordance with the manufacturer's recommendations and in certain conditions layers such as overhead situations where layers should be less than 25 mm to reduce the risk of the repair mortar detaching from the base concrete.
9. The completed repair should be cured similar to other concrete work. Moisture cure is the preferred method of curing, but the application of a curing membrane may be the more practicable for small areas. Curing should be applied for at least the first three days, which are the most critical to preventing premature cracking of the repair due to shrinkage.
10. Minimum cover of 50mm of concrete to the steel reinforcement should be provided unless the location is in a benign and moderate exposure classification for the environment in which case 20mm may be acceptable.
11. Where existing concrete does not have the minimum cover thickness the patch can be built out locally alternatively steel can be coated with a zinc rich epoxy primer (min 80% zinc).

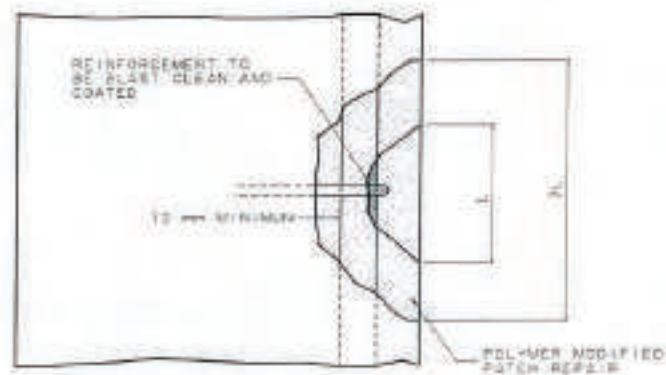


Figure 7.14: Basic concrete patch repair with reinforcement corrosion

7.3.4 Repair of Large Spalled Areas

For large areas of spalling, a suitable and economical repair method using sprayed concrete can be considered. The spalled areas can be in walls or in areas overhead such as the underside of a cast in-situ deck.

Sprayed concrete or shotcrete is used when a large area of concrete requires repair because of spalling or heavy water washes of the concrete surface fines. There are two methods of shotcreting:

- the wet method uses premixed mortar and pumps and sprays on the defect
- the dry method where the dry mixture is conveyed to the nozzle by compressed air and mixed with water at the nozzle.

The spray dry method is more reliant on the skill of the operator as the operator controls the amount of mixing water but it is quicker and more economical than the wet spray method. The high impact velocity of the spray results in compaction of the repair mortar.

If the intention is to restore the surface profile and seal the concrete surface, the existing surface should be given a high-pressure water clean prior to the shotcreting.

The mortars used are polymer modified cementitious products to ensure the bond is achieved with the base concrete and that the new concrete does not shrink away when hardened. The mortars also provide protection to the steel reinforcement that may otherwise have inadequate concrete cover.

7.3.4.1 Repair Method

1. The areas to be repaired must be prepared with deteriorated concrete removed, corroded steel fully exposed, cleaned and protected in accordance with a basic concrete patch repair in accordance with [RPM09] Spalling and Scaling Repairs.
2. The repair mortar is sprayed onto the existing concrete under pressure and is compacted on impact with the concrete surface causing some material loss due to rebounding. The repair must be built up in thin layers by the spray, and particular attention should be paid to ensuring the proper build-up of material behind the reinforcement and elimination of voids.
3. Any smoothing or working of the surface is difficult and must be done immediately before initial set of the sprayed concrete.
4. The concrete surface will require curing for a minimum of 3 days to prevent shrinkage cracking.

Extensive breaking back of concrete or replacement of reinforcement may mean that the concrete element, such as a column, is not serviceable during the repair in which case the component may need to be propped during the repair works.

7.3.5 Surface Erosion or Abrasion

Erosion is caused by abrasion or cavitation action. Abrasion action is caused by flowing liquids containing abrasive materials (i.e. debris in water). Cavitation action is caused by fast flowing water containing tiny vapor bubbles, rather than particles of solid matter. The bubbles collapse with impact and sudden changes in direction, causing high localized pressures sufficient to abrade the concrete surface and ultimately corrode the steel.

Concrete structures or components can be subjected to impact forces at their surface in the course of normal use, resulting in abrasion.

Abrasion damage is caused by movement of solid objects or particles over the concrete surface, causing wear of the surface by rolling, rubbing and friction. The damage results in a rough friable surface with grooving, potholing or spall particularly at the edges.

The combination of an abraded surface and the permeable nature of the remaining concrete increases the risk of corrosion of the reinforcement and the abraded surface should be replaced and sealed.

7.3.5.1 Repair Method

1. The existing concrete surface should be high-pressure water blasted to clean it and remove any loose concrete, providing a hard, sound surface for the repair.
2. A thin polymer modified mortar fairing coat should be applied by trowel to provide a hard, impermeable surface that will resist further abrasion. The new surface improves the performance of the cover concrete in preventing moisture penetration to the steel, thereby prolonging its service life.
3. The repair mortar requires curing to prevent shrinkage cracking especially in the first three days when moist curing is essential.
4. If large walls have been water washed and require sealing then, it may be more economical to spray the fairing coat mortar than to employ hand trowelling.

7.3.6 Reinforcement Corrosion Repair

Corrosion of steel reinforcement is one of the most common causes of deterioration of reinforced concrete (Figure 7.15). Steel reinforcement in concrete can corrode in the presence of moisture and oxygen. Chloride intrusion accelerates this attack by de-passivation of the steel and lowering of the electrical resistivity of the concrete.



Figure 7.15: Reinforcement corrosion of concrete

The acid forming gases influence corrosion of steel reinforcement in two ways. Firstly, by reducing the pH level of the concrete (neutralisation), thereby reducing the alkalinity of the area adjacent to the reinforcement. Secondly, by acting as an acid when combined with water, once the pH is reduced by the neutralisation process (known as the carbonation in the case of carbon dioxide). In

addition, the alkalinity of the environment surrounding the reinforcement and hence its passivating effect may be destroyed by water leaching the alkaline substances out of the concrete (concrete degradation).

The potential for diffusion of chloride ions into concrete and a resultant reduction in alkalinity is dependent on the quality and physical properties of concrete, particularly its permeability. Poor quality concrete can be quite permeable.



Figure 7.16: Severe corrosion of reinforcement

7.3.6.1 Repair Method

1. Remove the deteriorated concrete back to sound and dense concrete to provide a minimum clearance of 20 mm behind and around the rusted steel reinforcement. This clearance also provides a mechanical key for the concrete patch material.
2. Concrete should be removed along the length of visibly corroded reinforcement until at least 50mm of sound, rust free metal is exposed at each end.
3. The steel reinforcement commonly has corrosion byproducts around its whole diameter. Therefore, the bar is required to be fully exposed over the corroded length and at least double the length of the spall.
4. The reinforcement should be thoroughly cleaned back to bright metal and protected with a proprietary coating of zinc-rich paint.
5. With the reinforcement fully exposed and a depth of the bars of 12mm minimum, the repair can lock around the bar for support and strength.
6. Replace any reinforcement which has lost more than 25 percent of its cross-sectional area by cutting out the corroded section and lapping new reinforcement in its place. The advice of a structural engineer should be sought in determining the requirements for lapping of the reinforcement.
7. Apply an anti-corrosive coating to prevent further rusting
8. Renew the concrete cover using suitable materials, either a resin or cementitious mortar, depending on the quantity of material required to restore the original profile.
9. Complete the concrete patch repairs in accordance with the procedure for Spalling and Scaling [RPM 10].

7.3.7 Repair of Shear Key Failure in U-Slab

Reinforced concrete U-slab bridges were originally constructed between 1962 and 1976. These bridges have small sections or keys of 45 MPa concrete between their tops to transfer live load from one slab to another by shear action with the aid of connection bolts between legs of the U-slabs.

When bolts become loose as a result of traffic vibration effects, the slabs tend to move relative to one another resulting in damage to the shear key.

Initially, the bolts may be only slightly loose, and a reflective crack may appear in the sprayed seal above the shear key. As shear key continues to deteriorate, the asphalt above the shear keys and the shear key itself will disintegrate.

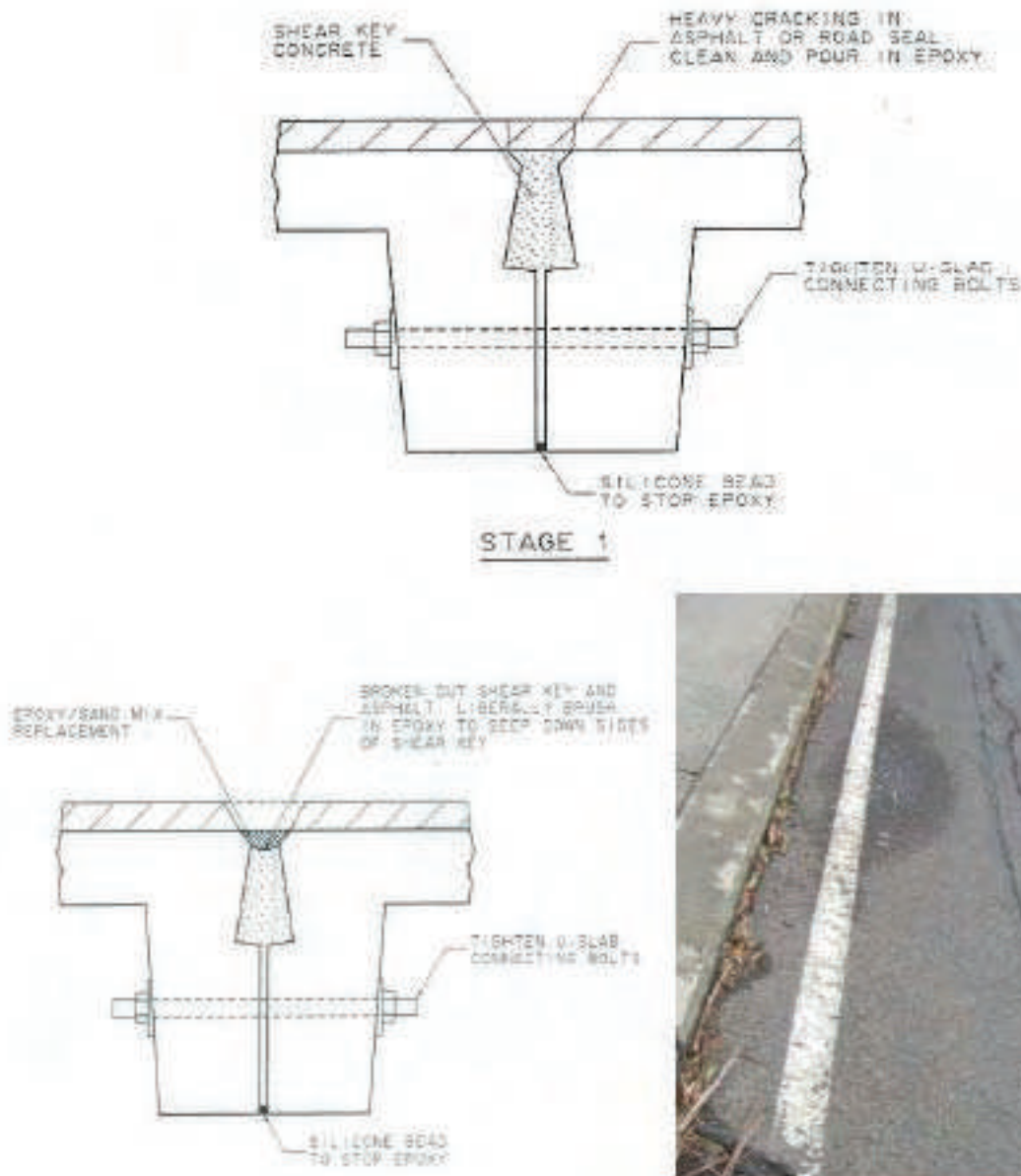


Figure 7.17: Shear key repair

Figure 7.18: Shear key failure of U-slab



Figure 7.19: Shear key failure of U-Slab bridge

7.3.7.1 Repair Method

1. Retighten the bolts to finger tight plus half a turn of the nut. It is important not to over tension the bolts, to avoid springing of the legs. Furthermore, excessive tightening may cause the unit to crack at either the junction between the leg and the slab or along the centre of the top slab.
2. Clean out asphalt cracks with compressed air and seal the underside of the U-Slab with a bead of silicone in preparation for the epoxy repair of the shear key.
3. Pour a viscous epoxy along the road surface crack allowing the epoxy to seep into the key shear crack to re-establish the bond between the two surfaces. Any residual cracks in the roadway are sealed with rubberised bitumen or polymer modified bitumen. Provided the deterioration is limited, this procedure should serve as a sufficient and effective repair method.
4. However, if the shear key concrete is beginning to break up and crumble then, it should be replaced, and an epoxy mortar mix installed in its place after steps 1 & 2.
5. Remove asphalt above the shear key, epoxy brush shear keys and allow epoxy to seep down the sides.
6. An epoxy/sand mix of 45 MPa strength is placed into the shear keys to top up the missing concrete.
7. Reinstall asphalt after epoxy has hardened.
8. If cracks appear in the U-slab and moderate to heavy rust staining is observed or cracks are wider than 0.2mm and extend to the top of the slab, then u-slabs should also be overlaid with a composite reinforced concrete deck to strengthen the slabs and restore the composite action under live loads. The concrete overlay is undertaken as part of the repair maintenance. The advice of a structural engineer should be sought with the design of the new deck overlay.

NB: Light spring washers can be used to extend the time between bolt tightening maintenance is required.

CHAPTER VIII: DECK SLAB, FOOTPATH AND RAILING MAINTENANCE AND REPAIR

8.1 Bridge Decks and Slabs Maintenance Introduction

The deck or slab of a bridge is like the roof on a house; it receives the brunt of the storm and shelters the contents below from the harsh elements. The deck is vulnerable to the effects of mechanical wear from traffic induced deflections, tire wear, and from environmental conditions such as rain, snow and ice. Consequently, decks and slabs typically require more maintenance and repair than any other bridge component. The following sections in this chapter will present common maintenance and preservation techniques for concrete, steel, timber and fiber reinforced polymer decks.

The most common bridge deck and slab material is concrete. The physical properties of concrete permit placement in various shapes and sizes, providing the bridge designer and the bridge builder with a variety of construction methods. Concrete bridge decks resist wear and weather well, provide good traction and ride quality, and in many designs provide a part of the structural strength of the bridge. Careful design, proper selection of materials, and good construction practices are essential for durable bridge decks and slabs.

Among the design features that improve durability are adequate cover over the reinforcement, good drainage, smaller bridge skews, and thicker slabs. Material and construction practices that contribute to durability include achievement of the specified cover, use of air-entrained concrete in areas subject to freezing, use of concrete with appropriate water-cement ratios, good consolidation, and proper curing procedures that prevent excessive shrinkage cracking. The main cause of concrete deck and slab deterioration is corrosion of the reinforcing steel within the deck. Corrosion is typically caused by small cracks in the concrete that provide pathways for water and chlorides to penetrate the concrete and attack the steel. Once the steel begins to corrode, the steel corrosion by products expands and cause deck delaminations or spalling. To slow the corrosion cycle, the water and chlorides must be kept from reaching the reinforcing steel.

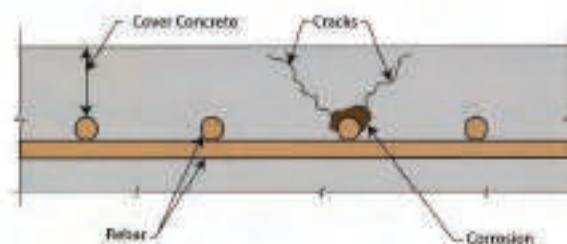


Figure 8.1: Typical Reinforced Concrete Deck and Slab Deterioration

- Cracks
- Spalls
- Leaks on underside of deck
- Efflorescence in edges or underside deck

- Accumulated debris on deck
- Evidence of ponding on deck

8.2 Crack Sealing

Cracks in concrete bridge decks and slabs can be sealed using commercially available penetrating sealers appropriate for widespread cracking, or through pressure injection for larger discrete cracking, as shown in (Figure 8.2) Sealing deck and slab cracks will reduce the water and chlorides that are able to reach the reinforcing layer and cause corrosion. Depending on the nature of the cracking observed, a number of methods are available to waterproof, seal or repair the cracks.

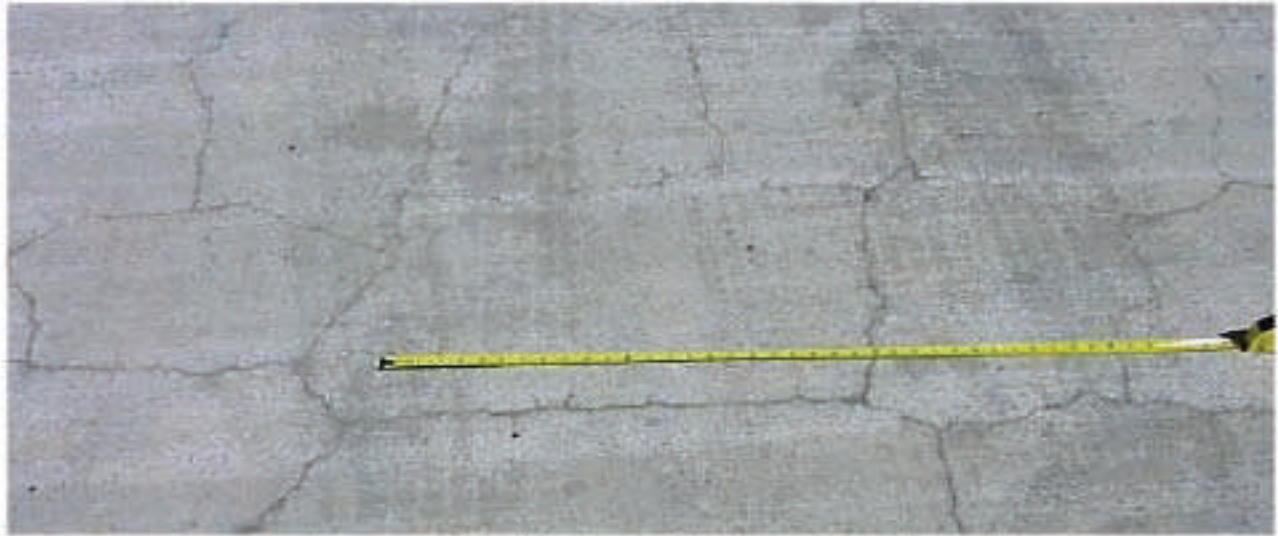


Figure 8.2: Typical Widespread Cracking Pattern on a Bridge Deck



Figure 8.3: Typical Discrete Cracking Pattern on a Deck

8.2.1 Penetrating Sealers

Bridge decks and slabs that have cracks over a large portion of the surface area are good candidates for penetrating sealers. Penetrating sealers are applied to bridge decks and slabs by spraying or pouring on the deck and spreading over the entire area using brooms or squeegees. Penetrating sealers are typically low viscosity which allows them to be drawn into concrete cracks through

capillary action. Viscosity is measured by the force per unit area resisting a flow and is measured in the units of centipoise. For comparison, several viscosities are presented:

Water ("low" viscosity): Around 1 centipoise

Methacrylates: Around 10 centipoises

Epoxy based healer sealers: Around 80 centipoises

Epoxy based thin epoxy overlays: Typically, 1500 centipoise (similar to honey or heavy weight motor oil)

The sealers fill the concrete cracks and voids. On curing, the sealers forms bonds to the walls of the cracks that effectively seal the cracks. The crack sealers significantly reduce the permeability of the concrete. The permeability reduction minimizes the ingress of water and chlorides from road salts or deicing chemicals. There are many commercially available penetrating cracks sealers that are designed for specific applications. The selection of the appropriate crack sealer depends on a number of variables including: crack width, available cure times and desired flexibility of the cured product.

8.2.2 Crack Injection and Epoxy Sealing

Concrete deck and slab cracks that are discrete or localized can be sealed or injected with epoxy resins or other polymers to fill the void and re-bond the concrete surfaces back together. Epoxy injection can restore structural integrity and reduce moisture penetration through concrete cracks 0.002 inches in width and greater. (ACI RAP Bulletin 1: *Structural Crack Repair by Epoxy Injection*).

Full-depth deck cracks must be capped from both the top and below before they are pressure injected from top. Caps have been successfully installed using epoxies, polyesters, paraffin wax, and silicone caulk. The selection of the cap material should consider the following criteria, subject to the type of crack to be repaired: Non-sag consistency (for overhead); Moisture- tolerance; Working life; and Rigidity (modulus of elasticity). (ACI RAP Bulletin 1: *Structural Crack Repair by Epoxy Injection*).

For cracks that are opening and closing under traffic loads, a flexible elastomeric crack sealer may be warranted. The following conditions may indicate a working crack:

- The crack is open and shows evidence of movement
- Has been sealed before and has failed again
- Concrete dust fills the crack

The injected crack sealer material should be designed for the anticipated movement of the deck or slab crack. A summary of the options for bridge deck crack sealing are presented in Table 8.1:

Table 8.1: Crack Sealer Summary

Attribute	Silane/Siloxane	Heavy Weight Methyl Methacrylate Polyurethanes	Epoxy Healer Sealer	Thin Epoxy Overlay	Injection Epoxy
Typical Applications*	Waterproofing good condition concrete	Crack sealers for widespread fine cracking	Crack sealers for widespread fine cracking	Crack sealer for widespread discrete larger crack widths	Crack sealer for widespread discrete larger crack widths
Viscosity (Centipoise)	<1	20 - 200	50 - 150	1500 - 3000	500 - 2000
Pot Life	NA	5 - 45	20 - 60	15 - 30	15 - 30

Attribute	Silane/Siloxane	Heavy Weight Methyl Methacrylate Polyurethanes	Epoxy Healer Sealer	Thin Epoxy Overlay	Injection Epoxy
(Minutes)					
Minimum Cure Time (Hours)	1 - 4	2 - 12	3	1	2
Skid Resistance	Original Surface	Broadcast fine aggregate (sand)	Broadcast fine aggregate (sand)	Broadcast fine aggregate (sand)	Broadcast fine aggregate (sand)
Expected Useful life (Years)	3 - 5	5 - 10	5 - 10	10 - 15	5 - 10

*Sealers should be formulated specifically for the crack(s) being addressed.

8.3 Deck and Slab Overlays

Bridge deck and slab overlays provide a wearing surface that can protect the underlying deck and provide a maintainable layer on the riding surface. Wearing surfaces add protection for the underlying deck or slab but also add dead load weight to the bridge and thus reduce the live load capacity. Adding a new wearing surface or increasing the thickness of an existing wearing surface should be evaluated by an engineer for impact on the live load capacity. Wearing surfaces can generally be categorized into three general classes; asphalt, concrete, and polymeroverlays.

8.3.1 Asphalt Concrete Overlays with Waterproofing Membrane

Asphalt concrete overlays are very common because they are easy to install, relatively inexpensive and most agencies have established mix specifications and a long history of using the product. Asphalt overlays can vary in their performance depending on the density, aggregate and binders used. Asphalt does tend to trap and hold moisture and chlorides that can lead to deterioration of the underlying deck if not properly protected. It is a good practice to include a waterproofing membrane under an asphalt overlay unless the asphalt is a temporary measure to improve ride quality on bridges or bridge decks planned for rehabilitation or replacement.

Water proofing membranes provide a moisture barrier between the asphalt and the underlying deck material. Membranes can be categorized as constructed-in-place or preformed membranetypes.

Constructed-in-place membranes include:

- Bituminous
- Spray applied resins

Preformed membranes include:

- Asphalt-impregnated fabrics
- Polymer
- Elastomeric
- Asphalt-laminated board systems

The most commonly used water proofing membranes are bituminous and asphalt-impregnated fabrics. Both common membrane systems consist of a tack coat on the prepared concrete deck, a spray applied or preformed membrane, a protective board, tack coat and then the asphalt applied on top. A schematic of these layers is shown in (Figure 8.4).

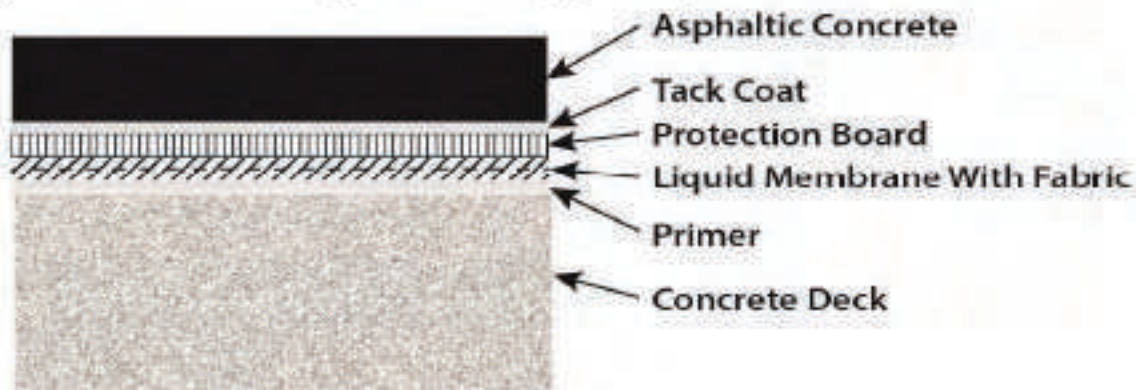


Figure 8.4: Typical Membrane Configuration

8.3.2 Concrete Overlays

Rigid concrete overlays are very common on bridge decks and slabs because they have a relatively long life, have quick installation techniques known by most contractors and provide good ride quality when cured. Concrete overlay types include the following concrete variants:

- High performance concrete
- Latex modified concrete
- Silica fume concrete*
- Fly ash modified concrete
- Polymer concretes

*Note that several states have reported bonding issues with silica fume concrete, particularly with patching operations.

Concrete overlays provide a durable wearing surface that stiffens the deck or slab, provides additional cover over reinforcing steel, and reduces permeability of the deck or slab. All of these attributes can slow or eliminate the ingress of water and chlorides that lead to corrosion. Concrete overlays also provide a single step means to patch localized deck spalls and overlay in a monolithic placement of concrete that can lead to improved ride quality.

8.3.3 Polymer Overlays

Thin bonded polymer overlays are very common overlays for bridge decks and slabs because they are light weight, result in minimal grade differences, reduce permeability and the bridge can be put back into service quickly. Polymer overlays are typically less than one half inch thick and can be applied in one or more layers. Polymer overlays use resins of epoxy, epoxy urethane, or methyl methacrylate. A comparison between the overlay options available is presented in Table 8.2:

Table 8.2: Bridge Deck Overlay Summary*

Attributes	Asphalts	Concretes	Polymers
Typical Applications	Thick flexible wearing surface over all common deck materials. With membrane	Durable thick rigid wearing surface and deck protection	Thinner more flexible deck protection and wearing surface
Typical Thickness Range (inches)	1.5 - 6	1 - 5	0.5 - 1.5
Surface Prep	Clean and milled or roughened	Steel shot, ground or roughened	Steel shot, ground or roughened
Minimum Cure Time for Traffic (hours)	1 - 4	2 - 12	2 - 16
Skid Resistance	Original Surface	Concrete Finish, grind or grooving	Broadcast fine aggregate (sand)
Expected Useful Life (years)	15 - 20	15 - 30	10 - 20

* For more information refer to: International Concrete Repair Institute, Technical Guideline No. 310.2R-2013, Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, Polymer Overlays, and Concrete Repair.

8.4 Epoxy Crack Injection

Discrete larger deck and slab cracks can be sealed and re-bonded through surface or pressure injection with a material such as epoxy or polyurethane. Following is a procedure for crack injection:

1. Clean out the cracks to be sealed. Methods to clean cracks include:
 - a. Vacuuming
 - b. Oil-free compressed air
 - c. High pressure water
2. Seal the crack surface. Apply cap sealer epoxy along the dry top surface of the crack and allow it to harden. If the crack has spalled edges or if injection pressures are too high for a surface seal, saw cut the crack first and then, seal the surface by cutting a v-groove and filling it with mortar.
3. Use surface ports or drill injection port holes along the crack.
4. Inject the sealant through the injection ports until the cracks are filled.
5. Allow to cure before returning traffic.



Figure 8.5: Epoxy Injection Repair

For surface sealing with epoxy, the following procedure applies

1. Clean out the cracks to be sealed. Methods to clean cracks include:
 - a. Vacuuming
 - b. Oil-free compressed air
 - c. Abrasive blasting (i.e, shot blasting or sandblasting).
2. Apply epoxy onto the clean dry crack on deck surface.
3. Broom epoxy along the dry top surface of the crack the width of the fiber reinforcement.
4. Roll out the fiber reinforcement fabric onto the wet epoxy surface and press into the epoxy until the fabric is saturated (see figure 8.5). Overlay as needed for ride quality.
5. Allow to cure before returning traffic.

Examples of surface epoxy sealing with optional fabric are shown in Figure 9.6. Rolling out the fiber reinforcement fabric is shown in Figure 8.6 (left). Detail view of the fabric in-place is shown in Figure 8.6 (right).



Figure 8.6: Surface Epoxy Sealing with Optional Fabric

8.5 Applying Penetrating Deck Seals

Widespread smaller cracks can be sealed using penetrating sealers to slow the ingress of water and chlorides into the deck or slab. The following is a procedure for applying penetrating deck sealers:

1. Repair any spalled or delaminated areas following the procedure provide below. Do not patch surfaces until fully cured.
2. Clean the surface by shot-blasting to remove all contaminants and to open the surface and cracks (see Figure 8.7).
3. Remove dust and debris by blowing off with oil-free compressed air and vacuuming.
4. Protect the expansion joints, and any other areas not to be sealed, from damage during preparation of the surface. The protection must be removed once the epoxy and aggregate have been applied and prior to initial set. Removing the protection must be done soon enough to ensure no harm to the adjacent sealed surface.
5. Mix components per manufacturer recommendations by volume. Mix only the quantity that can be used within its gel time.
6. Apply mixed product in accordance with manufacturer instructions, typically by pouring or spraying on the surface. Distribute material in accordance with manufacturer instructions, which is often evenly with a squeegee, roller or broom, maintaining a liquid head over cracks. Remove all excess material with squeegee or broom. Most products should be dispensed immediately after mixing to ensure adequate crack penetration time (see Figure 9.8).
7. Broadcast fine aggregate (sand). Application of aggregate must be of sufficient quantity to cover the entire surface in excess. No bleed through or wet spots shall be visible. The aggregate should be clean and moisture free. The broadcast of sand will wear off leaving sealed cracks behind (see Figure 9.9).
8. After initial cure, remove excess sand or aggregate by broom or compressed air. Excess sand may reduce surface friction if not removed.
9. When used as primer or pre -treatment prior to overlay consult manufacturer technical representative for appropriate curing before overlay. Some products are designed to act as a primer for a “wet on wet” application; other products should gel or cure prior to the application of overlays.



Figure 8.7: Shot Abrading Deck Preparation



Figure 8.8: Sealer Application



Figure 8.9: Broadcasting Sand (Aggregate)

8.6 Carbon Fiber Sheet (CFS) Bonding

CFS for reinforced concrete Maintenance and strengthening systems consists of a combination of carbon fiber reinforcing material and adhesive resin such as epoxies and other materials. This composite product is intended to enhance the capacity of the concrete deck slab and extend its service life. The function of the resin is to serve as a CFS for reinforced concrete Maintenance and strengthening systems consists of a combination of carbon fiber reinforcing material and adhesive resin such as epoxies and other materials. This composite product is intended to enhance the capacity of the concrete deck slab and extend its service life. The function of the resin is to serve as an adhesive bond onto the concrete surface and facilitate the transfer of stresses to and from the carbon fiber sheet.

The works shall consist of furnishing and installing two types of CFS as shown in Figure 8.10 for concrete strengthening systems in accordance with the plans and specifications. The systems shall be designed to strengthen and stiffen concrete bridge deck slab and tested by the Engineer to verify performance.

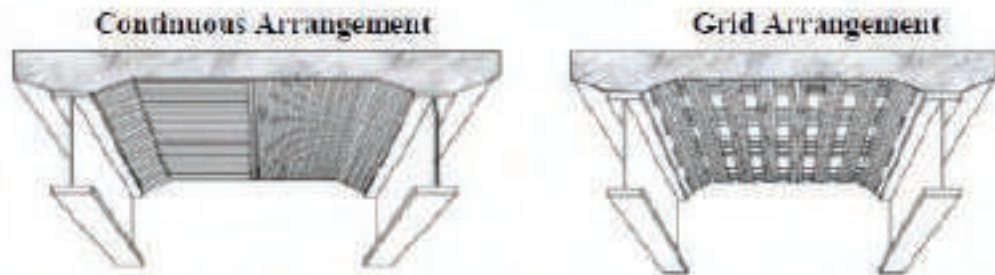


Figure 8.10: Arrangement of Carbon Fiber Sheet

8.6.1 Application Method

The related strengthening system for the concrete deck slab shall generally consist of woven carbon fiber sheet (CFS) reinforcing layers, bonded to the concrete surface with epoxy adhesive. The continuous arrangement is commonly used during the early stage of CFS bonding application at the bottom of the deck slab. However, in most of the cases observed, entrapped air which could not be easily released was found in the installed CFS. These air voids reduce bond strength between CFS and concrete surface and must be squeezed by roller. Meanwhile in the grid arrangement, CFS does not totally cover the required surface. Instead, the CFS is installed in strap-type method in both directions. According to experimental results, effectiveness of the second system is almost the same as that of the continuous arrangement. Moreover, entrapped air in the second system can be squeezed out easily using a roller. Thus, in this manual, the grid arrangement is recommended considering its ease of application, least cost and acceptable effectiveness. The carbon fiber sheet should be applied as two layers in both the longitudinal and transversal directions, as shown in Figure 8.11.

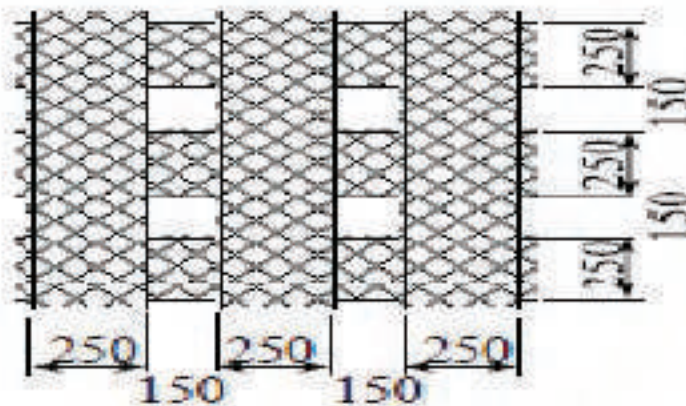


Figure 8.11: Carbon Fiber Sheet Arrangement

The CFS shall conform to the requirements of the specifications shown in Table 9.3 or equivalent ASTM Specifications.

Table 8.3: Specification of CFS

Property	Specifications	Test Method
Fiber Areal Weight	300 g/m ² above	ISO 18319
Design thickness	0.167 mm	ISO 18319
Tensile strength	3400 N/mm ² above	ISO 10406-2

Property	Specifications	Test Method
Tensile bond to concrete	1.5 N/mm ² above	ISO 10406-2
Young's modulus	(2.45±0.24) x 10 ⁵ N/mm ²	ISO 10406-2

The Epoxy Resin Adhesive shall conform to the requirements of the specifications shown in Table 8.4, or equivalent ISO Specifications.

Table 8.4: Specification of Epoxy Resin Adhesive

Property	Test Method	Unit	Primer	Epoxy Putty	Penetrating Epoxy Resin
Viscosity	ASTM D2393	mPa.s	1000 below	Paste-like	20,000 5000
Modulus of Elasticity	ASTM D695M	N/mm ²	1500/above	1500/above	1500/above
Slant Shear Bond to Concrete	ASTM C882	N/mm ²	15	15	15
Bond Strength to Concrete Dry/ Wet	ASTM D7234	N/mm ²	1.5 above	1.5 above	1.5 above

8.6.2 Work Requirement

The Contractor shall submit the Methodology Procedures of the Work to the Engineer for his review and approval before commencement of the work.

Surface preparation - Disc grinder or abrasive sandblasting is used to clean and smoothen the concrete surface. All concrete surfaces shall be clean, sound and free from surface moisture. Crack sealing or waterproofing shall be provided prior to concrete surface restoration. If water leaks through cracks on concrete surface to be covered with CFS, surface preparation and application of the CFS shall be in accordance with the approved manufacturer's specifications. Suitability of any changes to the application methods proposed by the Engineer. Cracks larger than 0.3 mm shall be injected with epoxy using a system/method approved by the Engineers suitability of any changes to the application methods proposed by the Engineer. Cracks larger than 0.3 mm shall be injected with epoxy using a system/method approved by the Engineer.



Figure 8.12: Surface preparation

Application of Primer - The surface is coated with a primer resin to increase the strength of concrete surface and improve its bonding with CFS. Contact surface shall be dry before coating with primer. The primer should be formulated and compatible to the carbon fiber material and should not be applied when raining, during stormy weather, when air is misty or when conditions remain unsatisfactory in the opinion of the Engineer.

Application rate shall be such as to ensure complete saturation of the contact surface. Primer should be cured between 2~3 hours before proceeding to the next step.



Figure 8.13: Application of Primer

Adjustment of Unevenness with Putty - Any concave, pores, gaps on the concrete surface must be smoothed using epoxy putty. This work involves application of epoxy putty onto the primer-coated concrete surface, using trowel or spatula, to smoothen the surface. The putty is applied when the primer is already tack-free. The application method is as follows:

- ✓ Mix 2 parts of epoxy putty until the mixture is homogenized.
- ✓ Apply the putty to smoothen the surface. Allowable unevenness after putty application is 1 mm/m



Figure 8.14: Adjustment of Unevenness with Putty

Application of Epoxy Resin for Undercoat - When the epoxy putty becomes tack-free, epoxy resin is applied to the concrete, acting as adhesive to bond the CFS. The molded composite is achieved as the resin impregnates into the CFS. Prior to undercoating epoxy resin adhesive, ambient temperature at the work site shall be checked to confirm the curing conditions for applying the resin.



Figure 8.15: Application of Epoxy Resin for Undercoat

The Contractor shall check and confirm that the primer and putty have become tack-free and that no clay and dust exist on the concrete surface prior to Engineers Investigation. If there is a time interval of longer than 3 days after the primer and putty application, the coated surface should be roughened with sandpaper and cleaned before the resin application

The contact surface condition shall be tack-free and application shall not be done when raining, air is misty, or when in the opinion of the Engineer, conditions are unsatisfactory to carry on with the work. The following specified quantity of the resin is estimated for reference only. The actual quantity should be determined in consideration with ambient temperature and manufacturer's recommendation subject to Engineer's approval.

- ✓ Mix the 2 parts of epoxy resin until homogenized,
- ✓ Apply the epoxy resin on the surface at a rate of 0.5 kg/sqm,

Putting Longitudinal Layer of CFS - Properly aligned CFS strips are installed in longitudinal direction to the adhesive coated concrete surface. Press the carbon fiber sheet by using plastic roller starting from the center toward the edge. The ideal length of carbon sheet is the full length of the slab. It should be avoided wrinkles with adequate caution.



Figure 8.16: Putting Longitudinal Layer of CFS

The CFS shall be applied as per the following:

- ✓ Stick the CFS in the longitudinal direction with a reasonable lapse of 20~30 minutes after the epoxy resin application,
- ✓ Press the CFS using a roller (plastic roller is preferred) starting from the center Towards the edge to squeeze out entrapped air before the epoxy resin sets.
- ✓ When lapping of two CFS is required, a lap length of not less than 20 cm shall be provided.

The specified normal curing time is only for reference purposes. The actual curing period should be determined in consideration of the ambient temperature and manufacturer's recommendation for the work site, subject to the Engineer's approval. ambient temperature and manufacturers recommendation

Putting Transverse Layer of CFS - Properly aligned CFS strips are installed in the transverse direction to the adhesive coated concrete surface. Press the carbon fiber sheet by using plastic roller starting from the center toward the edge. After the longitudinal layer CFS application, the transverse layer CFS is applied at right angles to but in the same manner as the longitudinal application.



Figure 8.17: Putting Transverse Layer of CFS



Figure 8.18: More application of CFS

Squeezing of Strip to Entrapped Air - For complete impregnation, entrapped air is squeezed out of the strips using the roller, before the adhesive sets. Do not apply the roller against the direction of the placed CFS to avoid damaging the material.



Figure 8.19: Squeezing of Strip to Entrapped Air

Over-Coat Application using Epoxy Resin - Mixing and application procedure for the overcoat shall be similar to that of the under-coating resin. The standard quantity of over-coating resin is 0.2 kg/m^2 . The actual quantity should be determined in consideration of the ambient temperature and manufacturer's recommendation for the work site, subject to the Engineer's approval.

Surface Protection - For safety purposes, fireproof protection coating may be applied to the finished surface.



Figure 8.20: Surface Protection

Quality Control and Investigation - The Contractor shall conduct a quality control program that includes, but is not limited to, the following:

- ✓ Investigation of all materials to ensure conformity with contract requirements, and that all materials are new and undamaged.
- ✓ Investigation of all surface preparation carried out prior to CFS application.
- ✓ Investigation of work in progress to ensure work is being done in accordance with approved manufacturer's instructions.
- ✓ Investigation of all work completed including verification of all Maintenances for debonding, and correction of any defective work.

After allowing at least 24 hours for initial resin saturate to cure, the Contractor shall perform a visual and acoustic tap test investigation of the layered surface. All voids, bubbles and delimitations shall be required in accordance manufacturer's recommendations. The Contractor shall conduct adhesion testing of the fully cured CFS and assembly using direct pull-off tests, at locations determined by the Engineer. Failure at the bond line at tensile stress below 14kgf/cm² (200 psi) shall be cause for rejecting the Maintenance works. A minimum of two pull-off tests per system(span) shall be performed. The test shall be completed prior to the application of the protective top coat on the CFS.

8.7 Full and Partial Depth Concrete Spall Repairs

Deck spalls are a common outcome of cracking, vehicular impact, or reinforcing corrosion in the deck or slab. Concrete patching can be used to repair the spalls. The following procedure will result in a quality patch. Note that if unsound concrete is removed to the extent that the bottom mat of reinforcement is exposed, stop the patching partial depth procedure and switch to the full depth procedure:

1. Identify the deck or slab location to be repair.
2. Hammer sound or chain the area around the spall to identify and mark adjacent unsound concrete. Mark off square areas outside the limits of the unsound concrete.

3. Combine patches closer than 1 foot into larger patches.
4. Sawcut to depth of at least 3/4 to 1 inch in geometric pattern marked in previous step. Be careful not to cut through reinforcing steel (see Figure 8.21 and Figure 8.22)
5. If the patch is full depth, protect the area under the work area from falling debris. This can be done by placing wooden or metal forms attached to the girders or soffits of the bridge (see Figure 8.23). Forms may be suspended from reinforcing steel by wire ties for areas of less than 3 square feet. In the case of larger openings, forms shall be supported from below by blocking to ensure the form can support the wet concrete weight.
6. Use a lightweight chipping gun (maximum weight of 30 pounds) to chip concrete out between the edge of the spall and the sawcut lines. Pneumatic hammers shall be worked at an angle of 45 to 60 degrees to the plane of the concrete being removed. Chip the concrete out max aggregate plus 1/4 inch below the top mat of steel or any other exposed reinforcing steel layers (see Figure 8.24). All loose concrete within the limits of the patch should be removed. Do not place chipping gun on the reinforcement.
7. Inspect edge of patch to ensure the limits of the delamination have been removed. If a concrete separation is visible (see Figure 8.25), increase the size of patch.
8. Use sand blasting equipment to remove any rust from exposed reinforcing steel and to clean the exposed concrete surfaces. Touch up epoxy bars.
9. Replace any missing or corroded steel with new bars providing proper overlap lengths. Consult an engineer as needed. Overlap requirements may require a large area to be chipped out, or drill and epoxy and core bars, or mechanically splice.
10. Galvanic anodes may be used for additional corrosion protection in the patch region.
11. Check the repair area one more time for any loose concrete or debris inside the spalled area and remove any found. Blow out the repair area with compressed air (see figure 8.26). Prepared concrete area is shown in figure no. 8.27.
12. Apply a bonding agent as specified by the engineer. A bonding agent may not be specified in all cases.
13. Weight the concrete in the patch area but do not allow standing water (Saturated Surface Dry or SSD condition)
14. Mix rapid setting concrete (or conventional concrete) and place concrete in the patch area up to the surface of the surrounding deck (see Figure 8.28). Vibrate the concrete, especially in the corners and around the reinforcing steel (see Figure 8.29).
15. Screed the newly placed concrete to the level of the surrounding deck. Use concrete finishing tools appropriate for the size of the patch.
16. Use a wet broom to provide a roughened surface in a transverse direction to traffic.
17. Cure the concrete per the manufacturer recommendation before restoring service on the bridge (see Figure 8.21). Concrete patches will continue to harden over time but can often support traffic within 2 to 4 hours for rapid setting patch materials.



Figure 8.21: Concrete Repair Photos. Full Depth Repair (left) and Partial Depth Repair (right)

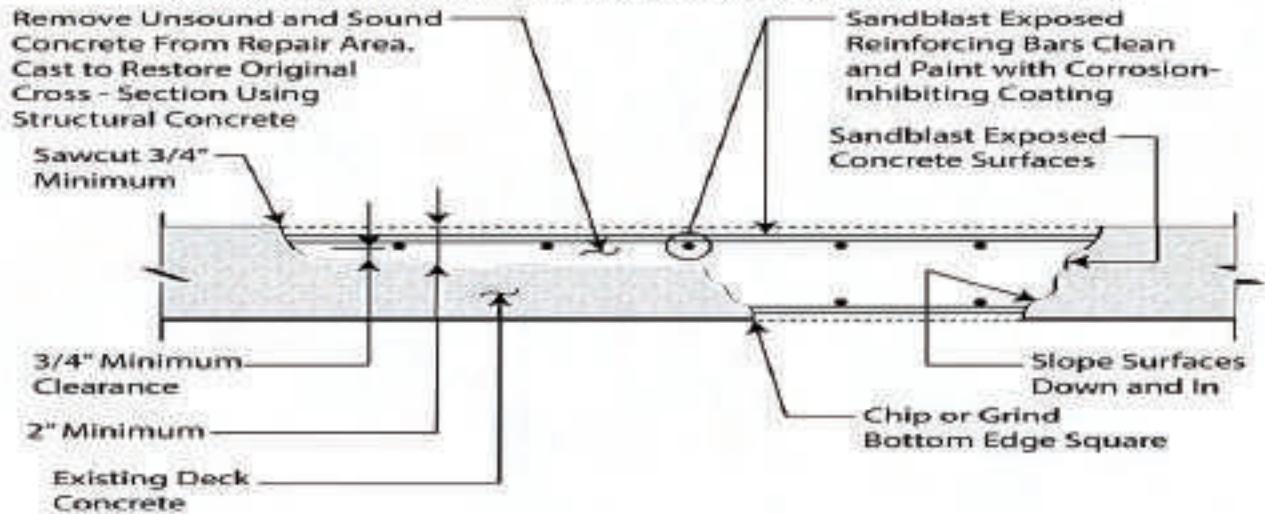


Figure 8.22: Typical Spall Repair Plan

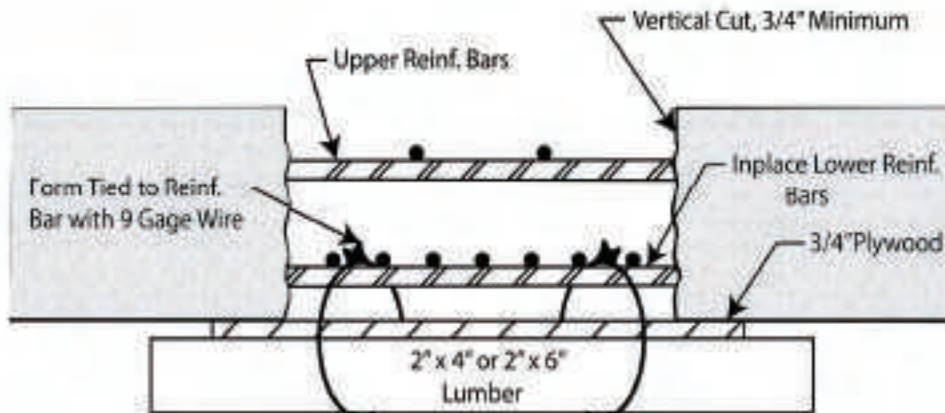


Figure 8.23: Typical Full Depth Repair Forming



Figure 8.24: exposed reinforcing steel layers



Figure 8.25: Delamination at Edge of Patch



Figure 8.26: Cleaning with Compressed Air



Figure 8.27: Prepared Patch Areas



Figure 8.28: Concrete Placement



Figure 8.29: Vibration and Finishing



Figure 8.30: Concrete Wet Cure Examples. Wet Cure Blanket (left) and Burlap Sheets (right)

8.8 Concrete Deck and Slab Delamination Repair

Delamination of a concrete deck or slab is typically caused by corrosion of the reinforcing steel that results in corrosion and expansive carbonation at the rebar level that cracks the concrete along rebar lines parallel to the surface of the deck or slab. Delaminations can be identified through acoustic methods such as chaining or impact echo methods or through density test methods such as ground penetrating radar. The following procedure can be used to repair deck or slab delaminations:

1. Identify the delaminated deck areas using one of the identified location methods. Deck chaining and hammer sounding are the most common methods (See figure 8.31).
2. Remove the delaminated concrete areas (See Figure 8.32 and Figure 8.33)
 - a. For a small percentage of delaminations, repair can be accomplished following the spall repair procedure.
 - b. For larger delamination areas, hydro-demolition equipment should be used to remove the concrete and clean the reinforcing steel of corrosion. Follow steps 3 through 11 for suggested procedure: Placement of concrete overlay (see Figure 8.34 and Figure 8.35)

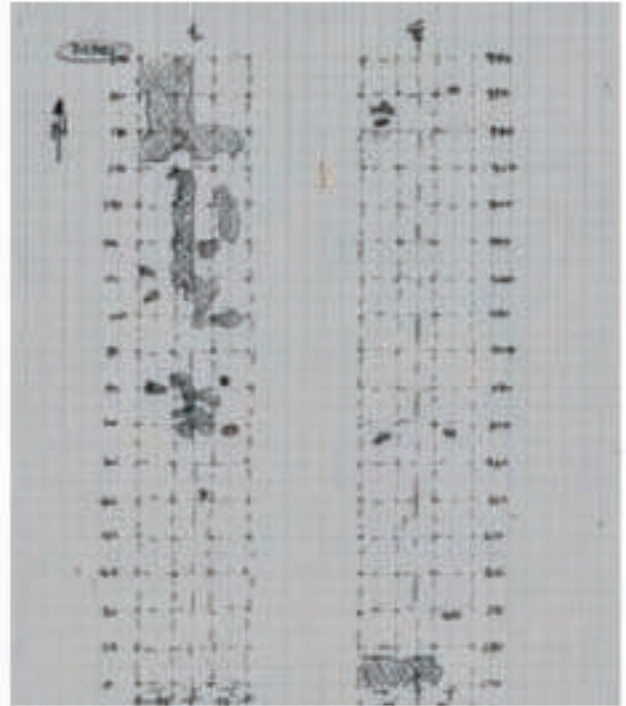


Figure 8.31: Chain Sounding Tools (left) and Delamination Mapping (right)



Figure 8.32: Removing the Delaminated Concrete



Figure 8.33: Sawcut, Chipped Out and Cleaned Concrete Removal Shown to the Top Mat Only



Figure 8.34: Placing the New Polyester Concrete



Figure 8.35: Curing the Concrete

8.9 Placement of Asphalt Overlay with Membrane

Asphalt overlays may be used as an application for waterproofing concrete bridge decks. There are two recommended practices for asphalt overlays on decks.

1. Asphalt overlay with membrane. A membrane is placed, and the asphalt is paved. The membrane is either a pre-formed sheet or a liquid hot-applied rubberized asphalt with polymer resins. For examples of these steps used in placing an asphalt overlay with membrane, where the following are observed: (a) Application of primer to the concrete deck, (b) Laying out the sheet membrane, (c) Heating the sheet membrane with a torch, (d) Sealing the overlap seams with a hand roller, (e) Completed membrane, (f) Compacting the hot mix asphalt.
2. Polymer modified asphalt (PMA). A PMA is paved, and no membrane is used because the PMA has low permeability.

The following suggested procedure applies to asphalt overlay with a membrane.

Before beginning, the deck needs to be swept or air blasted to remove loose material. Any sealers, coatings, or foreign substances that might affect the adhesion or curing of the asphalt or tack coat should be removed.

The steps for applying and asphalt overlay with a pre-formed membrane sheet are:

1. Apply the primer, such as asphalt emulsion.
2. Apply an adhesive, such as oxidized asphalt.
3. Apply an optional ventilating and leveling layer, such as sand asphalt.
4. Place the membrane, lapping the sheets.
5. Place an optional felt protection board.
6. Apply an asphalt tack coat.
7. Apply an asphalt base mixture, which has a life of 30 years.
8. Apply the asphalt surface mixture, which has a life of 10 to 50 years.

Examples of these steps are shown in figure 8.36.



(e)



(f)



(g)



(d)



(b)



(i)

Figure 8.36: Steps in the Installation of a Preformed Sheet Membrane

8.9.1 Polymer Modified Asphalt

The only step with a polymer modified asphalt is to pave the asphalt. Follow the membrane manufacturer recommendations regarding use of tack coat prior of the placement of a polymer modified asphalt overlay. The most common tack coat used for polymer modified asphalt is a non-diluted asphaltic emulsion.

8.10 Placement of Concrete and Polymer-Modified Concrete Overlay

Overlays can be an effective deck preservation method if the underlying concrete is properly patched and delaminations are addressed prior to the placement of the overlay. Proper hydrodemolition will remove all delaminations. Concrete damaged to or beyond the depth of the bottom layer of reinforcement requires full depth repairs. Partial and full depth repairs may be poured at the same time as the overlay. The following procedure applies for all concrete and polymer-modified concrete overlays:

Placement of concrete or polymer-modified concrete overlay

1. Remove deteriorated concrete and repair any delaminated or spalled areas. Concrete delaminated or spalled to or beyond the depth of the bottom layer of reinforcement requires full depth repairs. Partial depth delamination or spalls can be repaired with suitable patch materials and overlaid after appropriate curing. Alternatively, partial depth delaminations or spalls as well as full depth repairs may be poured at the same time as the overlay. See spall and delamination procedure.
2. In preparation for the placement of the overlay, clean the entire surface area of the deck by shot blasting or other approved means (See Figure 8.36) within 24 hours of the overlay placement. Protect metal deck drains and areas of the curb or railing above the proposed surface from the shot blast. The engineer must approve the use of scarifiers or milling machines in lieu of shot blasting. Hydrodemolition may also be used for to prepare the deck for an overlay.
3. Further clean the deck of any debris by air blasting with dry oil free air or vacuuming. Brooming is not acceptable.
4. Remove all loose concrete, dirt, paint, oil, asphalt, laitance carbonation and curing materials from patches and other foreign material from the surface of the deck.
5. Produce a surface relief equal to the International Concrete Repair Institute (ICRI) surface preparation level 6 to seven or ASTM E 965 Pavement Macro texture depth of 0.04 to 0.08 inch.
6. Remove any contamination of the prepared deck surface or surface of subsequent courses. Sand blast contaminated areas to produce an acceptable surface for placement of the overlay.
7. Protect deck drains so the overlay materials shall not pass into the drains.
8. Rain will not necessarily contaminate the surface. However, care must be taken that no contamination occurs.
9. For conventional concrete overlays, the surface shall be thoroughly water soaked without allowing standing water prior to the placement of the new concrete. For polymer modified concrete, follow the manufacturer instructions.
10. Place the first pour within 24 hours of preparing the deck surface. Deck surfaces exposed for more than 24 hours must be sand blasted and step 3 repeated prior to application of the overlay (see Figure 8.38)
11. Cure Overlay as specified.



Figure 8.37: Preparing the Deck with Shot Blasting.



Figure 8.38: Placing the Concrete Overlay

8.10.1 Placement of Polymer Overlay

The following procedure applies to epoxy polymer overlays.

Placement of Epoxy Polymer Overlay

1. Remove deteriorated concrete and repair any delaminated or spalled areas. Concrete delaminated or spalled to or beyond the depth of the bottom layer of reinforcement requires full depth repairs. Full or partial depth areas can be repaired with suitable patch materials and overlaid after appropriate curing. See spall and delamination procedure.

2. In preparation for the placement of the overlay, clean the entire surface area of the deck by shot blasting or other approved means (see Figure 8.39 a and Figure 8.39 b) within 24 hours of the overlay placement, to produce a surface relief consistent with epoxy overlay manufacturer requirements. Protect metal deck drains and areas of the curb or railing above the proposed surface from the shot blast. The Engineer must approve the use of scarifiers or milling machines in lieu of shot blasting. Hydrodemolition may also be used for to prepare the deck for an overlay.
3. Remove any contamination of the prepared deck surface such as traffic striping. Sand blast contaminated areas to produce an acceptable surface for placement of the overlay.
4. Further clean the deck of any debris by air blasting with dry oil free air or vacuuming.
5. Protect deck drains so the overlay material shall not pass into the drains.
6. Deck must typically be dry for a specified period of time in accordance with epoxy manufacturer recommendations, and must remain dry during the cure period. As such, epoxy overlay should not be placed if rain is forecast before completion of cure, typically 6 to 24 hours depending on manufacturer.
7. Mix epoxy as specified by manufacturer in batches (see Figure 8.39 b) that can be mixed and applied while maintaining a "wet edge" for beginning to spread the next batch.
8. Spread the mixed epoxy onto the substrate with a notched squeegee at the recommended rate (Figure 8.39 c and Figure 8.39 d). Place the epoxy to permit a continuous operation. Apply the second batch immediately behind the first batch.
9. Broadcast the aggregate immediately (Figure 8.39 e), but stop to maintain a wet edge. Broadcast the aggregate to complete saturation (approximately 1.1 pound per square foot). If wet spots develop, immediately broadcast additional aggregate until a dry surface is re-established. Cure overlay as specified.
10. Once the epoxy has cured, sweep the bridge deck to remove any loose aggregate (Figure 8.39 f).
11. Apply the second coat in the same matter but at a rate that is typically double the rate of the first coat due the roughness of the aggregate. The maximum recoat window is 24 hours.
12. Allow to cure, and sweep any loose aggregate from deck, prior to opening to traffic.

The photo sequence in Figure 8.39 shows several steps in the installation of a polymer overlay(i.e., a two-part epoxy resin in this example).

The following steps are shown in the figure:

- (a) Shotblasting to clean the deck surface,
- (b) Mixing the two-part epoxy,
- (c) Pouring the epoxy from buckets,
- (d) Spreading the epoxy with notched squeegee brushes,
- (e) Broadcasting aggregate onto the epoxy surface,
- (f) Sweeping up loose aggregate prior to opening the bridge to traffic.



Figure 8.39: Common Steps in Epoxy Polymer Overlay

8.11 Sealing Potholes and Gravel Edges on Bridge decks

A number of older type structures have gravel fill, 100 to 200mm thick, on the concrete deck. The gravel fill commonly has a sprayed seal to match the width of the road on the approaches. This leaves edges on the structures unsealed, allowing moisture penetration and weed growth due to the saturation of the gravel fill. The retained moisture permeates the concrete deck causing corrosion of the reinforcement over time. The fill may also be distorted in shape and causes cracks and potholes under the heavy truck loading, especially if the fill becomes saturated.



Figure 8.40: Potholes on the deck

Maintenance Method

1. All weed growth should be removed and the unsealed edges spray sealed to the kerb face to prevent further moisture ingress.
2. Deck drainage should be investigated to remove the surface water from the bridge and eliminate any water ponding to prevent any rutting and formation of potholes.
3. Surface cracking should be corrected with any crack sealant as detailed in treatment for crack repair of asphalt. Potholes should be repaired by removal of gravel in damaged area and backfilled with cold mix/hot mix asphalt (as specified), and well compacted using a vibrating plate or repeated drop weight on a heavy plate.

NB: Care should be exercised in sealing works to not restrict the functionality of scuppers by reducing the diameter, sealing over them or affecting surface levels and hence fall towards the scupper.

8.12 Steel Decks

8.12.1 Introduction

Steel bridge decks are common on moveable bridges and other bridge designs that require a lightweight deck. The weight of a steel deck per unit area is much less than that of concrete. This weight reduction of the deck means the machinery that lifts the bridge can be economized. For open grid steel decks, water passing through the deck and onto the superstructure and substructure elements can lead to deterioration. Steel grid decks can be filled or partially filled with concrete to prevent the water from passing through and causing deterioration. Steel bridge decks can be found in design situations where forming concrete is difficult or rapid deck construction is required. In these situations, corrugated, orthotropic or exodermis steel sections may be used by the designer.

8.14.2 Coating Systems – Paint and Galvanization

Paints provide protection from moisture, oxygen, and chlorides. Usually, three coats of paint are applied. The first coat is the primer, the next is the intermediate coat, and the final coat is the topcoat. Various types of paint are used, such as oil/alkyd, vinyl, epoxy, urethane, zinc-rich primer, and latex paints. Galvanizing can be used to protect steel decks. The galvanized coating retards the corrosion process and lengthens the life of the steel deck. The two dissimilar metals form an electrical current which reduces the corrosion in the steel. There are two methods of galvanizing steel decks (shop applied and field applied). Hot-dipping the steel deck member usually takes place at a fabrication shop prior to the initial placement of the steel deck. Steel sections are placed into a molten bath of zinc fusing the zinc to the surface of the steel.

When sections of the deck are too large or when field coating is required, the metallizing process is used. Metallizing is a thermal spray of zinc protective coating for steel. Zinc or aluminum wire is continuously melted in an electric arc spray or gas flame spray gun. Droplets of molten metal from the wire are deposited onto the steel forming the protective coating. This sprayed metal coating is both a barrier coating and a galvanic coating in one. The coating provides protection to the steel similar to galvanizing. Metallized coatings often have a top coat (sealer) to extend service.

8.14.3 Repairing Broken Connections

The welds or rivets that join the deck grids and hold them down may break over time (see Figure 8.41). It may be necessary to reinforce or replace damaged grid plates. When several plates have been damaged or dislodged it is best to cut out and replace that portion of the deck.



Figure 8.41: Typical Broken Grid Section

The following procedure can be used to repair a broken grid section:

Broken Grid Section Repair

1. Closely inspect the damaged area around the broken members for additional cracking or bent grid sections.

2. Determine a rectangular section that will address all localized damage.
3. Cut the grid section out within the identified repair area.
4. Cut a replacement grid section to fit within the newly created repair area.
5. Weld the repair section into the prepared hole in the grid deck. Welding should be done on both sides of every grid member.
6. Protect the newly welded connections with zinc rich primer or by metalizing.

8.12.4 Mitigating Steel Corrosion

Steel deck corrosion is primarily mitigated through the use of surface protection system such as paint or galvanization. Paints can be applied to deck sections that are properly prepared using the following steps:

Paint Application

1. Some states recommend pressure washing for chloride removal prior to proceeding with SSPC surface preparation guidelines.
2. Allow the steel to fully dry.
3. Apply zinc primer, followed by base and top coat applications of paint.
4. Grid decks can be difficult to paint due to the number of edges and tight angles. Special care should be taken to adequately cover all areas.
5. For spray applied zinc (metalizing), consult a professional. Light sand blasting may also be in order if there is corrosion present.

8.12.5 Deck Rehabilitation

If more than 20% of a deck has been spalled or delaminated, it should be replaced. However, deck rehabilitation with overlays may be an option if the deck is suffering from simple cracks and micro fractures. Deck rehabilitation with overlays is suitable in the following situations:

- Rehabilitation or replacement of bridge deck overlays
- Repair of the first rebar layer of an existing bridge deck
- Making bridge decks continuous or composite
- Economical repair with minimum traffic interruption

Deck Rehabilitation can be accomplished with multiple materials depending on the need, such as:

- Polymer-Modified Concrete
- Latex-Modified Concrete
- Reinforced Concrete

8.13 Replacing Expansion Joints with Damaged Headers

Expansion Joint Maintenance and Replacement



Figure 8.42: Expansion Joint Maintenance and Replacement

Before and After

Bridge expansion joints allow traffic to continue seamlessly across a bridge structure. They also help to accommodate movement of the bridge, shrinkage, and variations in temperature that cause the bridge to naturally expand and contract.

However, as a bridge expands and contracts, the movement can result in concrete cracking and spalling, leading to leaks and the eventual failure of the expansion joint. Any evidence of cracking and spalling along expansion joints should warrant repair or replacement.

Expansion joints are one of the first elements of the bridge that must be repaired to expand the bridge's lifespan. Monitoring and repairing these joints is an important step in the proactive maintenance of a bridge.

8.13.1 Replacing Armored Joint with New Concrete Headers

A suggested procedure for replacing armored joint with new concrete headers is shown below:

Replacing Armored Joint with New Concrete Headers

1. Remove the armor plates, anchors, and deteriorated concrete.
2. The extent of the concrete to be removed should be marked out and saw cut approximately 1 inch deep. The concrete should be removed by hand tools and small pneumatic hammers (30 pounds above the reinforcing steel, and 15 pounds below the reinforcing steel).
3. Set and secure the joint edge form.
4. Clean the area by abrasive blast cleaning.
5. Place required new reinforcing steel and remove/replace/lap existing corroded rebar.
6. Take care to ensure that the concrete surface of the header matches the deck grade. Setting the joint too high or too low will create undesirable vehicle loads.

8.13.2 Replacing Headers with Polymer Concrete (Elastomeric Concrete)

A suggested procedure for replacing headers with polymer concrete (elastomeric concrete) is shown below:

Replacing Headers with Polymer Concrete (Elastomeric Concrete)

1. Remove the armor plates and anchors (if they are present) and also deteriorated concrete.
2. The extent of the concrete to be removed should be marked out and saw cut approximately 1 inch deep. The concrete should be removed by hand tools and small pneumatic hammers (30 pounds above the reinforcing steel and 15 pounds below the reinforcing steel).
3. On occasion the surface will be an asphalt overlay; this surface shall be saw cut as well, and the asphalt removed.
4. Ensure the block out is cleaned of old joint seals, concrete debris, asphalt overlay, water proofing membrane, dirt, grease, and any other deleterious material.
5. Form the joint edge to achieve the proper opening. This is based upon the seal type, the span length, and the temperature.
6. The substrate to which the elastomeric concrete is to be placed against should be coated with a primer.
7. Place polymer concrete header in accordance with manufacturer recommendations. The header must be flush with the roadway surface and should be cured in accordance with manufacturer recommendations.
8. A number of seals can be used with this header type including compression seals, silicone seals, and strip seals.

8.15.3 Asphaltic Plug Joints (APJ)

Asphaltic Plug Joints are easily installed and a low-cost bridge expansion joint type. They can replace joints used with an asphalt wearing surface. Although they are easy to install, careful attention to the placement of the plug is required to obtain its desired performance characteristics (Figure 8.43). A block out for the plug joint should be provided. The overlay should be saw cut at the block out limits. It is desirable to saw cut freshly laid asphalt, this will require that the existing APJ be milled out and temporarily paved over. The block outs are typically 20 inches wide. The depth of the saw cut should be established by drilling holes to determine the pavement depth. The existing pavement is then removed with jackhammers and hand tools. Care should be taken to ensure the existing deck is not damaged during the process. Once exposed the concrete deck should be repaired and should be level to ensure the levelness of the bridging plate, it should not rock or deflect. The horizontal and vertical faces of the block out should be abrasive blast cleaned, and dried. The backer rod should then be installed 1 inch below the top of the deck surface. Once the backer rod has been installed the asphalt binder is placed above the backer rod and applied to all surfaces of the block out. Bridging plates are then installed; the plates should be centered over the joint and secured with locating pins along the centerline. The joint is then tanked (flooded) again with the binder material.

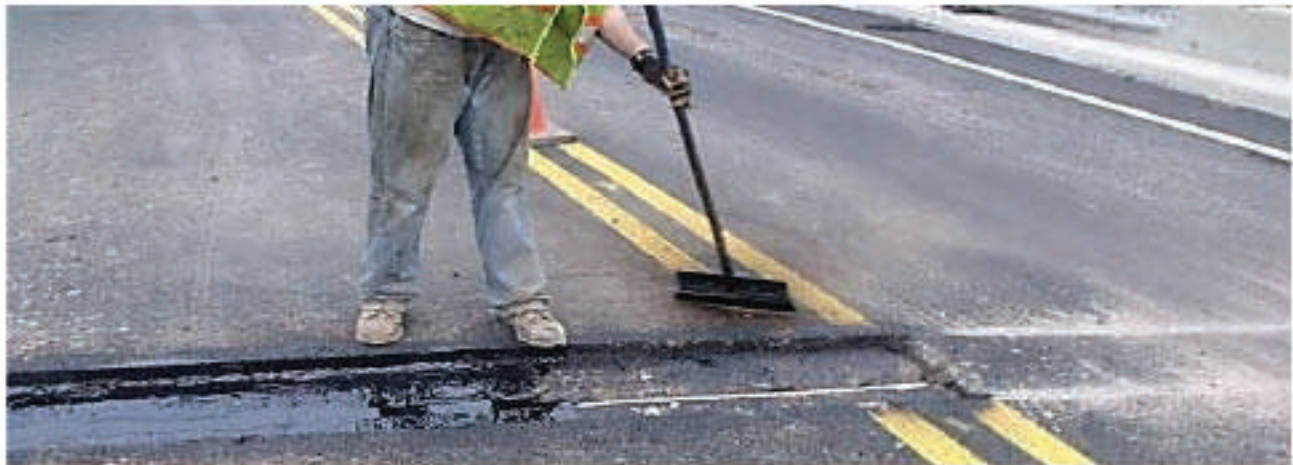


Figure 8.43: Asphaltic Plug Joint with Binder Placed atop Backer Rod Prior to Bridging Plate Installation

The aggregates and binder are prepared for mixing. The aggregate should be heated and then placed in a separate drum to be coated with the asphalt binder (this method ensures best results for coating the aggregate). The coated aggregate should then be loosely placed in the cut out, then leveled and compacted in 2-inch lifts. The aggregate mixture is then tanked (flooded) with the binder material to fill the voids, only the tips of the aggregate should be seen. The next lift is placed in a similar manner. In the final lift the aggregate should not protrude above the wearing course surface. The binder is applied to the final lift ensuring there are no voids and no stones protruding (Figure 8.44). The final course is topped with a thin anti-tracking layer to provide skid resistance. The temperature at the time of installation, as well as the temperature of the binder and the aggregate, is important to the installation of this joint. It is critical to know the temperatures of the materials and to calibrate any temperature monitors. The cleanliness of the block out and the aggregate are also important to monitor.



Figure 8.44: Plug Installation Showing Coated Aggregate and Binder

A suggested procedure for asphaltic plug seal installation:

Asphaltic Plug Seal Installation

1. Saw-cut and remove the wearing surface and waterproofing membrane (if applicable) from the deck.
2. The block out is then cleaned and dried, repairing concrete as necessary.

3. The backer rod is then placed allowing at least 1 inch depth of binder material from the top of the deck. Note: the binder material is typically polymer modified asphalt that is commercially available for asphalt plug seal installation.
4. Then the joint is tanked (flooded) with binder material heated to manufacturer specifications.
5. Next the bridging plate is installed (Figure 9.45), centered over the gap with locating pins, and the binder is placed atop the bridging plate.
6. The aggregate must be clean and graded properly then heated to the appropriate temperature. The heated aggregate is then mixed with heated binder to provide a coating.
7. The coated aggregate is then placed in the block out in a maximum of 2 inch lift.
8. The binder is heated to the appropriate pouring temperature and poured/flooded into the block out with the coated aggregate.
9. This process is repeated until the block out is filled.
10. A thin layer of anti-tracking material is placed upon the top surface to provide skid resistance.



Figure 8.45: Bridging Plate Installation

8.14 Repairing Bridge Footpath and Kerb

Footpaths should be checked for hazards. The concrete kerbs and footpath slab may be cracked, spalled, or broken due to wheel impact or due to insufficient concrete cover to the reinforcement. The ends of the precast kerb units may be spalled or broken away from the cast in-situ kerb sections. Footpath precast slabs can crack severely or suffer vehicle damage. Uneven surfaces due to slab rotation, uneven support or errors with the original levels can present hazards to pedestrians. A large step at the end of the deck where it abuts the approach footpaths can also be hazardous. Asphalt and macadam footpaths can become uneven with cracking, heaving, shoving, broken up areas or pooled water. Furthermore, the steel armouring (edging) retaining the asphalt or macadam may become loose, dislodged or corroded over time.

A large gap, e.g. in the expansion joint or joint between precast concrete footpath slabs over the top of embedded services, could permit a high heel to slip into the joint gap causing a fall. If the units have a differential in the height of 10 mm, this could present a tripping hazard.

Maintenance Method

1. Badly spalled or broken out areas should be re-cast or patch repaired in accordance with Spalling and Scaling Repairs.
2. Steps in precast slabs should be ground down to reduce the risk of pedestrians tripping on the steps. Alternatively, the approach footpaths should be raised to the bridge deck level.
3. Badly cracked and broken slabs that have failed will require replacement. This type of work should be programmed as repair work.
4. Macadam surfaces which are uneven, very porous and leak badly should be dug out and replaced with asphalt or concrete to minimise future maintenance.

Steel armouring should be secured or replaced to retain the infill; uneven asphalt should be repaired to provide a smooth well-drained surface.

8.15 Preventive and Basic Maintenance of Railing Systems

Preventive maintenance can be thought of as the process of protecting the railing system from the unavoidable deterioration caused by moisture, de-icing chemicals, traffic vibration and the environment. Sometimes materials are flawed from the start, such as thin spots of galvanizing on steel or cracks in concrete due to improper curing when poured, which can cause early failure. It should be kept in mind that any railing system begins to deteriorate the day it is constructed. It is the purpose of preventative maintenance to slow the failure process as much as possible by using techniques and materials that are considered best practices. This section presents some methods to provide such maintenance; it is not meant to be all inclusive.

Prior to working on any bridge railing, you should determine the extent of maintenance and supplies that will be required, as well as what signs and traffic controls are needed.

8.15.1 Concrete Maintenance

In concrete maintenance work following point has to be look for

- Cracks
- Spall
- Scalling
- Impact Damage
- Deteriorating Concrete/Reinforcing
- Vegetation Growing in cracks

Pressure washing of barriers to remove salt or other contaminants and dirt should occur on a regular basis. Just like the deck of the bridge, the concrete railing is as susceptible to cracks and spalls from salt damage. Therefore, it is essential to protect the surface and seal cracks with the appropriate sealant. The same methods of crack sealing can be used on the deck can be used on the railing. Application of concrete sealants and coatings will protect the reinforcing steel from corrosion by stopping or minimizing the intrusion of water and salt through the concrete. There are more options for protective coatings because the railing does not receive the same traffic wear as the deck; i.e