

**Wisconsin Highway Research Program**

**Project NO. 0092-00-15**

**Non-Destructive Testing of Wisconsin Highway  
Bridges**

**Final Report**

by

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## **Executive Summary**

This project was initiated by the Wisconsin Highway Research Program to develop appropriate guidelines for non-destructive evaluation (NDE) of bridge structures. The current method of bridge inspection is based largely on visual inspection. The integration of NDE techniques into bridge inspection can yield useful data with regards to bridge condition. This data can be utilized in bridge management systems to aid those involved in the rehabilitation and assessment of bridges.

Bridge deterioration mechanisms are numerous and vary from bridge to bridge. A guideline for relating various deterioration mechanisms to proper non-destructive evaluation techniques does not currently exist in an easy to use form. The choice of the proper NDE technique is dependant upon several variables. This is not only dependent upon the bridge structure, but also the inspection agencies' personnel skill and other available equipment and resources. Guidelines relating common flaw indications during visual inspection to appropriate non-destructive testing techniques could provide an effective aid to those responsible for bridge inspection and assessment. This document is developed to present such guidelines.

Since the majority of bridges (approximately ninety-five percent) in the State of Wisconsin are either steel or concrete, the guidelines in this report cover these two bridge types. The guidelines also include special components and structures such as substructures, bearings, and moveable bridges. The document is comprised of several sections, divided primarily by these bridge types. The guidelines also include descriptions of defects and possible causes of problems in both steel and concrete bridge materials. This will facilitate the identification of the underlying cause, leading to the identification of the appropriate test or tests for a given situation. This document also includes an index of the different testing techniques that are suitable for bridge condition evaluation. The index gives a description of the technique, possible standard test procedures and other relevant references.

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## Introduction

The intent of this document is to provide guidelines for basic inspection of reinforced concrete, prestressed or post-tensioned concrete, and steel bridge structures using basic methods of non-destructive evaluation.

The structural integrity and safe load-carrying capacities of our state and national bridges ensure the safety of the public and growth of our economy. An effective assessment of the condition of in-service bridges can help in better management of these structures and can provide useful information to make critical decisions regarding their replacement or rehabilitation. Currently, bridge condition information is obtained primarily through visual inspection. A properly performed visual inspection can provide valuable information regarding bridge condition. Visual inspection can identify problem areas that may need further inspection. Non-destructive evaluation (NDE) can provide knowledge that may be impossible to deduce from visual inspection alone. The integration of both visual inspection and non-destructive evaluation methods is key to a complete bridge condition assessment. Guidelines relating common flaws identified during visual inspection to possible non-destructive testing techniques can provide an effective aid to those responsible for bridge inspection. In the sections that follow, the basic inspection of reinforced concrete, prestressed or post-tensioned concrete, and steel bridge structures is discussed and relationships between typical visual clues obtained during visual inspection and possible follow-up non-destructive testing techniques are given.

The United States Federal government requires inspection every two years on bridges over twenty feet in length on all public roads <sup>(1)</sup>. In some special cases, this frequency can be decreased to every four years based upon numerous factors including satisfactory performance, favorable prior experience and analysis, structure age, traffic, etc. Current Wisconsin standards for bridge inspection reflect requirements for a two-year or shorter inspection cycles <sup>(2)</sup>. Several Midwestern states—Illinois, Indiana, Iowa, Michigan, and Minnesota—conduct routine inspections on a biennial basis. The State of Wisconsin previously required annual inspection of Wisconsin bridges; this code was changed in August 1999 to reflect federal requirements. The inspection terms are the minimum required by law for *routine* inspections. *Interim* inspections occur at more frequent intervals. These are “hands-on,” detailed inspections that may be required depending upon previous routine inspection results. The interim inspection will likely involve some non-destructive or exploratory techniques. Two other important guidelines for bridge inspection are also available. Federal and State of Wisconsin inspection standards both refer to the “*Manual for Condition Evaluation of Bridges 1994*” from the American Association of State Highway and Transportation Officials (AASHTO) in various capacities. The Federal Highway Administration’s (FHWA) “*Bridge Inspector’s Training Manual 90*”, is also referenced throughout Wisconsin bridge inspection standards. These contain valuable information regarding numerous aspects of bridge evaluation.

The majority of Wisconsin bridges—approximately ninety-five percent— are comprised of reinforced, pre-stressed or post-tensioned concrete, or steel construction <sup>(3)</sup>. This document discusses only these types of structures. The deterioration mechanisms for steel or concrete are often universal, regardless of structure type. This document presents a brief discussion of the deterioration mechanisms for both steel and concrete. The visible defects in bridge structures are indications of the underlying deterioration mechanism. Non-destructive testing techniques are often suited to a specific type of deterioration mechanism. These techniques attempt to validate the assumed deterioration mechanism and to determine the extent of the damage caused by that mechanism. The critical relationship between visual inspection and relevant non-destructive evaluation is provided through the identification of the deterioration mechanism.

## **I. Fundamental Aspects of Bridge Inspection**

Bridge inspection is a major part of a bridge maintenance program. A full understanding of the condition of a bridge structure will yield important information; data that can be utilized in making critical decisions regarding future actions on a bridge structure. The best means of ensuring proper inspection is knowledge of the fundamental aspects of bridge inspection. These aspects include:

- A. Bridge inspection personnel qualifications
- B. Inspection preparation
- C. Inspection types (routine, in-depth)
- D. Safety
- E. Appropriateness of various NDE techniques
- F. Quality control

### **A. Bridge Inspection Personnel Qualifications**

Only qualified individuals should perform bridge assessments. The State of Wisconsin requires individuals involved in bridge inspection to meet the minimum requirements of AASHTO's *Manual for Condition Evaluation of Bridges* <sup>(4)</sup>. These requirements refer to the responsibilities of the top two levels of bridge inspection personnel. These are:

- Inspection Program Manager
- Inspection Team Leader

The inspection program manager is the leader at the highest level of the inspection department. He/She is the leader of the organizational unit responsible for bridge inspection and acts as an authority to all team leaders. An ideal inspection program manager will have extensive knowledge and experience in a variety of areas of bridge engineering: non-destructive testing, design, construction, rehabilitation, maintenance, and load rating. He/She will possess good judgment in evaluating bridge condition or gauging the urgency of a particular problem. He/She will realize when lack of expertise limits possible bridge evaluation, and seek consultation with relevant engineers in areas such as: structural design, construction, non-destructive testing, materials, maintenance, electrical equipment, machinery, hydrodynamics, soils and emergency structural repairs. However, the *minimum* requirements of the inspection program manager are either of the two shown below <sup>(5)</sup>:

- Registration, or qualification for registration, as a professional engineer in the state of Wisconsin; or
- Possess a minimum of 10 years of bridge inspection experience in a responsible capacity and have completed a training course based on the FHWA's "Bridge Inspector's Training Manual"

It must be noted that although the first requirement is indeed listed in the manual, the professional registration alone, or qualification for it,

cannot be adequate for any individual to lead any inspection effort at a state level.

Also, the program manager should have some prior experience in NDE. This person is the authority to the field personnel and should be knowledgeable in the various field aspects of the related NDE problems that may arise.

The inspection team leader is responsible for on-site bridge evaluation. His/Her responsibilities include: planning, preparing, and performing the field bridge inspection. He/She should be present at all times during a bridge inspection. The inspection team leader should possess either of the two following *minimum* qualifications:

- A minimum of 5 years of bridge inspection experience in a responsible capacity and have completed a training course based on the FHWA's "Bridge Inspector's Training Manual"
- Certification in Bridge Safety Inspection at Level III or IV from the National Institute for Certification in Engineering Technologies (NICET).

Also, the team leader should have prior experience in NDE. This person is the authority to other team members and should be knowledgeable in the proper operation of testing equipment and interpretation of testing results.

The inspection program manager, or one holding the same qualifications, can also function as the inspection team leader. These requirements are consistent with federal standards listed in the Code of Federal Regulations<sup>(6)</sup>.

## **B. Inspection Preparation**

Inspection preparation is an important aspect of bridge inspection. Proper inspection preparation and planning ensures an effective and efficient inspection. A review of bridge history can yield problem areas that may justify extra attention. There are several factors to consider in bridge inspection planning. The inspection plan should be developed based on reviewing the bridge record. The bridge record contains the cumulative history of the bridge. Although the contents of a bridge record may vary, several of the following are likely contained in the bridge record <sup>(7)</sup>:

- Plans
  - Design and Construction Plans
  - Work and Shop Drawings
  - As-Built Drawings
- Specifications
- Correspondence

- Photographs
- Material Information
  - Material Certification
  - Material Test Data
  - Load Test Data
- Maintenance and Repair History
- Protective Coating History
- Accident Records
- Bridge Posting Actions
- Permitted Load Information
- Flood Data
- Traffic Data
- Inspection History
- Inspection Requirements
- Structural Inventory and Appraisal Sheets
- Past Inventories and Inspection Records
- Rating Records

A site visit may also aid the inspection preparation. The following items should be considered when planning a bridge inspection <sup>(8)</sup>:

- Type of inspection needed
- Number of personnel needed
- Traffic control needs
  - Ramp closures
  - Detours
  - Time Restrictions
  - Flagmen needs
  - Coastguard warnings
  - Permission
- Access Equipment (including qualified personnel for operation)
  - Scaffolding
  - Aerial lifts
  - Truck lifts
  - Reach-alls
  - Swing staging
- Other Tools and equipment (including power) needed
- Estimation of inspection duration
- Coordination with other relevant agencies
- Identification of previously defined problem areas
- Determination of need for underwater inspection
- Determination of non-destructive testing integration based on previous visual inspections
- Identification of details that may require special inspection
  - Fracture Critical Members
  - Fatigue-prone details

- Non-redundant members
- Unusual or Special details

Member identification is also an important aspect of inspection. Appropriate identification insures the correct area can again be located accurately for future repair or further examination. If drawings or previous inspection reports are available, use the same identification system for future inspections. If neither drawings nor previous inspection reports exist, the following system from the FHWA's Bridge Inspection Training Manual could be used for identification:

### **FHWA Identification Numbering System** <sup>(9)</sup>

The following outlines the numbering system suggested by the FHWA with regards to major bridge components.

**Orientation** - Route direction can be determined based on mile markers or stationing. This direction can be used to identify the beginning and end of the bridge.

**Deck** - The numbering system should include deck sections between construction joints, expansion joints, parapets, and railings. These are to be numbered consecutively from the beginning to end of the bridge.

**Superstructure** – The numbering system should include the spans, beams, girders, stringers, and floor beams. Spans should be numbered consecutively between abutments, piers, or bents. Span 1 being between the first and second abutment, pier, or bent at the beginning of the bridge. Multiple longitudinal beams or girders (parallel to traffic flow) should be numbered consecutively from left to right facing in the route direction. Stringers can be numbered in a similar fashion. Floor beams should be numbered consecutively from the beginning to the end of the bridge. If two sets of floor beams exist, the sets can be delineated by direction (i.e., east, west, north, south, etc.).

**Trusses** – The truss member connection points can be grouped by their vertical location (upper, middle, lower, etc.). The connection points should be numbered starting with 0 from the beginning and increasing to the end of the bridge. Upper connection points will be represented by "U," i.e. U0, U1, etc. Middle and lower connection points follow the same notation pattern, i.e. M0, M1 and L0, L1.

**Substructure** – The substructure numbering system should include abutments and piers. Abutments will start with Abutment 1 at the beginning of the bridge and end with Abutment 2 at the end. Piers are to be designated consecutively. Numbering will start with Pier 1 at the beginning of the bridge.

Although not a requirement, an appropriate inspection sequence could also be developed and followed to save time and achieve efficiency. Inspection sequences are dependent upon many factors: type of bridge, condition of bridge components, overall condition, inspection requirements, size and complexity of bridge, traffic, special procedures, and procurement of equipment.

### **C. Inspection Types**

The utilization of a particular inspection type is dictated by structure age, condition, and use. There are five basic types of bridge inspections<sup>(10)</sup>:

- Initial Inspection
- Routine Inspection
- In-Depth (Interim) Inspection
- Damage Inspection
- Special Inspection

The initial inspection is the first inspection after construction, either new construction or retrofitting, or change of ownership. This inspection provides a baseline for all subsequent inspections. Problem areas, or potential problem areas, including fracture critical members, are identified in this inspection. The load capacity is determined through analytical means to set the bridge load rating.

Routine inspections are regularly scheduled inspections that monitor any changes from previously observed conditions. The State government sets the frequency of the routine inspection. Results of a routine inspection will identify areas for further in-depth, or interim, inspections. Areas critical to the load-carrying capacity of the bridge will be monitored closely. Visual inspection is the primary method used in routine evaluations. Some simple non-destructive techniques can be integrated with visual inspections such as hammer sounding, rebound hammer, dye penetrant, and magnetic particle. The results of a routine inspection need to be documented thoroughly for the bridge file. Recommendations for further inspection and maintenance need to be included in the documentation as well. Appropriate photographs and sketches should be included to aid in the description of the bridge condition.

In-depth inspections are close-up inspections of areas identified by routine inspections. These inspections are hands-on and may necessitate the use of special lift equipment to gain this close access. In-depth inspections typically incorporate various non-destructive techniques. Routine inspections identify defects; the extent of a particular defect is determined during the in-depth inspection. These inspections can occur above or below the water line. Scour investigations may be part of the in-depth inspection. In-depth inspections should be documented completely, noting the procedures used and resultant findings.

Damage inspections are unscheduled inspections that assess physical damage resulting from human factors and the environment. The findings of these inspections may necessitate an emergency load restriction or closure of the bridge. The extent of misalignment, section loss, or other damage must be carefully documented. The potential for litigation may exist; the extent of documentation should reflect this possibility. A timely in-depth inspection should supplement this inspection.

Special inspections may be needed to supplement other inspections. These inspections may involve specialized techniques. Diving underwater inspection is one type of special inspection. Federal standards require inspection of underwater members every five years <sup>(11)</sup>. Other specialized techniques involving advanced non-destructive techniques may be required during a special inspection instead of an in-depth inspection.

#### **D. Safety**

Safety is a major concern in the field. Bridge inspection is inherently dangerous and therefore requires continuous attention from each member of the inspection team. Attitude, alertness, and common sense are three important factors in maintaining safety. All safety procedures should be followed, such as:

- Occupational Safety and Health Administration (OSHA) rules and regulations
- Wisconsin DOT requirements
- Specific inspection team requirements

Written safety procedures should be available for any given situation that can be encountered in the field.

Traffic control is integral to both the safety of the inspector and the driving public. There are four basic rules to follow for good traffic control and inspector safety <sup>(12)</sup>:

- Inform the motorist of what to expect; avoid a sudden surprise
- Control the speed of the motorist
- Provide a clearly marked path for the motorist to travel through the inspection zone
- Use a shadow vehicle (crash truck) equipped with a crash attenuator as protection from the traffic

Each inspector is ultimately responsible for his or her own safety. The inspector should understand the operation of access vehicles and safety apparatus. If questions regarding the safety equipment or working environment exist, these should be answered before inspection continues. If an accident occurs, it is essential to report it as soon as possible.

### **E. Appropriateness of NDE Techniques**

A proper non-destructive testing technique is selected based on a variety of factors including:

- Structure type and possible deterioration mechanism
- Safety requirements
- Weather
- Overall practicality and feasibility
- Availability of testing apparatus
- Access to testing areas on the bridge
- Expense of the technique
- Possible subjectivity involved in test results interpretation
- Needs of the bridge management system

### **F. Quality Control**

Quality control is a vital part of any type of construction. Shop and construction errors can have a major effect on the problems that can exist in a bridge structure. Inspection is needed to eliminate these errors to ensure that the structure is built as designed. The inspection needs to be undertaken by well-trained individuals at every level of bridge production: design, fabrication, construction, and maintenance.

Fabrication errors such as defective welds, improper material sizes, improper material lengths, etc., need to be detected by quality control personnel at the shop and fabrication levels to ensure such errors do not make it into field use. Construction inspectors need to ensure that the bridge is built per design plans with correct material strengths, material sizes, and that no damage occurs during various retrofitting projects such as deck replacement. The maintenance personnel's duty is to examine the bridge for their deterioration during in-service life.

## **II. Concrete Defects and Causes**

Concrete is a versatile and cost effective structural building material. It is easily formed and generally fire resistant. However, concrete is permeable to water and has volumetric changes due to temperature and moisture. It exhibits somewhat uniform mechanical properties in all directions (isotropic). However, permanent deformation under sustained loads (creep) may occur. Concrete does not deform elastically as much as steel does, therefore sudden failure can occur with overload. Concrete is weak in tension and shear, but high in compressive strength. The tensile weakness of concrete is the reason for the utilization of reinforcing steel in combination with concrete.

There are numerous NDE testing techniques that are often suited for determination of the extent of damage due to a particular deterioration mechanism. Deterioration mechanisms can occur singularly or simultaneously. Knowledge of the deterioration mechanisms provides valuable information regarding the proper non-destructive technique to use, the appropriate repair technique, and the timeliness of that repair. Subsection A discusses basic visual indicators that may be encountered during a typical visual inspection. The relationship between these indications and possible causes is given in subsection B.

## A. Concrete Problems/Visual Indications

A visual inspection of reinforced and non-reinforced concrete structures can reveal various flaws. The visual clues may reveal the underlying problem directly or may only give a possible indication of a larger underlying problem. Concrete problems may be due to one or more causes. The following describes typical visual indicators encountered during a visual inspection:

**1. Cracking** – Cracks are largely linear fractures in concrete. Cracking can occur by structural and non-structural means. The age, material used, loading pattern, environment, crack orientation, and other factors can help determine the cause of cracking. The following delineates some of the different types of cracking found in concrete <sup>(13, 14)</sup>.

**a. Checking** – Shallow cracks at irregular, closely spaced intervals on the concrete surface (Figure 1).

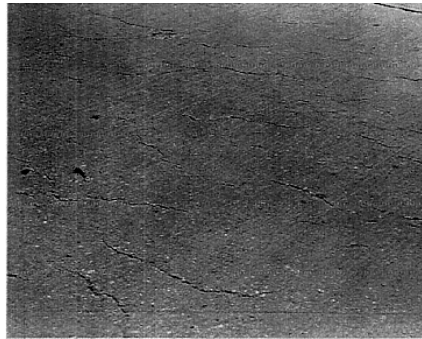


Figure 1 - Checking

(Source: ACI 201.1 R-92)

**b. Craze Cracking** – Fine random cracks in the concrete surface (Figure 2).

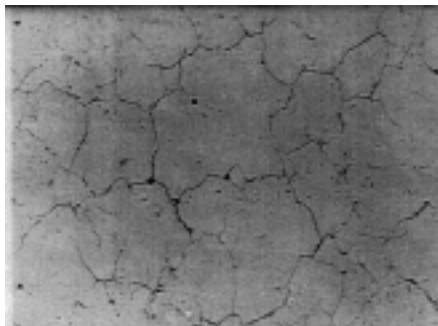


Figure 2 - Craze Cracking

(Source: ACI 201.1 R-92)

**c. D-Cracking** – A series of cracks in concrete near and somewhat parallel to joints, edges, and large structural cracks (Figure 3).

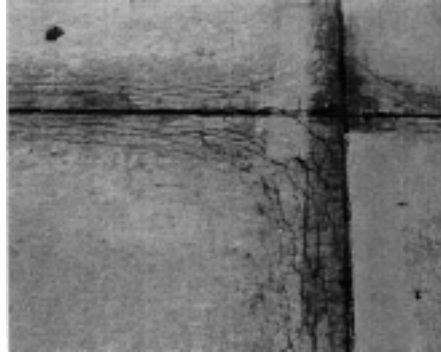


Figure 3 - D-Cracking  
(Source: ACI 201.1 R-92)

**d. Diagonal Cracking** – Cracks occurring at roughly 45° angles to either vertical or horizontal directions (Figure 4).

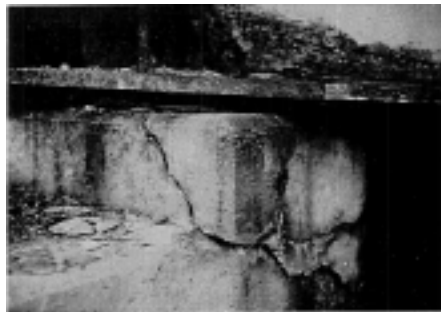


Figure 4 - Diagonal Cracking  
(Source: ACI 201.1 R-92)

**e. Drying Shrinkage Cracking** – Cracking caused by restraint of aging concrete as it tries to shrink (Figure 5).

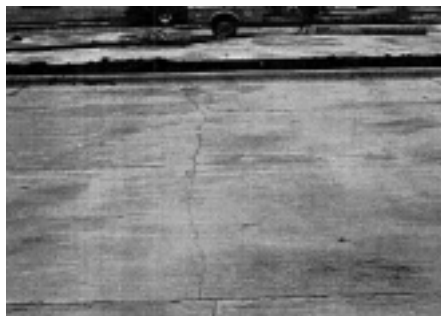


Figure 5 - Drying Shrinkage Cracking  
(Source: ACI 201.1 R-92)

**f. Pattern Cracking** – Cracks on the concrete surface that form various patterns. These cracks can interlock or nearly interlock on the concrete surface (Figures 6 to 11).

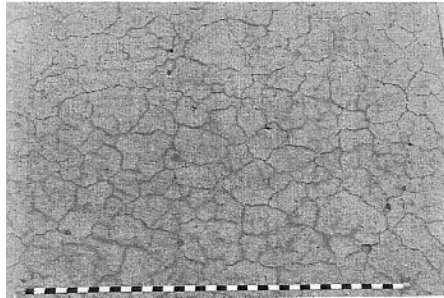


Figure 6 - Medium Pattern Cracking  
(Source: ACI 201.1 R-92)



Figure 7 - Fine Pattern Cracking  
(Source: ACI 201.1 R-92)

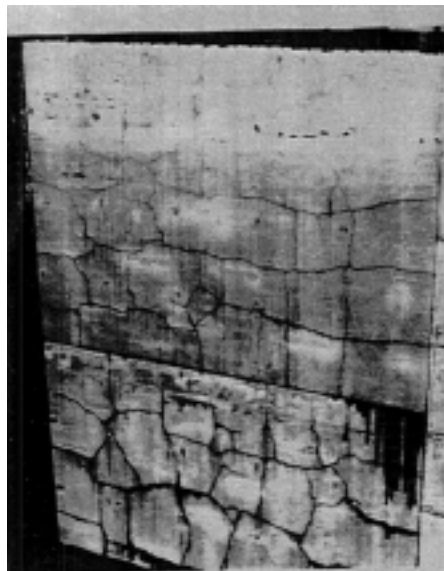


Figure 9 - Restraint of Volume Change Pattern Cracking  
(Source: ACI 201.1 R-92)



Figure 8 - Alkali-Silica Reaction Pattern Cracking  
(Source: ACI 201.1 R-92)



Figure 10 - Wide Pattern Cracking  
(Source: ACI 201.1 R-92)

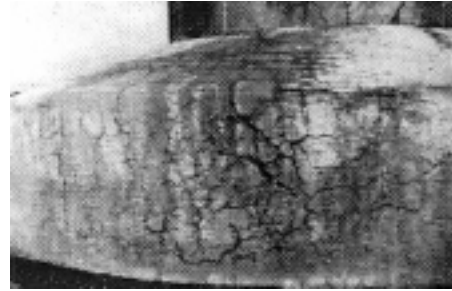


Figure 11 - Alkali-Carbonate Reaction Pattern Cracking  
(Source: ACI 201.1 R-92)

**g. Plastic Shrinkage Cracking** – Cracking that occurs soon after the concrete is placed, while it still exhibits plastic material behavior (Figure 12).

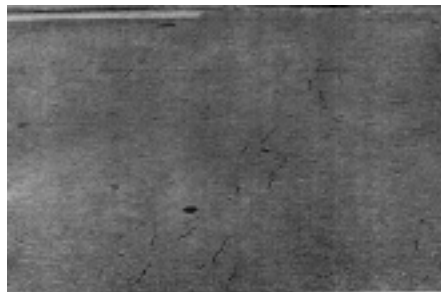


Figure 12 - Plastic Shrinkage Cracking  
(Source: ACI 201.1 R-92)

**h. Temperature Cracking** – Cracking caused by restraint of the concrete member as it expands and contracts. This is similar to shrinkage cracking. This can occur at three distinct times in a concrete member's life: 1) Soon after placement, the hydration reaction of cement for large concrete pours causes an extreme rise in temperature. 2) Changes in climatic conditions also cause temperature changes in a concrete member. 3) Other factors, such as fire, that can cause change of temperature.

**i. Transverse Cracking** – Cracks that develop at an angle perpendicular to the long direction of the member (Figure 13).

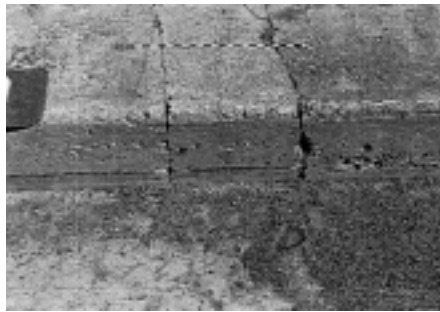


Figure 13 - Transverse Cracking

(Source: ACI 201.1 R-92)

**2. Physical Damage** – Physical damage can greatly reduce the load carrying capacity of a member, depending upon the extent. Bridges traversing roadways are very susceptible to damage from impact by over-height trucks.

**3. Delamination** – Delamination is the separation of the concrete above the outermost layer of reinforcing steel or above another layer of concrete or subsurface material. This may occur due to reinforcement corrosion, ineffective bond between layers, or various environmental effects.

**4. Efflorescence** – Efflorescence can be observed on concrete surface in the form of a dirty white coating around crack openings. Efflorescence is the re-crystallization of contaminating salts and calcium carbonate leached out of the cement paste. The salts work their way into the concrete in liquid form to the level of reinforcement and beyond. The salt solution can cause reinforcement corrosion and subsequent cracking in the concrete. The solution attracts the calcium carbonate and then leaches its way out of the crack, where it contacts the atmosphere. This, in turn, changes the liquid salt solution to a solid coating, or crystals, on the surface (Figure 14).



Figure 14 - Efflorescence

(Source: ACI 201.1R-92)

**5. Exudation** – Exudation is the presence of a gel discharged through cracks and other openings in the concrete. This gel is the result of alkali-silica reactivity. When this gel interacts with the atmosphere, it takes on a white appearance (Figure 15).

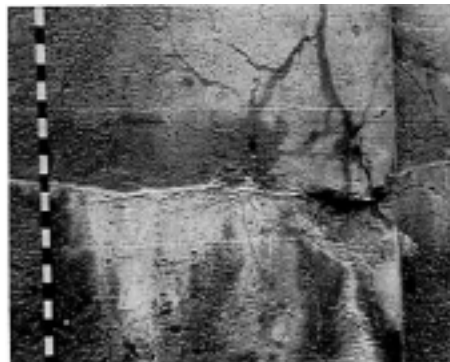


Figure 15 - Exudation

(Source: ACI 201.1R-92)

**6. Honeycombing** – Honeycombs are voids which occur in the concrete due to improper vibration or placement during initial construction. The large and small aggregates separate from each other and create air voids in the concrete (Figure 16).

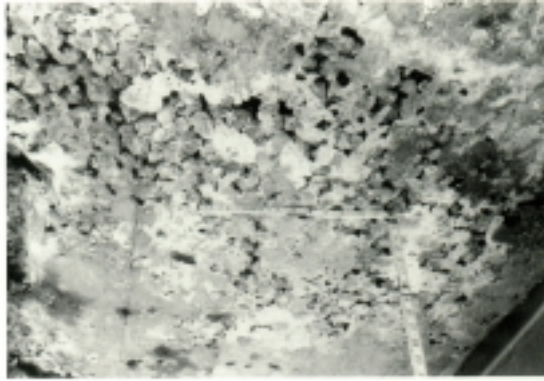


Figure 16 - Honeycombing

(Source: US DOT FHWA Bridge Inspector's Manual 90)

**7. Pop-outs** – Pop-outs are cone shaped fragments that break out of the surface of concrete. This type of defect can be caused by expansive aggregates, fire, sulfate attack, freeze-thaw, overload, or other means (Figures 17 to 20).



Figure 18 - Popout (Close-up)

(Source: ACI 201.1R-92)



Figure 17 - Small Popouts

(Source: ACI 201.1R-92)

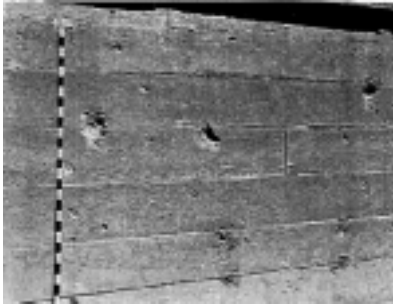


Figure 19 - Medium Popouts  
(Source: ACI 201.1R-92)

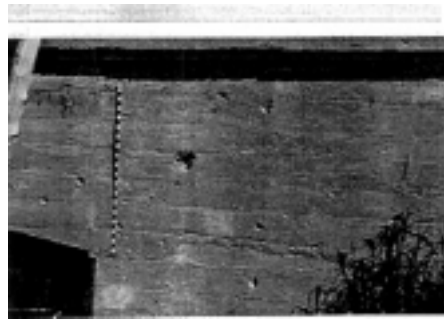


Figure 20 - Large Popouts  
(Source: ACI 201.1R-92)

**8. Rust Staining** – Rust staining is an indication of reinforcement corrosion. The rust particles leach out of the concrete by water flow. When this “rusty” water comes in contact with the atmosphere, the water evaporates, leaving the reddish rust stain on the surface.

**9. Scaling** – Scaling is the deterioration of the outermost layer of concrete, often due to freeze-thaw cycles. This is usually a gradual process in which the surface mortar and aggregate disintegrate over the life of the structure. Scaling can be classified by measurement of the depth of scale. (Figures 21 to 24).

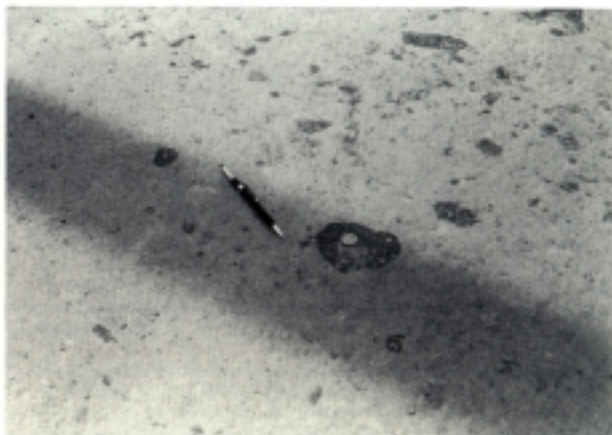


Figure 21 - Light Scaling (Loss of surface mortar up to  $\frac{1}{4}$ " )  
(Source: US DOT FHWA Safety Inspection of In-service Bridges)



Figure 22 - Medium Scaling (Loss of surface mortar  $\frac{1}{4}$ " to  $\frac{1}{2}$ " )  
(Source: US DOT FHWA Safety Inspection of In-service Bridges)

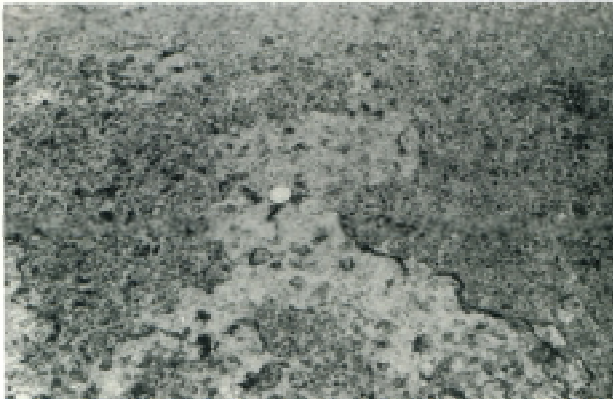


Figure 23 - Heavy Scaling (Loss of surface mortar ½" to 1")  
(Source: US DOT FHWA Safety Inspection of In-Service Bridges)

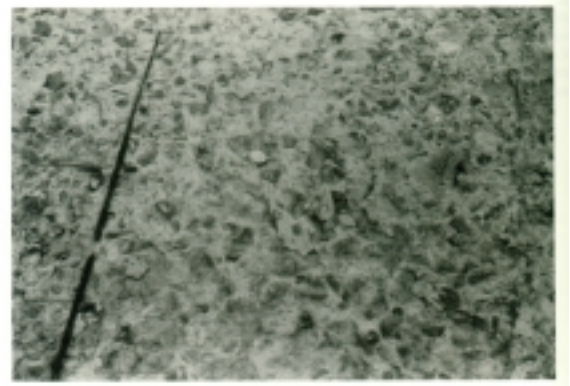


Figure 24 - Severe Scaling (Loss of surface mortar over 1")  
(Source: US DOT FHWA Safety Inspection of In-Service Bridges)

**10. Spalling** – Spalls are roughly circular or oval shape depressions in concrete. Spalls occur when the delamination finally separates from the member (Figure 25 and 26).



Figure 26 - Spall on a Reinforced Concrete Column

(Source: US DOT FHWA Bridge Inspector's Manual 90)

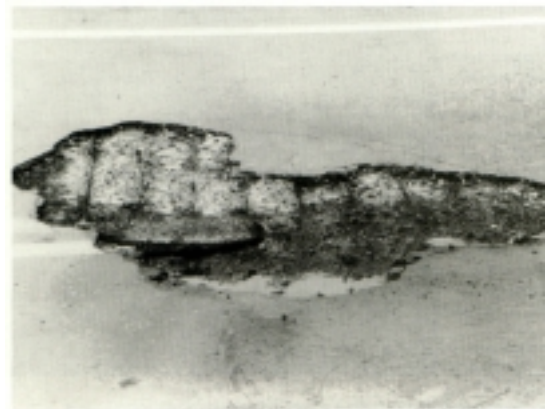


Figure 25 - Spall and Delamination on a Roadway  
(Source: US DOT FHWA Bridge Inspector's Manual 90)

**11. Wear** – Wear is the result of external frictional forces acting on the surface of the member. This can occur in many forms and in many different structures (Figure 27).

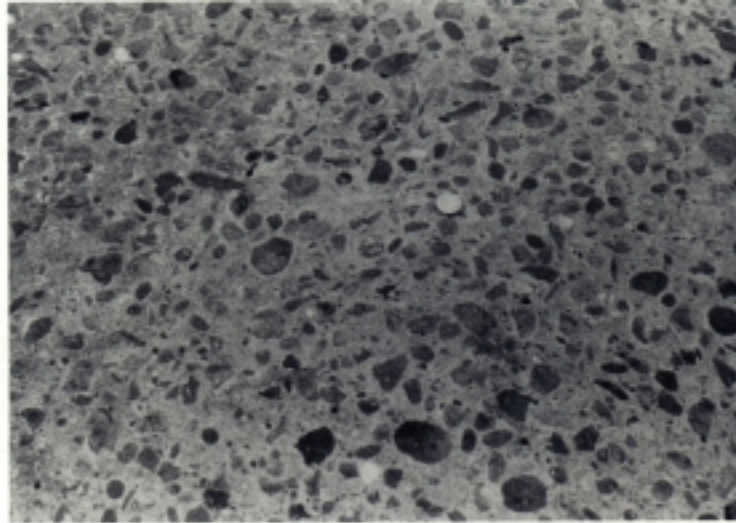


Figure 27 - Wear

(Source: US DOT FHWA Bridge Inspector's Manual 90)

## **B. Causes of Concrete Problems**

Causes of concrete problems may be singular occurrences that lead to other problems. Concrete problems or mechanisms can be divided into different classifications: 1) Early Age deterioration mechanisms, 2) Long Term deterioration mechanisms, and 3) In-Service deterioration mechanisms.

**1. Early Age Deterioration Mechanisms** – These are mechanisms that occur soon after concrete placement or construction. Though not likely to cause major problems alone, the defects they cause can increase susceptibility to an array of future deterioration mechanisms.

**a. Early Thermal Movement** – In concrete, the main reaction between cement and water causes a period of temperature increase. The expansion and contraction associated with the increased temperature and subsequent cooling causes cracking. Similar to the case of plastic shrinkage cracks, a restraint causes tension stresses and eventual cracking. The restraint occurs for several reasons: different cooling rates for different sides of a member, construction over a previously constructed base, or different cooling rates between the interior and exterior of a member <sup>(15)</sup>.

**Possible Defects:** Cracks occurring soon after concrete placement.

**b. Plastic Settlement Cracking** – Plastic settlement cracks result from the settlement of heavier solid components downward as the water and lightweight components leech upward toward the surface. Reinforcement plays a role in this type of crack development. Concrete above the reinforcement drapes itself over the bars; eventually leading to cracking which generally follows the outline of the reinforcement layout. The concrete below the reinforcement separates from the reinforcing bars and causes a void below. The surface profile looking in the axis of the reinforcement undulates over the bars. Cracks are about 1/16" to 1/8" (2-3mm) in width and taper to the level of the reinforcement when looking through the concrete section <sup>(16)</sup> (Figure 28). Downward movement of solids can also cause plastic settlement cracking at section changes where formwork restrains the concrete (Figures 29 and 30).

**Possible Defects:** Cracking will likely follow the general orientation of the underlying reinforcement. Cracking may occur at changes in section due to formwork restraint.

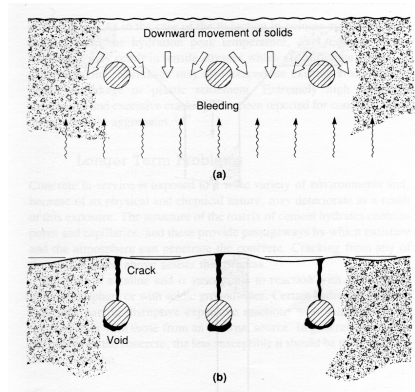


Figure 28 - Plastic Settlement Cracking

(Source: T. Kay, Assessment and Renovation of Concrete Structures)

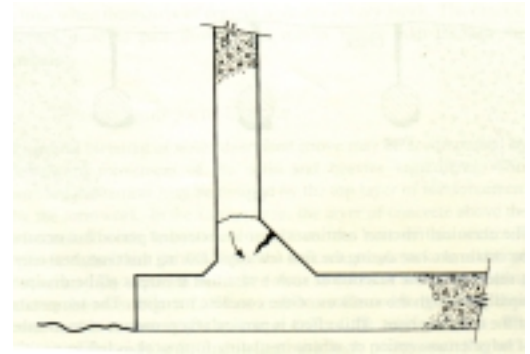


Figure 29 - Plastic Settlement Cracking due to Formwork Restraint

(Source: T. Kay, Assessment and Renovation of Concrete Structures)

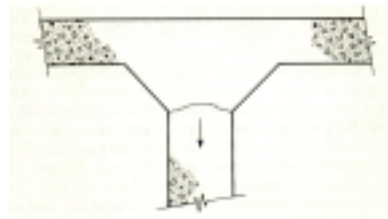


Figure 30 - Plastic Settlement Cracking due to Formwork Restraint

(Source: T. Kay, Assessment and Renovation of Concrete Structures)

**c. Plastic Shrinkage** – Plastic shrinkage cracks occur as the water is evaporating from the concrete surface. Usually, the evaporating water is replaced by bleed water traveling to the surface from lower depths of the concrete. When the surface moisture evaporates faster than the bleed water replenishing it, there is a reduction in volume, or shrinkage, at the surface causing tension cracks. These generally shallow cracks may follow the orientation of the top layer of reinforcing steel if the steel is close to the outside surface of the member. These cracks are usually 1/16" to 1/8" (2 to 3mm) wide, with width rapidly decreasing while moving deeper into the section<sup>(17)</sup> (Figure 31).

**Possible Defects:** Cracks occurring soon after placement. These cracks are typically wide, shallow, and generally isolated.

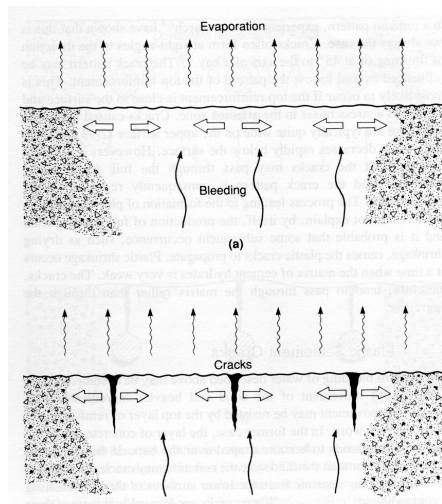


Figure 31 - Plastic Shrinkage Cracking

(Source: T. Kay, Assessment and Renovation of Concrete Structures)

**2. Long-Term Deterioration Mechanisms** – Concrete bridge structures are exposed to a variety of different environments in Wisconsin’s changing seasons. The long-term effects that follow are indicative of some of the possible mechanisms that may present themselves throughout the life of a structure.

**a. Alkali-Silica Reactivity** – Alkali elements are present in small quantities in cement in the form of potassium and sodium. Certain aggregates contain silica and are capable of chemically reacting with the alkali. The alkali content of cement depends on the source of the materials and the details of the manufacturing process. Alkali content in concrete mixes may also be better attributed to other cementitious materials that are sometimes added such as pulverized fly ash or ground granulated blast-furnace slag (GGBS). Three conditions must be present for alkali-silica reactivity to occur: a reactive aggregate must be present in sufficient quantities, the concrete mix must contain alkali metal ions, and sufficient water must be present to sustain the expansion reaction of the gel <sup>(18)</sup>. This type of deterioration mechanism may stabilize when one of the three conditions is not satisfied. When the reaction does take place, a gel is formed. This gel absorbs water and expands. As it expands, it exerts pressure on the surrounding concrete. The first signs of the mechanism are fine cracks radiating from a point. As time passes, the cracks propagate and join to form a pattern of cracking, mapping a large portion of the structure. The crack pattern may occur within three to five years of construction. Another indication of alkali-silica reactivity is the appearance of a white gel around a crack that can exude onto the surface of the concrete. Interaction with the atmosphere causes a chemical reaction that results in a white appearance; referred to as exudation. Crack patterns may be modified by the presence of external restraints and conventional prestressing or post-tensioning

reinforcement. The swelling due to the gel expansion may also cause pop-outs. The final cracks are generally 1"-2" deep (25-50mm) at right angles into the surface of the concrete <sup>(19)</sup> (Figure 32).

**Possible Defects** – Pattern cracking radiating from multiple points. Pattern cracking accompanied by white gel in cracks. Other indications are crazing, checking, pop-outs, and scaling.

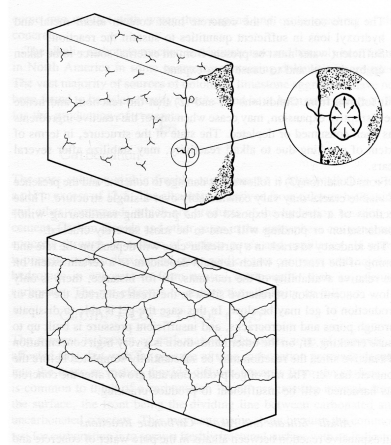


Figure 32 - Pattern Cracking due to an Alkali Reaction (Silica or Carbonate)

(Source: Kay, T. "Assessment & Renovation of Concrete Structures")

**b. Alkali-Carbonate Reactivity** – This mechanism is similar to Alkali-Silica reactivity, however the constituents are different. The most significant difference being the lack of white gel that occurs. Typical limestone contains calcium carbonate. Some dolomitic limestones used in aggregates also contain magnesium carbonate. The magnesium carbonate reacts with alkali metals to form calcium carbonates, magnesium oxide, and alkali carbonates. This reaction exposes clay particles within the dolomite crystals. When these clay particles interact with water, they expand and cause cracking. Pattern cracking radiates from these points. The cracking at various points then radiate with time and eventually interconnect (Figure 32).

**Possible Defects** – Pattern cracking radiating from multiple points *lacking* any traces of white gel in cracks. Other indicators are pop-outs, scaling, crazing, and checking

**c. Carbonation** – Concrete provides a protective alkaline environment for the embedded reinforcement. This protective environment has a pH of about 12.5. Gases, such as carbon dioxide, can penetrate into the concrete eliminating the protective alkaline environment and increasing susceptibility to corrosion of

reinforcement. The rate of deterioration due to carbonation is dependant upon the quality of the concrete. A higher quality concrete, one with less porosity, high cement content, and low water to cement ratio, will have a greater resiliency to carbonation. The depth of clear cover between the outside surface of the concrete and the reinforcement can also dictate the susceptibility to corrosion with respect to carbonation. The rate of carbonation is also dependant upon the ambient relative humidity and the degree of saturation of the concrete. If the concrete is blocked by water, the carbon dioxide has no opportunity to penetrate the concrete pores. Carbonation cannot take place in completely dry concrete since the reaction requires the presence of water to propagate. The depth of carbonation will be greater in areas containing cracks, which offer an easy means of carbon dioxide penetration. The depth of the carbonation is usually referred to as the carbonation “front” as it progresses into the concrete (Figure 33). The depth of the carbonation is approximately proportional to the square root of the time of exposure. If the time of exposure increases by a factor of four, the depth of the carbonation front doubles<sup>(20)</sup>.

**Possible Defects** – Corrosion of reinforcement due to loss of alkaline environment.

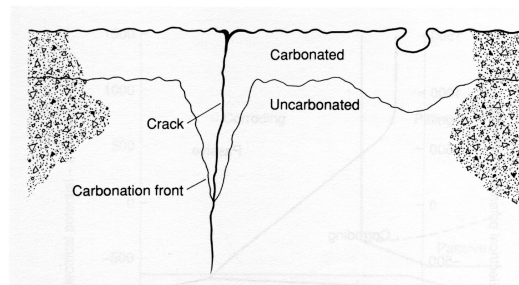


Figure 33 - Carbonation depth near a crack

(Source: Kay, T. "Assessment & Renovation of Concrete Structures")

**d. Corrosion of Reinforcement** – The alkaline environment of concrete protects the reinforcing steel from corroding. High quality concrete—less porous, high cement content, low water to cement ratio and sufficient clear cover—provides a barrier from air and water penetration, both of which are required for steel corrosion. Chemical salts can also destroy the protective environment. These salts are commonly chlorides. Chlorides can be introduced into the concrete as additives to reduce setting time and to allow continued concrete placement during cold weather. Lastly, chlorides are

introduced into concrete through the use of de-icing salts on Wisconsin roads. Areas exposed to de-icing salt solution spray, such as columns, piers, and abutments are also susceptible to chloride intrusion. The expansion of corroded steel introduces localized pressure points in the concrete. This pressure causes the concrete to crack, when these cracks interact and reach the outer surfaces, delaminations and spalls result (Figure 34).

**Possible Defects** – Cracking primarily in the plane of reinforcement layout. Rust staining may be present. Delaminations and spalls may occur. Efflorescence may be present near cracks. Circular or elliptical cracks (ellipse may be elongated in the direction of the reinforcement).

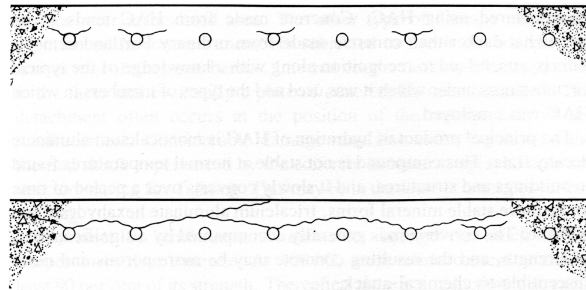


Figure 34 - Hypothetical Delaminations through reinforced concrete cross-section due to expansion during reinforcement corrosion

(Source: Kay, T. "Assessment and Renovation of Concrete Structures")

**e. Drying Shrinkage** – As time passes, moisture in concrete evaporates into the atmosphere. The loss of moisture is accompanied by a volume reduction. The greater the amount of moisture initially present, the more the volume reduction. Any restraint in this volume reduction could result in significant tensile stresses in the concrete and subsequent cracking<sup>(21)</sup>.

**Possible Defects** – Linear cracking perpendicular to the restraining force. Orthogonal pattern cracking is typical of drying shrinkage cracking. Cracks are typically fine in width and shallow in depth.

**f. Expansion/Contraction** – Concrete expands and contracts as the temperature rises and falls. Expansion and contraction is acceptable, providing movement is not restrained. There are several factors that can restrain movement, these include: inoperative bearing devices and clogged, non-operational expansion joints. Friction can also act as a restraint to concrete movement. Expansion of aggregates in the concrete mix can introduce pressure on the surrounding concrete and cause cracking and pop-outs.

**Possible Defects** – Cracking mostly linear in nature, occurring perpendicular to the restraining force. Checking and crazing may also occur depending upon the nature and orientation of the restraint.

**g. Freeze-Thaw Cycles** – The near surface pores of a concrete structure can become saturated through the course of the seasons. If these water-soaked pores are exposed to freezing temperatures, ice crystals can develop in the pores. As crystals grow within the pores, they exert pressure on the surrounding concrete. The expansion of these crystals can cause cracking throughout the surface exposed to weathering. The cracking eventually leads to deterioration and spall of the topmost layer of concrete. The subsequent thaw facilitates water travel deeper into the concrete, followed by a subsequent freezing and again further deterioration deeper into the concrete. Horizontal surfaces and joints are particularly susceptible to this deterioration <sup>(22)</sup>.

**Possible Defects** – D-cracking near joints and large cracks. Scaling, spalling, delaminations, crazing, and checking are possible. Pop-outs are also possible when aggregates are more expansive than the cement paste (Figure 35).



Figure 35 - Pop-outs due to expansive aggregates

(Source: Kay, T. "Assessment and Renovation of Concrete Structures")

**h. Sulfate Attack** – Soils, rocks, aggregates, and groundwater may contain sulfates that can attack concrete. Gypsum, which is calcium sulfate, is present in some clay soils. If gypsum is present in the concrete, the cement in the mix can react with water to form ettringite. Ettringite occupies a larger volume than the components that react to form it. The expansion of the ettringite causes tensile stresses in the cement paste and introduces pressure on the surrounding concrete. Cracking can then occur throughout the concrete <sup>(23)</sup>.

**Possible Defects** – Scaling, checking, and crazing, accompanied by disintegration of cement paste.

**3. In-Service Deterioration Mechanisms** – These deterioration mechanisms are encountered during the in-service life and are generally unavoidable but can be minimized. The following are a few examples of possible defects that can occur in concrete structures due to improper use.

**a. Chloride Intrusion** – Chloride intrusion is the presence of re-crystallized soluble salts, commonly caused by road deicing salts. These salts "intrude" into areas at or near the reinforcement level,

and interact with the reinforcing steel to cause corrosion and cracking of the concrete.

**Possible Defects** – Corrosion of reinforcing steel and cracking of concrete. Efflorescence may be present in and around any cracks.

**b. Defective Expansion Joints** – The most common areas to undergo damage due to deicing salt exposure are regions at expansion joints. Expansion joints are typically ineffective in a harsh environment and are damaged due to automobile traffic wear and snowplow damage. The deicing salt exposure can cause corrosion of steel and eventually lead to cracking in the member.

**Possible Defects** – Corrosion of reinforcing steel and cracking of concrete.

**c. Fire Damage** – Extreme heat will damage concrete. Cement paste is weakened by extreme temperatures and can cause cracking. Extreme heat can cause the reinforcing steel to relax, increasing deformations and possibly causing cracking. Fire damaged concrete can have a variety of defects depending on exposure time and concrete moisture content <sup>(24)</sup>. Explosive spalling may occur during early stages of the fire, progressively leading to the removal of sections of the concrete, likely to the level of the outer layer of reinforcing steel. Columns and beams may experience internal cracking eventually rounding the outer edges of the member. The sudden application of water to fire-heated concrete can also lead to cracking and spalling. Prestressed concrete members can lose a significant portion of their strength when exposed to fire due to the reduction in steel modulus and elongation of prestressing strands.

**Possible Defects** – Pattern cracking, craze cracking, checking, scaling, spalling, rounding of concrete member edges due to spalling, possible regions of soot-coated concrete, flexure cracking and excessive deformation due to elongation of steel reinforcement.

**d. Flexure Cracks** – Flexural cracks are generally perpendicular to the longitudinal axis of the member. These cracks occur in the tension region due to excessive flexural load or lack of adequate reinforcing steel (Figure 36).

**Possible Defects** – Linear cracks mostly parallel to the direction of applied load in moment carrying members at midspan in simply supported structures, near midspan and above continuous supports in continuous members.

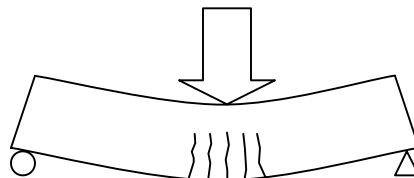


Figure 36 - Flexure Cracking

**e. Foundation movements** – Foundation movements introduce unintended loads into bridge structures. These unintended loads can cause a variety of defects throughout the structure depending on the extent of the loading.

**Possible Defects** – Large cracks spreading from points of bearing load, crushing, spalls, or delaminations near the moving foundation.

**f. Impact Loads/Collision** – All areas exposed to possible damage from trucks, ships, and others should be monitored for impact damage. Loss of section, particularly in prestressed or post-tensioned members, can greatly reduce load carrying capacity and lead to structural failure.

**Possible Defects** – Physical damage to members and excessive deformation.

**g. Overload** – Overloads can occur during in-service lives of bridge structures and can cause significant damage including excessive deformation, cracking and structural failure.

**Possible Defects** – Excessive deformation, vertical cracking, and spalls near midspan and over continuous supports.

**h. Shear Cracks** – Shear cracks exist diagonal to the direction of loading and occur typically near the supports. These cracks are formed due to excessive load or lack of adequate shear reinforcing steel. Cracking will likely occur from the bottom of the section if the section is rectangular (Figure 37). If the section is an I-shape, shear cracks may exist in the web of a member.

**Possible Defects** – Diagonal cracks at areas of high shear at simple and continuous supports.

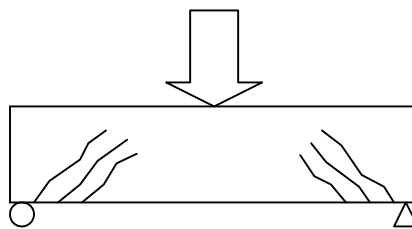


Figure 37 - Shear Cracking

**i. Wear/Abrasion** – Snow plows, sweepers, and auto and truck tires (with and without chains) can cause wear damage over the lifespan of a bridge structure.

**Possible Defects** – Loss of the top surface of concrete that will expose aggregates to other mechanisms such as freeze-thaw and increased carbonation depth.

### III. Steel Defects and Causes

Steel has high strength, elasticity, shock, and creep resistance, and is present in nearly all forms of infrastructure. Unlike concrete, steel is strong in tension and is available in several shapes for building materials: wires, cables, plates, bars, and rolled sections. Steel is elastic and generally resistant to impact load. Un-cracked or un-damaged steel typically undergoes a period of elongation before fracture if statically loaded. A major problem concerning steel is fatigue cracking. Fatigue cracking can lead to sudden fracture, which could be catastrophic in nature. Corrosion can also present problems in steel bridge structures. If care is not taken, corrosion can lead to significant loss of cross-section in a steel member. It can also introduce stress raisers in areas that may lead to fatigue or other types of cracking. Vehicular impact damage to bridge members is also possible. The following discusses typical visual indications encountered during visual inspections in steel bridge structures:

#### A. Steel Problems/Visual Indications

Primary problems encountered in steel structures include: corrosion, cracking, and various physical damage. Steel and concrete are similar in that most of their deterioration mechanisms typically occur over time, with use and exposure to the environment.

**1. Corrosion** – Corrosion is the result of iron particle oxidation. It can lead to loss of section and reduction of the overall load carrying capacity. Corrosion is an electrochemical process in which current or electrons flow between an anode (an area that has a high tendency to corrode) and a cathode (an area that has a lower tendency to corrode). For corrosion to occur three factors must exist: an electrolyte (typically water in bridge structures) for electrical conductivity on the steel surfaces; oxygen to proceed the reaction; and an anode region on one metallic surface in relation to a second cathode area <sup>(25)</sup>. The process of electron and ion flow produces expansive corrosion products, specifically rust. In addition to section loss, corrosion can cause restraint of movement, as is the case with bearings and joints. The pin connections of bearing devices may corrode and therefore restrict rotation of the pin connection causing unintended overload in the structure. Areas of corrosion can also create stress raisers, leading to stress corrosion cracking or sources of fatigue crack initiation.

**2. Cracking** – A major cause of cracking in steel bridge structures is fatigue, which is due to cyclic loading. Cyclic loading in bridge structures occurs through vehicular traffic loads and changes in temperature. Fatigue is dependent upon three basic factors: tensile stress range, number of cycles, and geometry of the member. Fatigue crack growth occurs over three phases. The first phase is the development of the initial flaw. This occurs at a microscopic level at material discontinuities; commonly weld defects and other stress raisers. The second phase is a

period of stable crack growth; leading to a possible fracture. The final phase is a period of rapid, unstable crack growth ending with a brittle sudden fracture. Once a fatigue crack is visible to the naked eye, the majority of the fatigue life of the member has been depleted. This represents the fatigue life during the first two phases of crack growth. Certain welded steel details are fatigue prone. Out-of-plane distortions can occur in some details. These details may also introduce stress concentrations into certain bridge detail areas, either by material geometry or load flow.

**3. Brittle Failure** - Intersecting welds can produce a dangerous condition of possible brittle failure. For example, in the Hoan Bridge failure, intersecting welds in connection areas produced highly constrained regions where, with the introduction of forces from several directions, resulted in high tri-axial stresses. These highly stressed details abruptly failed in a brittle nature without noticeable cracking before failure <sup>(26)</sup>. Brittle failures also may result due to cold temperatures and inadequate fracture toughness in the steel.

**4. Fire Damage** – Fire damage can cause significant reductions in the modulus and strength of steel members. Fire damage may be due to changes in the molecular structure or excessive deformation. The extent of damage varies depending upon the level of the temperature and the amount of exposure time. The ultimate failure temperature of steel can vary depending upon a variety of factors: insulation provided by surrounding materials, restraint provided by connections, or magnitude of loads <sup>(27)</sup>. Fire damage may be recognizable by the soot in the surrounding areas and the deformation of the steel section.

**5. Impact Damage** – Impact loads on steel structures could lead to significant reduction or loss of load-carrying capacity and to failure. Areas prone to damage from vehicular or ship traffic need to be closely monitored. Large deformations or ruptures caused by the impact can change the structural characteristics and behavior of bridge members.

## **B. Causes of Steel Problems**

Problems and defects in steel structures may be due to one or more mechanisms. The following presents insight into the causes of different defects in steel structures and offers visual illustrations:

**1. Corrosive Environments/Corrosion Prone Details** – Under certain conditions, corrosion can typically form in sheltered areas in steel structures. Geometry, environmental debris, and even bird droppings can provide the sheltering effect on a portion of a steel surface. Sheltered areas create a zone with different characteristics than the remaining steel surface. The differences provide the anode and cathode zones to start

the corrosion process. Another possible corrosion process involves direct contact of dissimilar metals. Dissimilar metals provide the anode and cathode zones directly, no sheltering is needed to advance the corrosion process. The final, and least likely, form of corrosion is that involving water or fluid flow. The flow exposes fresh surfaces to corrosion, speeding the corrosion process <sup>(28)</sup>. The following discusses the various common corrosion types and describes possible resulting defects.

**a. Crevice Corrosion** – Crevice corrosion is a common type corrosion seen in steel bridges. It occurs in confined areas where there are various layers of steel in contact through bolts, rivets, or partial welds. These confined regions have higher concentrations of moisture, chloride ions, oxygen and other metal ions that facilitate corrosion. <sup>(29)</sup> (Figures 38 to 40).

**Possible Defects** – Corrosion at confined areas such as lateral bracing and diaphragm connections, pin and link bearing connections, expansion bearings, etc.



Figure 38 - Pressure due to Crevice Corrosion, Bending Steel at Truss Connection

(Source: NCHRP Report #333)



Figure 39 - Crevice Corrosion Between Concrete and Steel

(Source: NCHRP Report #333)



Figure 40 - Crevice Corrosion

(Source: NCHRP Report #333)

**b. Deposit Attack** – Corrosion can occur in steel bridge members in a manner similar to the crevice corrosion. However, deposit from foreign materials can contribute to the forming of the confined area, which can contain moisture and aggressive chemicals. These deposits can range from road salts to bird droppings to grain in farming regions <sup>(30)</sup>. The deposits may contain aggressive chemicals that can initiate or accelerate corrosion of steel (Figure 41).

**Possible Defects** – Corrosion and possible loss of section in confined areas such as exposed horizontal flange areas of beam sections, regions near connections, etc. Those regions exposed to deicing salt intrusion and spray may be more susceptible to deposit attack. Weathering steel is especially susceptible to deposit attack.



Figure 41 - Deposit Attack  
(Source: NCHRP Report #333)

**c. Erosion Corrosion** – Erosion corrosion occurs as a flow of fluid constantly overpasses a steel area, continually exposing fresh steel surfaces to further corrosion <sup>(31)</sup> (Figure 42).

**Possible Defects** – Corrosion and loss of section at members near waterways and other fluid flow. Areas such as steel grid decks, typically used on lift bridges, are possible areas of erosion corrosion.



Figure 42 - Deterioration of Steel Pier due to Erosion Corrosion  
(Source: NCHRP Report #333)

**d. Fretting Corrosion** – The motion of two non-lubricated or poorly lubricated surfaces against each other under load causes fretting that may facilitate corrosion. The abrasion breaks the existing steel surface oxidation. Once broken, the oxidation reforms, causing abrasion of the surfaces by the oxides and debris. This is generally not identifiable with the naked eye<sup>(32)</sup> (Figure 43).  
**Possible Defects** – This is possible in structures that use lubricated (or non-lubricated) bearing plates for bearing devices, or in pin and hanger connections.



Figure 43 - Possible Fretting Corrosion Area  
(Source: NCHRP Report #333)

**e. Galvanic Corrosion** – Galvanic corrosion occurs when two dissimilar metals come in contact with each other in the presence of an electrolyte (water). The differing electrical potential of the two metals produces electron flow. This often occurs in steel bridges with light poles, handrails, and electric conduits. This is also possible in areas where galvanized steel is in contact with non-galvanized steel. Galvanic corrosion may also occur in areas where mill scale is exposed <sup>(33)</sup> (Figure 44).

**Possible Defects** – Corrosion near attachments of dissimilar metals.



Figure 44 - Galvanic Corrosion

(Source: NCHRP Report #333)

**f. Pitting Corrosion** – Pitting is a local corrosion that can cause deep, narrow penetrations into the steel surface. Pits act as stress raisers and can lead to fatigue cracking. Exposure to deicing salt spray can lead to extensive pitting <sup>(34)</sup> (Figure 45).

**Possible Defects** – Pitting in regions exposed to deicing salt flow and spray.



Figure 45 - Pitting Corrosion

(Source: NCHRP Report #333)

**g. Stress Corrosion Cracking** – Stress corrosion cracking occurs in steel when tensile stresses occur in a corrosive environment. The corrosion causes an initial discontinuity that causes a stress raiser leading to developing cracks<sup>(35)</sup>. This is not identifiable by the naked eye (Figures 46 and 47).

**Possible Defects** – Bridge wires and strands are possible areas of stress corrosion cracking. Highly stressed bolts that are subjected to harsh corrosive environments may also experience stress corrosion cracking.



Figure 46 - Stress Corrosion Cracking  
(Source: NCHRP Report #333)



Figure 47 - Stress Corrosion Cracking of Bolts  
(Source: NCHRP Report #333)

**h. Underfilm Corrosion** – Underfilm corrosion is a crevice-type corrosion that occurs beneath paint. This occurs at defective or damaged areas in the paint covering a steel member, causing the paint to lose its bond from the steel surface. Cracking, peeling, or blistering paint film is an indication of possible corroded areas or corrosion prone areas<sup>(36)</sup> (Figure 48).

**Possible Defects** – Rust staining through paint cracks should be an indication of underfilm corrosion. Bulged regions below paint are also an indication of underfilm corrosion.

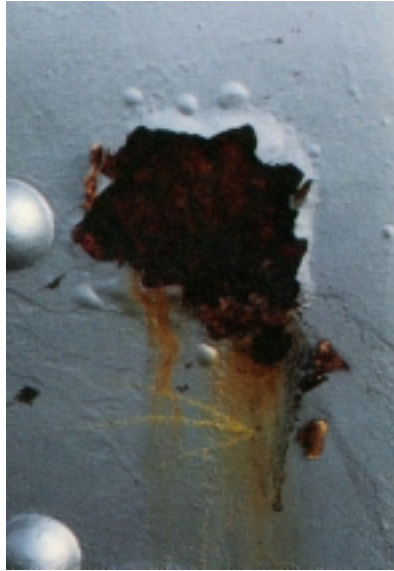


Figure 48 - Underfilm Corrosion

(Source: NCHRP Report #333)

**i. Weld Decay** – Weld decay is a local corrosion deterioration due to a decrease in the corrosion resistance of either the weld or base metal. The decrease in corrosion resistance is due to changes in the granular structure of the steel in the heat-affected zone. The corrosion typically occurs as a band parallel to the weld along the heat-affected zone <sup>(37)</sup> (Figures 49 and 50).

**Possible Defects** – Corrosion in the heat-affected zone of welded joints. Members such as box and plate girders with extensive weld should be monitored for corrosion in the heat-affected zone.



Figure 49 - Weld Decay

(Source: NCHRP Report #333)

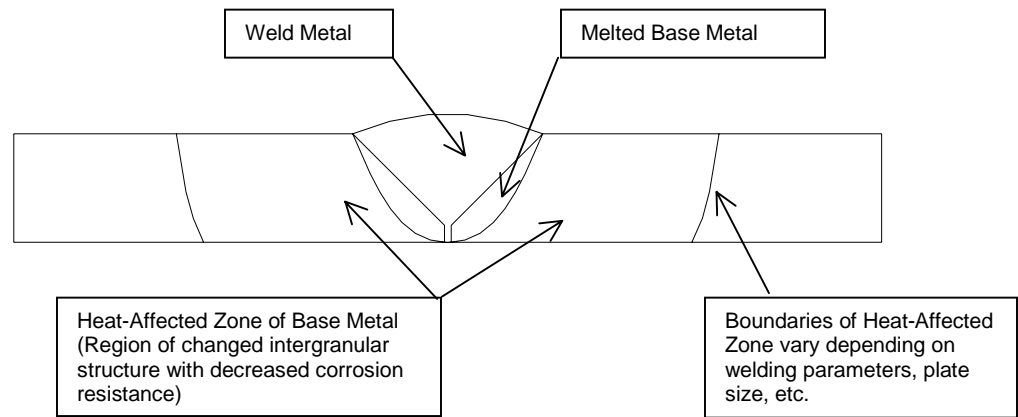


Figure 50 - Typical Zones Resulting from Welding  
(Butt joint example, general scenario can occur regardless of the orientation)

**2. Other Environmental Causes of Corrosion** – The following details some of the environmental factors that increase corrosion likelihood.

**a. Deicing Salts** – Water saturated with salt ions forms an electrolyte that can accelerate the corrosion reaction. Members with exposure to deicing salt spray or deicing salt runoff should be monitored for corrosion.

**Possible Defects** – Pitting corrosion, crevice corrosion, underfilm corrosion, and other corrosion type.

**b. Stray or Impressed Electrical currents** – Stray electrical currents can cause corrosion due to the formation of differing electrical potential zones on the steel surface. These areas are the anode and cathode. The presence of two separate zones of differing characteristics and the presence of electrolytes greatly increases the likelihood of corrosion. This can be caused when circuits that carry large currents are grounded through the steel member <sup>(38)</sup>.

**Possible Defects** – Corrosion near grounding regions of electrical devices.

**3. Fatigue Cracking/Out-of-Plane Distortion & Improper Details** –

Bridges are subjected to cyclic or fatigue loading in the form of vehicular traffic or from the effect of changes in temperature. Additional stresses develop in steel structures through out-of-plane distortion of thin-walled members at connections. Fatigue cracking typically occurs in areas where transverse connection plates have been welded to beam webs. Also, intersecting welds in connection areas can cause fatigue cracking. In an area with intersecting welds, the load flow through a connection is through a single point creating a stress concentration. Many details remain in service because their vulnerability to this type of problem was not known at the time of design and construction. Welding connection areas are typical places to check for fatigue prone details. The following offers insight into the fatigue prone details in steel bridge structures:

**a. Transverse Connection Areas/Web Gap Distortion** –

Transverse stiffeners provide added strength in shear for girders (Figure 51). Transverse connection plates on girders are used to support loads from the end of a member framed in the girder. The distinction between transverse stiffeners and transverse connection plates is often difficult <sup>(39)</sup>. The two appear very similar. The confusion between the two has led to improper details in some of today's structures. A few examples of proper and improper transverse stiffener and connection plate details are shown in Figures 51 to 53. Web gap distortion typically occurs when transverse connection plates are utilized in conjunction with a floor system containing stringers and floor beams that are framed into girders (Figure 53).

**Possible Defects** – Cracking in the web between the ends of the connection plate-to-web weld and the top and bottom flanges. These cracks are more common when the ends of the connection plate are not welded to the top and bottom flanges of the supporting girder. Typically, horseshoe shaped cracking can be seen around the ends of the plate-to-web weld. Cracking can also be seen in the heat-affected zone region of the flange-to-web weld in the region of the transverse connection. Distortion is likely at the web gap.

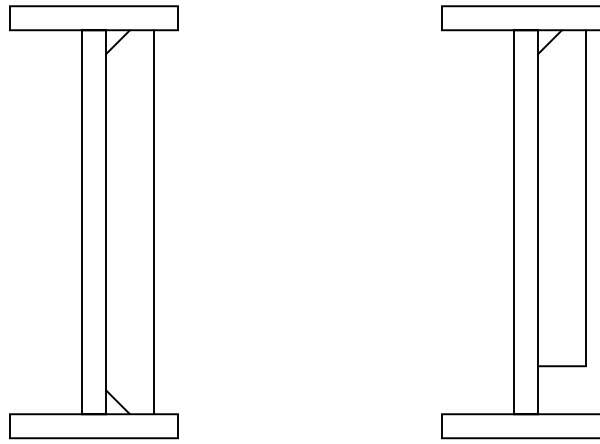


Figure 51 - Examples of Transverse Stiffeners details

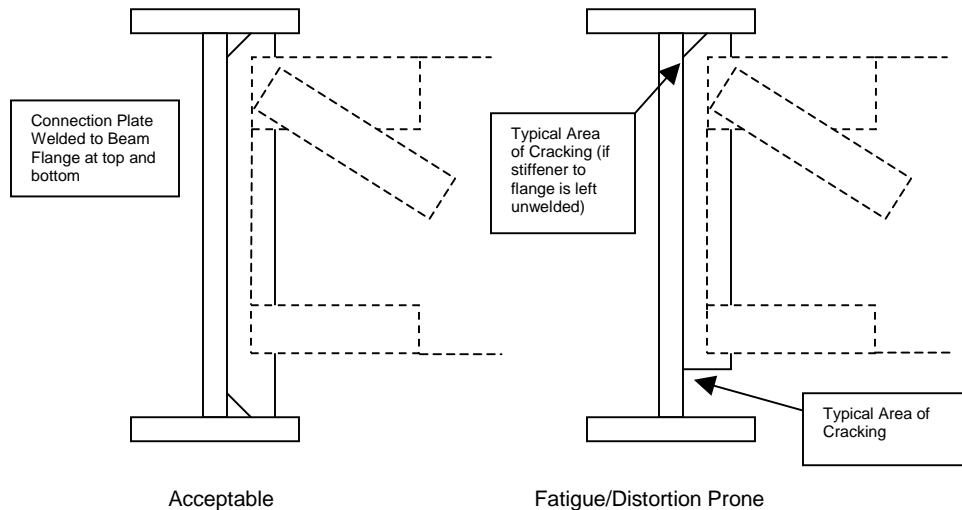


Figure 52 - Examples of Transverse Connection details

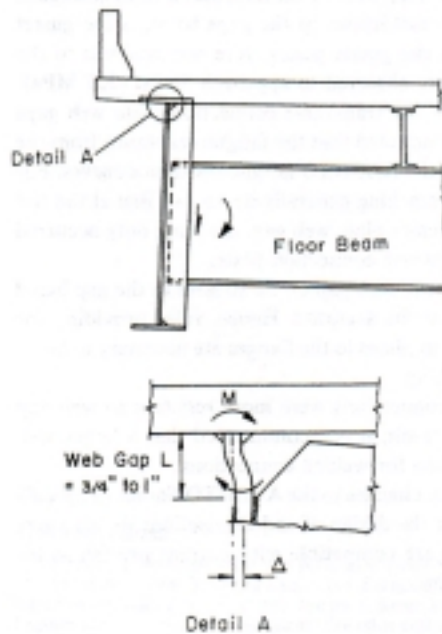


Figure 53 - Example of Out-of-plane distortion in web gap at end of floor beam transverse connection plate

(Source: NCHRP Report #336)

**b. Longitudinal Web Stiffeners** – Longitudinal stiffeners are not typical in new construction, but may be present in existing structures. These were often added to prevent web buckling in a beam section. Longitudinal web stiffeners have problems in the fact that the ground groove weld of the stiffener itself can provide a means of crack initiation into the beam/girder web; an example of intersecting welds. Cracks can originate at a splice in the stiffener plate and easily travel to the beam web (Figure 54).

**Possible Defects** – Vertical cracking in the web originating at the longitudinal stiffener-web weld.

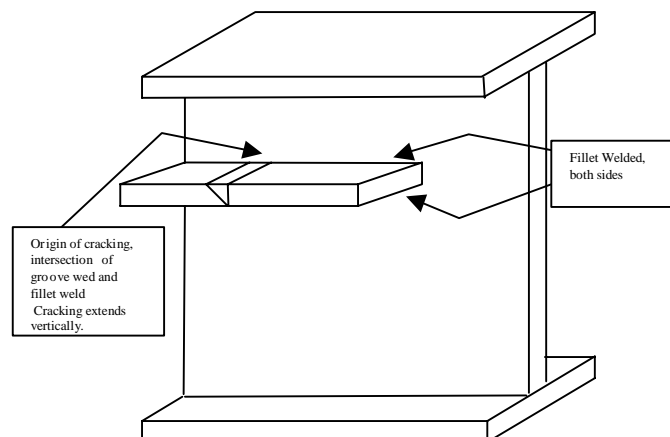


Figure 54 - Cracking Mechanism in Longitudinal Web Stiffeners

**c. Lateral Gusset Plate Connections** – Lateral gusset plates are sometimes used and are notched around transverse stiffeners. These details are similar to longitudinal web stiffeners or connection plates in that they are longitudinally placed elements that connect to the web. The occurrence of intersecting groove welds and longitudinal welds on a lateral gusset plate is unlikely, but distortion of the web in these connections causes fatigue cracking (Figure 55). The distortion occurs because of the load flow into the web, an inherently weak member when loaded out of its plane.

**Possible Defects** – Vertical cracking is possible near the heat-affected zone parallel to the transverse stiffener in the connection zone.

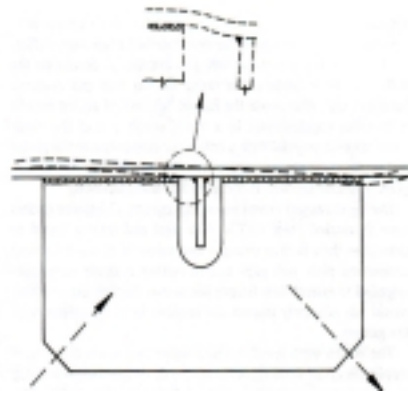


Figure 55 - Example of Out-of-plane distortion in web gap at lateral gusset plate

(Source: NCHRP Report #336)

**d. Cover Plates** – In many existing steel bridges, a cover plate may have been welded to the flange of a rolled I-section to increase the structural capacity. While this increased the moment capacity of the section, it also introduced stress concentrations in the area of highest stress, due to bending moment, in a beam and at the ends of the cover plate weld. Cracks often developed along the toe of the cover plate-to-flange weld and at the ends of the cover plate (Figures 56). Care must be taken in detection of these cracks as they are generally formed due to fatigue. These cracks are specified as categories D, E, E' and F by AASHTO.

**Possible Defects** – Cracking in the flange perpendicular to the longitudinal direction and at the ends of the cover plate welds. The cracking initiates at the weld and associated heat-affected zone.



Figure 56

Cracking at Coverplate-to-Flange Weld  
(Source: NCHRP Report #206)

**e. Intersecting Welds** – Intersecting welds introduce a multi-axial state of stress in a steel member. This, coupled with the possibility of inherent weld impurity and flaws, makes intersecting welds a typical area for cracking. The presence of loading from multiple directions introduces a triaxial stress state at the weld intersection that may rupture dramatically as was the case with the Hoan Bridge <sup>(26)</sup>.

**Possible Defects** – Cracking may occur in a variety of directions at the connections. The condition of intersecting weld should be documented and corrective measures should be taken to relieve excessive stresses.

**4. Physical Damage** – Physical damage can occur in many forms and have varying effects on steel. Load carrying capacity can be decreased depending on the extent of damage. The following is a sample of the possible damage phenomena.

**a. Normal Temperature Changes** – Steel expands and contracts with changing temperature. Resistance to this action may cause excessive stresses to be introduced into the member; causing cracks and excessive deformation.

**Possible Defects** – Distortion is possible due to differential temperature changes in bridge members. Thermally introduced loads further magnify the stress conditions in fatigue-prone areas.

**b. Fire** - Extreme heat will cause a reduction in the modulus and will cause the material to deform plastically. Elongation, buckling,

and twisting may occur due to extreme heat. Rivets and bolts may fail at connections due to extreme heat.

**Possible Defects** – Excessive deformation, elongation, buckling, and twisting of the section.

**c. Impact Loads/Collision** – Impact damage can occur on virtually any area of a structure exposed to vehicular or ship traffic. Members can be ruptured or distorted out of their original plane causing structural collapse.

**Possible Defects** – Distortion out of the plane of the member. Section loss due to rupture is possible, as are broken welds.

**d. Overloading** –Overloading can cause plastic deformation. Plastic deformation can occur in both compression and tension members. Tension members will lose cross-section and elongate. Compression members will likely bow, or buckle, in some fashion, either in double or single curvature.

**Possible Defects** – Permanent deformation, buckling of members.

#### **IV. Non-Destructive Evaluation of Concrete Materials and Structures**

A number of damage or deterioration types could affect adversely the performance of concrete structures. These include cracking, delamination, corrosion of reinforcing elements, concrete deterioration due to poor quality and/or the presence of aggressive environment, and others such as fire, impact loads, earthquake, etc. These can reduce the safe load carrying capability of the affected structures and lead to failures.

Descriptions of possible flaws in concrete materials and structures as well as appropriate NDE techniques for their detection and assessment are tabulated and presented in Appendix A. Summary discussions and appropriate references for various related NDE test methods are presented in Appendix C. The following presents appropriate test techniques that may be used to evaluate the condition of concrete bridge decks and super structures. For each test technique, a brief description, applicability, advantages, and disadvantages are given. It must be noted, however, that although categorized as non-destructive test methods, some techniques are inherently intrusive and therefore require subsequent repair after the completion of the test. The importance of an effective visual inspection by an experienced inspector cannot be understated, as it will generally reveal indications on the type of defects present. Visual inspection should always be incorporated into any bridge evaluation and condition assessment program.

Numerous laboratory and field evaluations were made as a part of this and other similar studies by the research team to obtain verifications of the characteristic features of several NDE test methods included in this report. These methods include acoustic emission, ultrasonic, ultrasonic imaging, pulse velocity, magnetic particles, dye penetrant, impact-echo, magnetic flux leakage, sounding, time domain reflectometry, Chloride content measurements, and half-cell potential. These NDE methods were studied in the various laboratories of the University of Wisconsin-Milwaukee. They were also evaluated at the sites of several bridge structures in Wisconsin and other states. Other NDE methods that are listed in this report were evaluated based on studying the available literature and using the past experience of the research team.

##### **A. Concrete Bridge Decks**

The most common defects in concrete bridge decks include delamination, cracking, corrosion of reinforcing or prestressing steel, honeycombing, excessive permeability to aggressive chemicals, and overall poor material quality. To detect and assess the extent of each defect, one or more appropriate test techniques should be identified and used. The following test techniques are appropriate for use in concrete bridge decks with specific defects or flaws.

**A.1. Delamination, Honeycombing, Voids** – The following test methods are suitable for detecting and evaluating the extent of delamination,

honeycombing, and voids in concrete decks. Some of these methods may also be used to determine the thickness of the concrete deck.

**A.1.1. Chain Dragging** – Chain dragging is a qualitative, simple and inexpensive means of determining general areas of delaminated concrete or overlay materials. Regions of delamination will have a hollow sound compared to a solid sound for satisfactory concrete.

**Applicability:** Chain dragging is capable of determining the general areas of delaminated concrete or overlay. Further testing will be necessary to determine the extent of reinforcement corrosion and accurate location of the delamination. An additional test often performed is the impact echo-technique.

**Advantages:** Simple, inexpensive, and quick.

**Disadvantages:** It is a qualitative test method that relies on the experience and tuned ear of the inspector using the chain drag device. Only gives a general assessment of damage, without indicating exact locations.

**A.1.2. Hammer Sounding** - Hammer sounding is a simple technique that can yield information regarding delaminated areas of concrete. Regions of delamination have a hollow sound compared to a solid sound for satisfactory concrete when struck with a hammer.

**Applicability:** Effective means of determining localized delaminations in concrete elements. It is a possible follow-up method to chain dragging test.

**Advantages:** Simple, inexpensive test.

**Disadvantages:** Relies on the experience and ear of the inspector. It is a qualitative test.

**A.1.3. Ground Penetrating Radar (GPR)** – GPR can provide an indication of the overall condition of the cross-section of the bridge deck. Depending on the moisture content in the bridge deck, the GPR test may yield information on delamination extent.

**Applicability:** Capable of determining the location of embedded materials, delamination, and voids. Specialized vehicles available for high-speed data collection along bridge decks.

**Advantages:** High speed data collection and examination.

**Disadvantages:** Relatively high cost and it requires experience with interpretation of results.

**A.1.4. Infrared Thermography (IT)** – Based on the variation in conductivity, voids and delaminations in bridge decks are identified using the IT technique. This method relies on the reactions of the bridge deck to heat. Due to the heat requirement, this method is ideally suited for sunrise or sunset when the changes in temperature are greatest in the bridge deck

**Applicability:** Capable of determining the location of delaminations, voids, and cracking in bridge decks.

**Advantages:** Capable of examining large areas quickly.

**Disadvantages:** Relatively high cost and it provides information on the extent of delamination only in two-dimensional terms. Defects located under delamination areas cannot be detected.

**A.1.5. Impact Echo (IE)** – The Impact Echo technique may be utilized from either side of the deck to determine the extent and depth of the delamination, voids, and cracks <sup>(40)</sup>. IE may also be used to obtain a qualitative indication for the quality of the concrete.

**Applicability:** Capable of determining the extent and depth of delaminations or member thickness. Also capable of determining extent and depth of voids, honeycombing, crack defective surface patches, and embedded metallic materials.

**Advantages:** Access to only one face of the element is required. Ideal for local monitoring. Relatively inexpensive. Capable of locating a variety of internal defects. Relatively inexpensive.

**Disadvantages:** Time consuming and expensive when large areas are to be monitored. Results interpretation depends on the operator's experience.

**A.1.6. Spectral Analysis of Surface Waves (SASW)** – SASW can be used to determine the characteristics of the concrete profile through its thickness. This is accomplished through the use of an impact device and two receivers. The signals received are analyzed and the profile is determined based on reflections from defects or artifacts inside the concrete. Analysis of the profile will give an indication of the soundness (or delamination, etc.) and condition of the layers through the bridge element.

**Applicability:** Can be used to determine the stiffness or delaminations through the bridge deck.

**Advantages:** Reasonably capable of determining the location of voids and hidden subsurface defects in the bridge deck. Relatively inexpensive.

**Disadvantages:** Time-consuming process for a general bridge deck analysis. Extensive data analysis is required by experienced operators.

**A.1.7. Radiography** – X-ray and Gamma ray radiography provide photo images of the interior of a concrete member and record the resulting images on film or in digital form by receiving sensors or detectors. This is done by introducing radiation into the member. Denser materials block more radiation, making identification of voids in concrete, voids in post-tensioning ducts, and reinforcement location or corrosion possible <sup>(41)</sup>.

**Applicability:** May be applied to concrete provided both sides of the member are accessible. The technique can determine the location of voids, honeycombs, or embedded materials as well as major corrosion of reinforcing steel.

**Advantages:** Internal flaws inside concrete can be imaged and inspected.

**Disadvantages:** Generally, it is a difficult test method for field applications and needs adequate radioactive protection. Licensed operators required for use. Cracks perpendicular to radiation beam are difficult to interpret. Gamma ray radiation is limited to 20 inches.

**A.1.8. Intrusive Visual Examination** –The access hole is drilled into the concrete to allow the fiber optics, video cameras, and periscopes to explore inside the member.

**Applicability:** This test method may be used to examine the extent of voids and similar flaws inside of concrete or partially filled post-tensioning tendons. It may also be used to explore hard to reach or inspect areas such as bearing areas or member ends near abutments or supports.

**Advantages:** Capable of viewing generally inaccessible areas of the bridge element. Relatively inexpensive and simple to use.

**Disadvantages:** Viewing image is limited to small areas.

**A.1.9. Core Sampling** – Core sampling may be used to verify the presence of steel corrosion, voids and delamination, and to determine concrete strength directly. Coring can also be used for verification of other NDT methods to determine concrete strength <sup>(42,43)</sup>. Various laboratory tests such as acid base indicators or petrographic analysis may be performed on extracted cores to obtain information on carbonation content and depth and on various deterioration mechanisms in concrete.

**Applicability:** Core sampling is applicable to all types of concrete elements provided the sampling does not affect the strength requirements of the bridge.

**Advantages:** Provides true indication of concrete's compressive strength. Provides validation for other NDE techniques.

**Disadvantages:** It is an intrusive method that requires subsequent repair. It is slow and relatively expensive for monitoring large areas.

**A.1.10. Ultrasonic Pulse Velocity (UPV)** – UPV may be used on the deck surface to measure the depths of cracks or it may be used to determine delaminations and voids. It can also be used to estimate the overall concrete quality. This is normally done by measuring velocity of high frequency mechanical waves inside concrete the use of piezoelectric sensors.

**Applicability:** UPV surface techniques can be used atop the bridge deck to estimate the depths of cracks. This technique can also be used atop the bridge deck to determine the thickness of surface layers, such as overlays. The general location of internal voids and defects can be determined using the UPV method across the deck cross-section. A reasonable relationship between the velocity of the pulse and concrete strength exists, that which could offer an approximate estimate of concrete strength.

**Advantages:** It is a portable and easy to use test method that

offers crack and deck depth measuring capabilities, as well as detecting delamination and voids.

**Disadvantages:** Can be a time-consuming process to be used for general evaluation. Only gives a reasonable approximation for crack depth when using surface methods.

**A.2. Cracking** – The following test methods are suitable for detecting and evaluating the extent of cracking in concrete decks:

**A.2.1. Visual Inspection** - Visual Inspection is the most important as it provides indications of possible causes of cracking directly from clues on the deck surface. Among these possible clues are:

- Presence of Exudation (alkali-silica reactivity).
- Presence of Efflorescence (chloride intrusion).
- Crack Orientation/Location
- Relative Age of Cracks/Structure
- Present state of cracking (active or not)

**Applicability:** Visual inspection must be an integral part of any through inspection program for nearly all types of structures.

**Advantages:** It provides an overall understanding of the state of the problem. It is quick and inexpensive and provides direction for the requirements of future investigations.

**Disadvantages:** Only visual indications are available. Needs experienced and knowledgeable individuals to perform visual inspection.

**A.2.2. Ultrasonic Pulse Velocity (UPV)**

(See section A.1.10 in this chapter)

**A.2.3. Impact Echo (IE)**

(See section A.1.5 in this chapter)

**A.3. Corrosion of Reinforcing/Prestressing/Post-Tensioning Steels –**

The following test methods are suitable for detecting and evaluating the extent of steel corrosion in concrete decks:

**A.3.1. Half-cell potential** – Half-cell potential is a measure of the likelihood of corrosion activity. This test requires direct access to a reinforcing bar or strand in concrete. When the results of the half-cell potential test at certain locations are less than  $-0.35$  volts it is assumed that there is a 90% likelihood of reinforcement corrosion <sup>(44)</sup>.

**Applicability:** May be used to determine areas of potential reinforcement corrosion. Readings will need to be taken on a grid system so the corrosion potential topography can be determined.

**Advantages:** Lightweight, portable equipment. Inexpensive, and quick.

**Disadvantages:** Requires a direct connection to the reinforcement and an internal connection among the reinforcing inside the concrete element. Concrete needs to be moist.

**A.3.2. Linear Polarization** – The Linear Polarization Method measures the rate of corrosion in reinforcing steels <sup>(45)</sup>. The method is based on measuring the change in the a short-circuited electrolytic cell in concrete when an external current is applied to the cell.

**Applicability:** Determines the corrosion rate of reinforcing steel in the field.

**Advantages:** Portable and lightweight equipment. Determines corrosion rate at time of test.

**Disadvantages:** Requires direct electrical connection to reinforcement and interconnection of the reinforcing internally. Cannot test epoxy-coated or galvanized reinforcement. Cover depth must be less than 2 inches. Concrete needs to be smooth and un-cracked.

**A.3.3. Magnetic Flux Leakage (MFL)** – MFL can be used to detect corrosion or fracture of reinforcement inside concrete. This system introduces a magnetic field into steel reinforcement embedded within the concrete deck or other members and measures changes in the magnetic field that are representative of damaged reinforcement <sup>(46)</sup>.

**Applicability:** Capable of determining the location and extent of reinforcement corrosion.

**Advantages:** Capable of determining small reductions in the cross-section of steel reinforcement.

**Disadvantages:** A commercial device is not available at this time.

**A.3.4. Covermeters (Pachometers)** – Covermeters are used to determine reinforcement location. The amount of concrete cover can then be determined if the reinforcement size is known <sup>(47)</sup>.

**Applicability:** Capable of determining reinforcement location and extent of cover if size can be determined from bridge construction documents. Can be utilized on any type of bridge element.

**Advantages:** Lightweight, portable equipment. Inexpensive.

**Disadvantages:** Calibration may be required if concrete contains fly ash; due to magnetic interference. Only capable of analyzing near surface reinforcement conditions.

#### **A.3.5. Core Sampling**

(See section A.1.9 in this chapter)

**A.4. Excessive Permeability and Chloride Content** – The following test methods are suitable for assessing the condition of concrete decks that are considered potential candidates for ingress of aggressive chemicals and corrosion of steel in the deck. Chloride intrusion into concrete can lead to corrosion of reinforcing steel and cracking of the concrete. Tests that allow determination of chloride permeability resistance or chloride content are partially destructive. Core samples must be extracted from the deck and tested in a laboratory in order to obtain the required knowledge.

**A.4.1. Chloride 90-day Ponding Test** – This test requires ponding of a 3 percent sodium chloride solution on the surface of the concrete for a period of at least four months. Cores are drilled and the chloride ion content is determined in the lab <sup>(48, 49)</sup>

**Applicability:** Test can occur on any area exposed to chloride ion intrusion.

**Advantages:** Capable of determining the susceptibility of concrete to chloride ion intrusion.

**Disadvantages:** Requires a long period (months) to complete test and analysis. Requires a laboratory testing and analysis. Cores must be extracted.

**A.4.2. Chloride Ion Resistance (Rapid Permeability Test)**– This test is a laboratory test conducted on field-extracted cores. An applied voltage is used to drive a 3 percent sodium chloride solution into the depths of the core. The characteristics of the time-current curve give indications of the permeability of the concrete <sup>(50,51)</sup>.

**Applicability:** Test can occur on any area exposed to chloride ion intrusion.

**Advantages:** Capable of determining the susceptibility of concrete to chloride ion intrusion. Total time required for full testing and analysis is approximately one day.

**Disadvantages:** Not as accurate as the 90-day ponding test.

**A.4.3. Drilling and Concrete Powder Sampling** –Chloride content may be determined at various depths of a concrete deck by drilling and extracting concrete powder from these depths. Various laboratory tests to determine the chemical contents may be performed on the extracted powder samples.

**Applicability:** The test may be performed on any concrete member suspected of having high chloride content.

**Advantages:** Gives relatively accurate information on the chloride content at the location of test.

**Disadvantages:** Time consuming. Information is valid only for the test location. Expensive for monitoring large areas.

**A.5. Low Material Quality** – The following test methods are suitable for assessing the quality of concrete material used for the construction of bridge decks:

**A.5.1. Ultrasonic Pulse Velocity (UPV)**

(See section A.1.10 in this chapter)

**A.5.2. Spectral Analysis of Surface Waves (SASW)**

(See section A.1.6 in this chapter)

**A.5.3. Impact Echo (IE)**

(See section A.1.5 in this chapter)

**A.5.4. Core Sampling**

(See section A.1.9 in this chapter)

**A.5.5. Pull-off Testing** – Pull-off Testing may be used to determine the tensile strength of concrete deck and overlays <sup>(52, 53)</sup>.

**Applicability:** Determines the tensile strength of concrete and concrete overlays.

**Advantages:** In-situ determination of tensile strength.

**Disadvantages:** Time consuming, is partially destructive.

**A.5.6. Windsor Probe** – The Windsor probe uses a powder charge to drive a steel probe into hardened concrete. The depth of penetration is an indication of concrete strength <sup>(54)</sup>.

**Applicability:** Capable of determining general uniformity in concrete strength and quality. Approximates concrete strength.

**Advantages:** Identifies questionable areas for further investigation. Fast and simple method of analysis.

**Disadvantages:** Calibration of probe depth to core sample compression strength is generally required. Results are approximate.

**A.5.7. Schmidt Hammer (Rebound Hammer)** – The Schmidt hammer uses a spring-loaded hammer to collide with the concrete surface. Calibration is extremely important since the surface strength of the concrete is correlated to the overall compressive strength of the concrete through calibration curves <sup>(55)</sup>.

**Applicability:** Capable of determining general uniformity of concrete strength and quality. Approximates concrete strength.

**Advantages:** Measures general uniformity. Identifies questionable areas for further investigation.

**Disadvantages:** Angle of test, surface smoothness, type of aggregate, and carbonation of concrete and moisture content effect results. Results are only an indication of surface conditions.

**A.5.8. Direct Transmission Radiometry (DTR)** – Direct Transmission Radiometry is capable of determining the in-place density of concrete. The density characteristics change when reinforcement is present in the area. This is a nuclear method; standards are available which discuss the use of nuclear methods <sup>(56)</sup>.

**Applicability:** Determines the in-place density of a concrete member. Density correlates to concrete strength.

**Advantages:** Portable and easy to use.

**Disadvantages:** Operators must be licensed. Requires access to both faces of the member.

**A.5.9. Backscatter Radiometry (BSR)** – Backscatter radiometry also determines the in-place density of concrete. In principal, the backscatter radiometry technique is the same as the DTR method <sup>(56)</sup>.

**Applicability:** Determines the density of concrete without removing core samples. Density can then be related to concrete strength.

Applicable to all concrete component types.

**Advantages:** Rapid test. No core samples needed.

**Disadvantages:** Operators must be licensed. Not as accurate as direct transmission radiometry.

**A.6. Other Defects** – There are other defects that are encountered in bridge decks. These include rust stain, scaling, wear, decoloration, physical damage due to impact, age, environment, fire, and others. There is no clear choice of an NDE technique that is fully suitable for these defects. The most appropriate evaluation of these defects should be initiated using visual inspection performed by one or more qualified individuals. Physical measurements and data from the field should be combined with engineering analysis and appropriate NDE techniques to determine the extent of the defect and its effect on the structure.

## **B. Concrete Bridge Superstructures**

Most non-destructive evaluation techniques described for bridge decks are suitable for use on bridge superstructure as well. The following recommendations regarding critical inspection areas should be followed when examining concrete superstructures.

**B.1. Cracked Areas** – All areas of a concrete structure should be examined visually for possible cracking. However, critical areas should be examined more closely. These include:

**Shear Zones** – Areas of high shear forces are generally located at the supports in either continuous or simple span slabs, beams, and girders. Cracks due to shear will be diagonal in orientation as previously discussed in Section II.B. Documentation of the size and extent of the cracking is required and should be added to the bridge file.

**Tension Zones** – Tension areas are a major concern in all bridge structures. Flexural cracks, which are vertical in nature, may develop at midspan in simply supported members. In continuous members, these cracks may develop at midspan and at intermediate and end supports.

**Bearing Regions** – Bearing regions are prone to spalling due to concrete expansion and contraction coupled with nonfunctioning bearing devices. Bearing regions may also be susceptible to damage from salt runoff. Scaling and spalling result from the freeze-thaw deterioration in these areas.

**B.2. Corrosion Prone Areas** – Tightly packed areas, near supports and connections, may be corrosion prone. Areas exposed to spray from the roadway are prone to chloride penetration, freeze-thaw deterioration, and the associated corrosion. Areas exposed to roadway runoff should also be examined for damage due to chloride intrusion, etc. Rust stained areas and cracks accompanied by efflorescence are indications of reinforcing deterioration.

**B.3. Vehicular Damaged Areas** – All bridge members exposed to traffic

must be examined for vehicular or ship impact damage. The examination should include condition assessment for exposed steel reinforcement and possible reduction in steel section.

**B.4. Excessive Deflection** – Physical damage and overload are the most common causes of excessive deflection in reinforced concrete elements. In addition to the effects of physical damage and overload, an excessive deflection in pre- and post-tensioned elements may be an indication of loss of bond and relaxation of the reinforcing strand, corrosion of the strand, corrosion in the anchorage zone, or loss of bond in the anchorage zone.

**B.5. Previously Repaired Areas** – Determination of the effectiveness of a previous repair should be made during inspection. Evidence of cracking or delamination may be an indication of an ineffective repair.

## **V. Non-Destructive Evaluation of Steel Bridges**

There are two primary problems affecting steel: corrosion and cracking. Physical damage, such as impact or fire is also of concern in steel bridges. Corrosion leads to both section loss and increased cracking due to the creation of corrosion stress raisers; ultimately reducing load carrying capacity. The other common problem in steel is fatigue cracking. Since fatigue cracking can lead to a catastrophic failure, it must first be detected, then monitored closely or repaired.

Descriptions of possible flaws in steel materials and structures as well as appropriate NDE techniques for their detection and assessment are tabulated and presented in Appendix B. Summary discussions and appropriate references for various related NDE test methods are presented in Appendix C. The following presents appropriate test techniques that may be used to evaluate the condition of steel bridge decks and superstructures. For each test technique, visual inspection by an experienced inspector is a necessity, as it will generally reveal indications on the type of defects present.

Numerous laboratory and field evaluations were made as a part of this and other similar studies by the research team to obtain verifications of the characteristic features of several NDE test methods included in this report. These methods include acoustic emission, ultrasonic, ultrasonic imaging, pulse velocity, magnetic particles, dye penetrant, impact-echo, magnetic flux leakage, sounding, time domain reflectometry, Chloride content measurements, and half-cell potential. These NDE methods were studied in the various laboratories of the University of Wisconsin-Milwaukee. They were also evaluated at the sites of several bridge structures in Wisconsin and other states. Other NDE methods that are listed in this report were evaluated based on studying the available literature and using the past experience of the research team.

### **A. Steel Bridge Decks**

Like any steel bridge member, steel bridge decks experience problems including corrosion, cracking, and physical damage. To detect and assess the extent of each defect, one or more appropriate test technique should be identified and used.

Steel decks in bridges are in the forms of grid decks (open or filled with concrete), orthotropic decks, and corrugated decks. The orthotropic and corrugated decks are normally covered with concrete/asphalt overlays. Due to the presence of welding and induced stresses, fatigue cracking is a concern. Also, due to the effect of environment, chloride effect, and geometry of the deck elements, corrosion can be a common problem in steel decks.

Inspection of steel decks can be performed most effectively through visual inspection. Most commonly used non-destructive testing techniques for steel structures, i.e., ultrasonic, dye penetrant, magnetic particles, and

radiography prove ineffective due to complex geometry and large areas to be inspected. NDE methods such as ultrasonic, acoustic emission, and radiography may be used for condition assessment in localized regions.

## **B. Steel Bridge Superstructures**

Steel bridge superstructures are also subjected to corrosion, cracking, and physical damage. Fatigue cracking due to cyclic loads from the effects of traffic and temperature fluctuation, coupled with the presence of various welded details has been a major problem in steel bridge structures due to poor design, construction, and/or maintenance.

Condition assessment for steel bridge superstructures may be performed best by integrating an effective visual inspection with the use of appropriate NDE test methods. The effectiveness of a visual inspection and appropriate NDE test depends on the knowledge of the bridge inspector to identify critical areas of the structure that have potential for problems. The inspector must also have general knowledge about the capability and limitations of various relevant NDE test methods. Critical areas in steel bridge superstructures that require closer and more in-depth inspection for cracking include:

- welded connections and details
- locations with intersecting welds
- details that induce out-of-plane distortion in a member
- details and regions that are subjected to high levels of tensile stress ranges
- details that contain copes.

Areas susceptible to corrosion must also be examined thoroughly. These include tightly-packed areas near supports and connection regions. Areas exposed to spray and runoff from the roadway are prone to chloride exposure and freeze-thaw deterioration. Another area in which corrosion is prevalent is the bearing region. Residue buildup in the normally tight area of the bearing region causes accumulation of moisture and aggressive chemicals that increase the possibility of corrosion.

## **C. Non-Destructive Test Techniques**

Most available NDE Test Methods could be applied to either steel bridge decks or superstructures. To detect and assess the extent of each defect, one or more appropriate test techniques should be identified and used. The following test techniques are appropriate for use in steel bridge members with specific defects or flaws.

**C.1. Cracking** - Cracking can be detected by a variety of means. The following lists the typical techniques used to detect cracked areas.

**C.1.1. Visual Inspection** – A thorough, in-depth, visual inspection will help determine areas of cracking. Although necessary and important, visual examination is not a reliable technique for crack detection in steel bridges.

**C.1.2. Ultrasonic Testing (UT)** – UT may be used to determine the extent or existence of cracking in various steel bridge components. UT testing introduces an ultrasonic pulse into the metal and the reflection of the signal from the internal boundaries of the material gives indications of the types and extent of defects.

**Applicability:** Ultrasonic testing can be utilized for nearly all steel components of a bridge. Piezoelectric transducers are available to test directly into the material or angled to force the ultrasonic beam into difficult to reach areas.

**Advantages:** Versatile for testing different steel sizes and orientations. Access to only one face is needed to perform testing.

**Disadvantages:** Interpretation of results depends on the operator's skill.

**C.1.3. Magnetic Particle Testing** – Magnetic particle testing can quickly identify cracked areas on the steel surface. This technique introduces a magnetic field into the steel surface and small metallic particles in the vicinity of a possible crack. The edges of the crack attract the particles and clearly identifies the surface crack.

**Applicability:** Capable of testing any steel surface for cracking. Identifies extent of surface cracking.

**Advantages:** It is an effective and quick test method for detecting small cracks. Does not require paint removal to perform test.

**Disadvantages:** Limited to near surface, surface, or thinly painted surfaces.

**C.1.4. Dye Penetrant Dye** – Dye penetrant testing can be used to identify surface cracking in steel. The steel surface must be cleaned to bare metal and a dye is applied to the surface. The dye is allowed to penetrate the metal and excess dye is removed. A developer is applied and the dye at the cracks is drawn out, identifying the extent of the cracking on the surface.

**Applicability:** Capable of determining the extent of surface cracking on steel surfaces.

**Advantages:** Effective and inexpensive materials. May be applied to Aluminum and stainless steel.

**Disadvantages:** Time-consuming process due to paint removal requirements. Depth of defects cannot be determined. Subsurface flaws cannot be detected. It is effective between 40 °F and 120 °F.

**C.1.5. Acoustic Emission (AE)** – Acoustic Emission is a technique that monitors the release of energy into the structure when microscopic cracks occur. Sensors are placed at crack prone areas. The sensors detect this released energy due to the cracking. AE is an effective technique for detection and monitoring of active cracks in local areas. It is possible to determine the extent of existing fatigue crack growth through the use of AE.

**Applicability:** Capable of determining the location of cracks and the occurrence of cracking. Highly capable crack monitoring

technique.

**Advantages:** Monitors local areas well, and can identify crack tip location.

**Disadvantages:** Monitoring of global areas requires numerous sensor placements. Equipment cost is relatively high.

**C.1.6. Coating Tolerance Thermography (CTT)** - Can be utilized to determine cracking extent. CTT utilizes the fact that heat flow characteristics are different between undamaged steel and cracked or corroded steel. This method involves the application of heat to either side of the possible defect. A camera is used to capture the thermographic image. After data processing, the cracks or corroded areas are differentiated in black and white on the thermographic image.

**Applicability:** Capable of identifying subsurface and surface cracking in steel materials. Currently, this technique is rarely used.

**Advantages:** Can identify cracking and corrosion beneath painted surfaces.

**Disadvantages:** Expensive equipment.

**C.1.7. Radiographic Testing** – Radiographic testing may be used to detect cracks and evaluate their extent. Radiographic testing introduces a radiation source at one side of the member and captures the image on the other side on film. Access to both side sides of the member is required.

**Applicability:** Useful in testing any steel materials provided access is available to introduce and capture the image from each side of the member.

**Advantages:** Capable of determining the internal and subsurface characteristics of the steel material. It may also be used for aluminum and stainless steel.

**Disadvantages:** Requires access to both sides of the member. Testing cost is high, requires protection shielding.

**C.2. Corrosion** - The typical techniques used for determining corrosion presence or extent are as follows:

**C.2.1. Visual Inspection**

(See section C.1.1 in this chapter)

**C.2.2. Ultrasonic Testing (UT)**

(See section C.1.2 in this chapter)

**C.2.3. Coating Tolerance Thermography (CTT)**

(See section C.1.6 in this chapter)

**C.2.4. Measurement of Section Loss** – Removal of debris and rust flakes will allow a determination of the change in geometry associated with corrosion by measurement and comparison to as-constructed conditions.

**C.3. Physical Damage/Misalignment/Excessive Deflection**

**C.3.1. Measurement/Surveying** – Visual examination and surveying can help determine any differential movement between different bridge

components. Surveying can also give an indication of actual in-service deflections of bridge components.

## **VI. Non-Destructive Evaluation of Bridge Substructures**

Bridge substructures generally include wingwalls, abutments, piers, piercaps, piles, pile caps, and spread footings. Other substructures such as caissons and special piles may also be used. There are generally three main problems involved in bridge substructures: movement, scour, and deterioration <sup>(57)</sup>.

**1. Movement** – Vertical, lateral, and rotational movements in substructures need to be closely monitored. Excessive movement of the substructure components can adversely affect the structure and lead to the development of unintended stresses within the bridge superstructure. If vertical movements occur uniformly over the entire structure, the resulting stresses will generally be small. Localized or differential movement of the substructure is the most damaging to a bridge structure. Lateral movements are commonly caused by soil failure. Rotation of abutments can result if applied lateral loads exceed the shear capacity of the supporting soil. Rotational movement also occurs as a result of scour, substructure material deterioration, or underlying soil failure.

**2. Scour** – Scour is the deterioration or removal of the soil under a submerged bridge substructure. The flow of water in the vicinity of the substructure continually forces the soil material away from the substructure and moves it downstream. Scour can be extremely destructive, causing excessive unintended movement. Scour can reduce the effective area of a foundation due to soil removal. The soil strength is also reduced due to instability.

**3. Deterioration** – Substructures used in bridges traversing a waterway are often directly exposed to water. Wet-dry cycles, due to the constant exposure to water, can lead to possible freeze-thaw deterioration. Other deterioration due to environmental conditions can occur in the form of scaling, spalling, and those problems resulting from reinforcement corrosion due to chloride penetration. Deterioration due to design, construction or erection problems, as well as overloads, is also possible.

These problems can occur individually or in combination with each other. The inspector's duty is to be cognizant of all problems and their indicators.

In the following section, the evaluation of bridge substructures is discussed. Those defects that are typical to the individual substructure types and applicable testing techniques are also provided.

### **A. Wingwalls, Abutments and Piers**

Wingwalls are used to retain fill and achieve an elevation change. Wingwalls are therefore primarily designed to resist overturning moment of the abutment due to lateral earth pressures. The main lateral forces acting on the structure to produce the overturning moment are the

combination of earth and traffic pressure transferred through the soil. The primary reinforcing steel locations are generally on the face of walls near the supported soil. At the outer toe of the footing, away from the vertical soil wall, the primary reinforcement is located at the bottom of the footing. Consequently, the main reinforcement is located in the upper portion of the footing underneath the retained soil <sup>(58)</sup>.

Abutments serve as a retaining wall to the approach embankment and support the end of the bridge structure. The size, shape, and type of the abutments affect the location of, or required use of, reinforcing steel. It is recommended that appropriate engineering documents for the abutment be reviewed to determine the reinforcement locations <sup>(59)</sup>.

Piers/Bents are intermediate supports, placed between abutments to extend the distance traversed. They differ from abutments in that they primarily resist axial load and/or moment without lateral earth pressure. Piers are constructed using either steel or concrete materials. Concrete piers are typically reinforced on their perimeter, with a varying amount of cover. When the pier is not directly connected to the bridge superstructure, a beam, or bent cap, is designed and used to integrate it with other piers and the bent cap accepts the loads from the bridge superstructure <sup>(60)</sup>.

#### **Inspection Comments for Wingwalls, Abutments and Piers:**

Similar inspection techniques and approaches may be used to evaluate the conditions of wingwalls, abutments, and piers. Typical NDE techniques for steel and concrete, as described in earlier sections of this report, may be used to evaluate the general conditions and to determine the extent of defects in the substructure components. Scour can be evaluated by both underwater and above water inspection techniques. Impact-echo, impulse response, and SASW are all possible inspection techniques for determining component geometry and scour extent from the exposed surface of the substructure. Generally, the main difficulty in evaluating scour is lack of access. The techniques mentioned above need access to one face only.

#### **B. Piles and Footings**

Piles are used to transfer loads from the superstructure back to the earth. Piles are used when the underlying soil conditions are unsatisfactory to resist the applied loads directly. They are driven into stronger soil or rock in order to transfer the loads to a stable area. Several piles may be utilized and connected via spread footings. Piles in bridge construction are usually concrete, but steel piles have also been used.

### **Inspection Comments for Piles and Footings:**

There are several techniques available for the verification of the geometry and soundness of footings and driven piles. Some of these techniques concentrate primarily on recording and evaluating the impact wave reflections from a source located at the top of the pile or footing. Others measure directly through the section of the pile to determine integrity, and require special considerations for positioning of the testing apparatus (tubes or drilled holes) and access along the length of the pile. In the case of surface reflective methods, pre-placed tubes are not required. Depending on how and where these methods are used, one can assess the condition and verify the geometry of different types of piles and footings. Test methods for verifying depth of piles and footings are as follows <sup>(61)</sup>:

**B.1. Sonic Echo** – This technique uses a small impact delivered to the head of a deep foundation. The impact wave is reflected to a transducer located near the impact and analysis of the resulting signals provides indications of the depth, or quality of the concrete.

**Advantages:** Does not require placement of tubes along length of pile. Equipment is portable and quick to use.

**Disadvantages:** Does not measure diameter of piles. Cannot determine defects in piles greater than 100 feet deep or in piles with length to diameter ratios greater than 30.

**B.2. Impedance Logging** – This technique combines the sonic-echo and impulse response data to determine the characteristics of the pile. Including diameter and depth.

**Advantages:** Shape of pile can be determined. No tubes need to be placed. Portable equipment. Rapid testing technique.

**Disadvantages:** It requires substantial data analysis.

**B.3. Impulse Response** – This technique introduces a known constant frequency force that vibrates the pile and the response of the pile is measured with velocity transducers. Interpretation of the results gives indications of pile depth or defect depth.

**Advantages:** No tubes need to be placed. Capable of measuring the stiffness of the pile and therefore determine the relative concrete strength.

**Disadvantages:** Interpretation of results is complicated. Cannot determine defects in piles greater than 100 feet deep or in piles with length to diameter ratios greater than 30.

**B.4. Cross-Hole Sonic Logging** – This technique is the same as UPV but through pre-placed or drilled tubes at certain locations in the pile. The pulse is sent between tubes with changes in the signal indicating defects.

**Advantages:** Fast technique. Detection of defects is more accurate than techniques that use surface impact.

**Disadvantages:** Tubes need to be placed or holes need to be drilled in the pile. May miss defects at the pile shaft edge.

**B.5. Parallel Seismic** – A borehole is drilled into the soil parallel and close to the foundation and the head of the foundation is struck with a hammer. Data acquisition from the hole and subsequent analysis can determine the internal characteristics of the pile, including its depth.

**Advantages:** Fast technique. Foundations under structures can be tested.

**Disadvantages:** Borehole must be drilled. Cannot detect defects past first defect encountered. Edge defects are difficult to detect.

**B.6. Gamma-Gamma Logging** – This technique is generally the same as cross-hole sonic logging, but with a radiographic source.

**Advantages:** Fast technique. Detection of defects is more accurate than techniques that use surface impact.

**Disadvantages:** Tubes need to be placed or holes need to be cored. May miss defects at the pile shaft edge.

## **VII. Non-Destructive Evaluation of Bridge Bearings**

Bridge bearings are used to transfer loads and to provide a means of movement to account for the expansive and contractive nature of both concrete and steel when subjected thermal changes. Bridge bearings can provide a near “simple” support, with actual pin and roller connections, meaning the structure condition in the field approximately mirrors that of the engineering analysis. A bridge bearing may be loaded in a fashion not intended or conceived in the design process if its functionality has been compromised due to defects or other problems

Typical types of bridge bearings include pot-type, elastomeric, structural slide, rocker, roller, pin and link, and restraining bearings. The following describes various types of bearing and associated problems. It also presents recommendations on their inspection <sup>(62)</sup>.

### **A. Bridge Bearing Types and Associated Problems**

Problems that are commonly encountered in bridge bearings are elastomeric and steel material deterioration and cracking of steel bearing components.

The elastomeric material deterioration is mostly due to the effects of harsh environment, overloads, and aging. The results are cracking and excessive bulging of the elastomeric components.

The material deterioration associated with steel components of bridge bearings mostly involves corrosion. Severe service and environmental conditions, including intrusion of sodium chloride, are primary factors contributing to the corrosion of these components. The severity of the problem is increased by the inherent location of bridge bearing components. Often, there is a build up of debris and residues containing aggressive chemicals and moisture accumulation that contribute to the corrosion process. There are also possibilities for fretting corrosion and galvanic corrosion when dissimilar metals are in contact with one another.

Although not as common as the corrosion problem, cracking may also occur in steel components of bridge bearings. The cracking of steel most commonly occurs in rocker, roller, pin and link types of bridge bearing.

### **B. Non-Destructive Test Techniques**

The following NDE test techniques are appropriate for inspection of various types of bridge bearings:

**B.1. Visual Inspection** – Detection and assessment of elastomeric material and steel deterioration are most easily accomplished by visual inspection.

**B.2. Magnetic Particle Testing** – Since most bridge bearing devices are painted, magnetic particle testing can be utilized quickly and effectively to identify crack presence and extent.

**B.3. Ultrasonic Testing** – Ultrasonic inspection can be used when access is available. Corrosion extent may be estimated by careful interpretation of the ultrasonic signals.

**B.4. Dye Penetrant Testing** – In areas where paint is not applied, or when it can be removed, dye penetrant inspection is an effective means of determining the presence of cracking. This technique requires access to the component.

## **VIII. Non-Destructive Evaluation of Movable Bridges**

Movable bridges are used across many navigable waterways as a means of allowing vehicular traffic across the bridge and marine vessels below the bridge. The overall inspection of movable bridges will involve considerations such as public safety, navigational safety, structure safety, and dependable operation <sup>(63)</sup>. Proper sequencing is required before and during an inspection. The following presents a discussion on the details relevant to the non-destructive evaluation aspects in movable bridge inspection. A highly recommended information source for inspection of movable bridges is the USDOT/FHWA's "Bridge Inspector's Training Manual/90."

Common types of movable bridges are Swing bridges, Bascule bridges, and Vertical lift bridges. Most problems encountered in moveable bridges include fatigue and other cracking, wear of contacting surfaces, mechanical malfunction of machinery, and corrosion. Non-destructive evaluation of the superstructure part of these bridges is similar to that for conventional steel bridge structures as presented in the earlier part of this report. Cracking, due to fatigue and overload, in gears, tracks, racks and pinions, wires and ropes, drums, sheaves, rotating shafts, castings, and other moving tracks, gears, and other elements have shown to create sources of fatigue crack initiation and should be monitored. Stress level in gears and other parts may increase during the life of the bridge due to an increase of restraining forces. These forces may develop from the effect of corrosion at joints and frictional forces due to lack or inadequacy of lubrication and wear of moving contact surfaces. It is recommended that true stress level in critical components is determined by measuring strains or by other means during the service lives of these bridges. The features, with regards to inspection of movable bridges, are discussed below.

### **A. Swing Bridges**

Swing bridges use continuous girders or trusses that are partially or fully supported by a center pivot. This pivot is located on a pier, typically centrally located in the navigable waterway. When open, the girders or trusses cantilever over this pivot. When closed, the ends are wedged to ensure support and load transfer through the outer edges of the cantilever span into the end piers or abutments.

#### **Typical Features:**

The swing bridge uses a central pivot and roller ring with radial roller shafts centering the rollers, as in a rim-bearing bridge. A central drum or girder with beveled rollers underneath may be utilized as in a center-bearing bridge. The spans of the bridge are constructed of steel with a steel grid deck. The required power in swing bridges is typically supplied by electric motors, hydraulic motors, or hydraulic cylinders. The rotation of the swing bridge is through a circular rack and pinion.

#### **Possible Inspection Areas:**

The surfaces of the rollers are possible areas of wear. The bases of the gear teeth at the rack and pinion are areas of possible cracking. The

central drum or girder will have regions of high stress at weld intersections or points of highly localized loading. The roller tracks are also possible areas of cracking.

**Applicable Inspection Procedures:**

Magnetic particle inspection will provide an effective means of determining the extent of cracking on painted and unpainted surfaces. Dye penetrant is well suited for gear teeth inspection since they should be paint free.

Ultrasonic inspection can provide an in depth determination of cracking.

Testing of shafts, which transfer power between different components, can be performed through dye penetrant inspection on unpainted surfaces and magnetic particle inspection or ultrasonic testing on painted surfaces.

Acoustic emission test techniques can be used for various components of moveable bridges to detect active fatigue cracks or to assess the level of their severity during the operation of the bridge.

**B. Bascule Bridges**

Bascule bridges utilize one or two movable portions, called leaves, of the bridge span. A single leaf orientation uses a counterweight at one end of the span. In a two-leaf bridge, the bridge is split usually at midspan and the individual leaves pivot on a horizontal axis to allow marine vessel movement through the navigable waterway.

**Typical Features:**

Bascule bridges typically utilize large castings at the rotation points supporting the individual leaf. The casting is rotated by a shaft that is supported at both sides of the casting. The large castings typically are part of a rack that connects to a driving pinion. The pinion rotates the casting and, in turn, the individual leaves. A series of gears are used to magnify the power provided by the driving motors.

**Possible Inspection Areas:**

Castings have inherent discontinuities, due to the casting process that can serve as cracking and deterioration initiation points. Cracking at the teeth of racks and pinions can occur, typically at the root of the tooth. The bearings and their supports may be subject to cracking. Cracking at the bearing will likely occur just below the bearing surface. Shafts and hubs are also subject to cracking, but the likelihood of cracking is less than in other areas.

**Applicable Inspection Procedures:**

The defects in the support castings can be evaluated using magnetic particle inspection or ultrasonic evaluation. The extent of cracking can be determined in bearing supports through magnetic particle inspection or ultrasonic testing. Dye penetrant testing can be used at non-painted areas such as gear teeth and pins. Acoustic emission test techniques can be used for various components of moveable bridges to detect active fatigue cracks or to assess the level of their severity during the operation of the bridge.

### **C. Vertical Lift Bridges**

Vertical lift bridges move the entire span above the navigable waterway. This is accomplished through the use of towers at each edge of the waterway. One type of vertical lift bridge utilizes ropes, sheaves, and drums driven by electric motors at the top of the bridge span. The other type utilizes ropes, sheaves, and drums located on the lift towers.

#### **Typical Features:**

Vertical lift bridges use wire, ropes, and sockets. The wire ropes wind on drums and are maneuvered onto the drum by sheaves and pulleys. The rope drums will likely be connected to a rack, connected to a pinion, which then drives the drum.

#### **Possible Inspection Areas:**

Wear and fracture of the wire rope is expected in lift bridges. The rope drum may crack in the rope grooves. The rope sheaves will likely undergo wear from rope movement. Cracking can occur in gear teeth, at bearing supports, at connection shafts, and hubs.

#### **Applicable Inspection Procedures:**

Dye penetrant testing can be used at non-painted areas such as gear teeth and pins. Magnetic flux leakage may be used to assess the condition ropes and wires during the service life of the bridge. Magnetic particle and ultrasonic inspection can be used on painted surfaces such as the support castings, motor shafts, etc. Acoustic emission test techniques can be used for various components of moveable bridges to detect active fatigue cracks or to assess the level of their severity during the operation of the bridge.

## **IX. NDE-Based Estimation of Remaining Life**

A reliable estimation of the remaining service life of a bridge structure may not be made without using appropriate quantitative non-destructive evaluation test data. In this context, the primary objective of performing an NDE test program should be to accurately determine the extent of damage and deterioration in various components of a bridge structure. The NDE test results must be quantitative so it could be incorporated into appropriate analysis and design approaches in order to calculate the remaining life of the structure. Estimating the remaining service lives of bridge structures under various conditions encompasses a broad engineering area as there are numerous bridge types, materials, and damage and deterioration types. While it is not within the scope of this study to offer methods for estimating the remaining service life of all bridge structure types, a general discussion is offered below to aid the bridge engineer in considering the relevant issues and taking appropriate actions.

Bridge engineers are generally familiar with the various aspects of bridge analysis, design and behavior. However, most may encounter difficulties in incorporating quantitative NDE test results into their efforts to estimate the remaining service life of bridge structures. This is due to the fact that knowledge in specialized engineering areas must be used in order to accomplish this task. Unfortunately, these specialized engineering topics are not generally included in the normal undergraduate structural engineering curriculum at the U.S. universities and, therefore, most bridge engineers do not have an opportunity to develop the relevant expertise. For success to be achieved in estimating the remaining service lives of bridge structures, the bridge engineer will need to either acquire the required knowledge or seek assistance from those who are experts in technical areas including qualitative and quantitative non-destructive testing and evaluation, materials behavior, corrosion engineering, materials and structural deterioration mechanisms, fatigue of materials, fracture mechanics, and other related topics.

Many different cases may be encountered in bridges when one must determine the remaining life of a bridge structure when damage or deterioration is detected. One example is when the amount and depth of chloride ion penetration in a concrete member is known from field and laboratory measurements combined with information associated with the length of time in-service, past and expected future levels of exposure to aggressive elements such as chloride ion, and the extent of existing corrosion in reinforcing elements. Combining the knowledge of bridge engineering and corrosion engineering would be required here to achieve reliable estimation of the remaining life in the structure. Another example is when a crack is detected in a fracture critical structural member and its size and depth are measured through NDE testing and it is required to determine the remaining life or safe available length of time to a future repair or retrofit effort. This goal may be achieved by utilizing the knowledge of bridge engineering, concepts of materials engineering, fatigue and fracture mechanics, and information on the environment and expected future load levels and frequency.

Other examples of damage or deterioration include cases that are resulted due to use of poor quality materials, poor design details, poor construction practice, adverse environmental effects, routine wear and usage, vehicular or ship impacts, overload, fire, scour, earthquake, act of terrorism, and others. Separate studies will need to be initiated and conducted to consider these cases so specific guidelines could be developed for dealing with those cases that are often encountered in bridge structures.

## **X. Conclusions and Recommendations**

This report provides relevant information to identify various defects and problems in Wisconsin bridges and it offers a guide for identifying and using appropriate non-destructive test techniques for condition assessment of different bridge components. Both steel and concrete bridges have been included in this study. Issues related to bridge superstructure, substructure, and moveable bridge components have been discussed in the report. For each defect type in a specific structure, appropriate NDE tests have been identified, and for each test the report provides information on the applicability, advantages, and disadvantages. Supplementary detailed information including various possible defects and associated NDE techniques and their descriptions have been provided in the Appendices of the report. A general discussion on the estimation of the remaining service lives of various bridge structures based on results from NDE tests has also been included in the report. Detailed methods of estimating the remaining service lives of bridge structures are not within the scope of this study, but the discussion in the report will aid the bridge engineer in considering the relevant issues and taking appropriate actions in these cases.

Since detailed studies of particular NDE test methods for bridge evaluation were outside of the scope of this study, it is recommended that a future study be initiated to address this need. The new study should identify field-worthy and effective NDE test methods and should further develop test procedures, data archiving methods, and result interpretation for the most common structures and member types that are present in Wisconsin bridges. A detailed guideline should be developed as a part of the study. It should include planning, mobilization, safety considerations, implementation, result collection and interpretation, and final structural assessment aspects for the selected test techniques.

Future research and development are recommended to arrive at accurate bridge rating and estimation of remaining service life through the integration of appropriate NDE test data and engineering analysis. From the results of NDE test, the extent of deterioration in bridge components may be determined that can allow a rating as well as estimation of the remaining service life of the bridge structure to be made with a higher degree of confidence. The study should also include the integration of the NDE test data into bridge management systems to enhance bridge management in Wisconsin.

It is also recommended that a program of training in the area of NDE of bridge structures be developed and implemented. The program should include the development of appropriate guidelines and a series of workshops to train the DOT personnel on the various aspects related to condition assessment of bridge structures in Wisconsin. This training should also include a program of acquisition of selected NDE equipment for the DOT personnel.

## **Appendix A. Concrete Problems and Associated NDE Techniques**

The following section details the concrete problems and NDE techniques discussed throughout this document. The information has been tabularized for easy reference. There are three tables given in this section and described as follows:

### **Table A. Overview of General Concrete Problems and Causes**

This table discusses the general problems encountered in concrete and relates them to their specific causes.

### **Table B. Cracking Indications in Concrete**

Since cracking is somewhat difficult to classify, this table was developed to help delineate between the different cracking indications experienced in concrete structures. The table links the cracking manifestation to the underlying deterioration mechanism.

### **Table C. Concrete Testing Techniques**

This table lists the available concrete testing techniques. The table is divided into sections by the type of information that will be gathered, i.e., material properties, chemical make-up, etc. Appropriate cross-references to standard tests or specifications are given.

**Table A. Overview of General Concrete Problems and Causes**

General Problem	Possible Causes
Cracking Types	
Checking	Expansion/Contraction, Alkali-Silica Reactivity, Alkali-Carbonate Reactivity, Fire Damage, Freeze-Thaw Damage
Circular or Elliptical cracks	Reinforcement corrosion, eventual spall and delamination
Crazing	Expansion/Contraction, Alkali-Silica Reactivity, Alkali-Carbonate Reactivity, Fire Damage, Freeze-Thaw Damage
D-Cracking	Freeze-Thaw Damage, likely near joints
Diagonal Cracking	Shear cracking in flexure members near simple supports Settling or failure of substructure components
Long, Linear Cracking, through cement paste (looking through section)	Plastic Shrinkage, Plastic Settlement, Early Thermal movement, Drying Shrinkage
Parallel cracking oriented with Reinf. Layout	Reinforcement corrosion, spall and delamination
Pattern Cracking	Alkali-silica reactivity, Alkali-Carbonate Reactivity, Restraint to volumetric changes
Transverse Cracking	Overload, Moment in flexural members at midspan in simply supported and above continuous supports
Vertical Cracking at Midspan (looking through section)	Flexure stresses
Damage	Traffic Induced distress
Delamination	Reinforcement Corrosion, Overload, Foundation Movements
Efflorescence	De-Icing (Chloride) Salt Intrusion
Exudation	Alkali-silica reactivity
Honeycombing	Improper vibration during construction
Pop-out	Freezing of concrete, Sulfate Attack, Alkali-Silica Reactivity, Alkali-Carbonate Reactivity
Reinforcement Corrosion	Carbonation, De-icing Salts, Water
Rust Staining	Reinforcement Corrosion
Scaling	Freeze-Thaw, Sulfate Attack, Improper concrete mix
Spalling	Reinforcement Corrosion, Overload, Foundation Movements
Wear	Traffic Induced distress

**Table B. Cracking Indications in Concrete**

Description	Possibly accompanied by	Crack due to:	Age of Structure
<b>Reinforced Concrete</b>			
Cracking generally following the top layer reinforcement orientation undulating over the top layer of reinforcement		Plastic settlement	Early
Isolated and shallow cracking		Internal restraint of expansion due to hydration reaction	Early
Deep cracking through most of the concrete section		External restraint of expansion due to hydration reaction	Early
Cracking occurring at changes in section		Plastic settlement	Early
Wide cracking, isolated, within a few hours after placement		Plastic shrinkage	Early
Pattern/Map cracking with cracks radiating from individual points	Disintegration of the concrete (the matrix first disintegrates and aggregate follows)	Sulphate attack	Late
Pattern/Map cracking with cracks radiating from individual points, indicating swelling of concrete	Lack of A-S gel	Alkali-carbonate reaction	Late
Pattern/Map cracking with cracks radiating from individual points, indicating swelling of concrete	Exudation of A-S gel	Alkali-silica reaction	Late
Regularly spaced cracking perpendicular to the larger dimensions of the concrete	Thermal changes since construction (winter to spring), ineffective expansion joints	Temperature cracking due to thermal changes (	Late
Spalling/Delamination throughout member	Differential movement of supporting members	Differential movement of supporting members	Late
Scaling, cracking parallel to the surface of the member	Disintegration (possibly extensive) of concrete, possible standing water	Freeze-thaw	Late
Fine Cracks, shallow, few inches in depth, orthogonal and blocky pattern	No indication of movement of supporting members	Drying shrinkage	Late
Cracking generally straight in nature, spaced at the intervals close to that of the underlying reinforcement	Rust-staining, efflorescence	Reinforcement corrosion	Late
Spalling/Delaminations	Rust-staining, efflorescence	Reinforcement corrosion	Late
Deep cracking of members	Indications of differential movement	Cracking due to movement of support structure	All
Spalling, Cracking at Midspan	Possible excessive deflection	Cracking likely due to overload	All
<b>Prestressed/Post-tensioned</b>			
Cracking at midspan and over continuous supports	Excessive deflection	Loss of prestress, damage to prestressing strands	All
Cracking at midspan and over continuous supports	Rust-staining	Corrosion of prestressing strands	All

Early Age - Structures constructed within a few months of insp. Late Age - Structures whose defects are largely caused by their environment  
 (Some portions based upon those indications discussed in the U.S. Army Corps of Engineers *Evaluation and Repair of Concrete Structures*, 1995 and Kay, T., *Assessment & Renovation of Concrete Structures*, Longman Group UK Limited 1992, pgs. 26-28)

**Table C. Concrete Testing Techniques**

External behavior	Test	Standard Test	Comment
Crack Location/Growth	Acoustic Emission <sup>*</sup>	ASTM E 650, ASTM E 569	Signal processing is extensive due to inhomogeneity of concrete. Multiple transducers needed for complete bridge analysis.
Cracks Associated with Delaminations	Hammer Sounding		Simple, effective method for smaller regions
	Chain Drag	ASTM D 4580	Simple, effective method for large regions
	Infrared Thermography	ASTM D 4788	Expensive. Trained operator. Yields various data regarding internal state (versatile)
	Impact Echo <sup>*</sup>		Access to one side needed. Experienced operator req'd. Members less than 6 ft thick. Gives 1-D view of defect. 2-D possible if proper documentation of position on member is kept.
	Ultrasonic Pulse Velocity <sup>*</sup>	ASTM C 597	Need access to both sides of member
	Remote Viewing		Requires access hole
	Intrusive Probing/Exploratory Removal		Slightly destructive
Deflections/Movements	Monitor movements <sup>*</sup>	ACI 437R	
Fire Damage	Petrographic Analysis <sup>*</sup>		Trained individual needed to perform examination.
	Rebound (Schmidt) Hammer	ASTM C 805	Correlation of rebound numbers to concrete strength needed (i.e., core compressive tests needed for the correlation)
	Spectral Analysis of Surface Waves		Data interpretation may be extensive. Capable of delineating the stiffness profile of the member.
	Ultrasonic Pulse Velocity		Need access to both sides of member
	Impact-Echo <sup>*</sup>		Access to one side needed. Experienced operator req'd. Members less than 6 ft thick. Gives 1-D view of defect. 2-D possible if proper documentation of position on member is kept.
	Impulse Response		Data interpretation may be extensive
Freeze-Thaw Damage		ASTM C 856, ASTM C 457, ASTM C 294, ASTM C 295	
	Petrographic Analysis <sup>*</sup>		Trained individual needed to perform examination.
	Spectral Analysis of Surface Waves		Data interpretation may be extensive. Gives an indication of changed stiffness profile
	Impulse Response		Data interpretation may be extensive. Gives general condition of member
General Cracking Extent	Ultrasonic Pulse Velocity <sup>*</sup>		Need access to both sides of member
	Impact-Echo <sup>*</sup>		Access to one side needed. Experienced operator req'd. Members less than 6 ft thick.
	Infrared Thermography	ASTM D 4788	Expensive. Trained operator. Yields various data regarding internal state (versatile)
	Petrographic Analysis	ASTM C 856, ASTM C 457, ASTM C 294, ASTM C 295	Trained individual needed to perform examination.
Leakage	Visual Inspection <sup>*</sup>		Easiest means of identifying leakage.
	Infrared Thermography	ASTM D 4788	Expensive. Trained operator. Yields various data regarding internal state (versatile)
Scaling		ASTM C 856, ASTM C 457, ASTM C 294, ASTM C 295	
	Petrographic Analysis <sup>*</sup>		Capable of determining depth of scaling cracking
	Infrared Thermography	ASTM D 4788	For determining flaking extent due to scaling
	Visual Inspection/Measurement <sup>*</sup>		Determine the depth of scaled areas
Temperature/Moisture conditions	Thermocouple <sup>*</sup>		
	Thermometer		
Wear	Visual Inspection/Measurement <sup>*</sup>		Determine the depth of wear from as-constructed state

**Most Appropriate Method(s)**

**Table C. Concrete Testing Techniques  
(continued)**

Internal Problem	Test	Standard Test	Comment
Air Void System in concrete	Void Determination	ASTM C 457	Lab Test
	Petrographic Analysis*	ASTM C 856, ASTM C 457, ASTM C 294, ASTM C 295	Trained individual needed to perform examination.
	Visual Inspection*		
(Piles, deep, and long members)	Impact Echo*		Access to one side needed. Experienced operator req'd. Members less than 6 ft thick. Gives 1-D view of defect. 2-D possible if proper documentation of position on member is kept
	Ground Penetrating Radar	ASTM D 4748	Fast, Expensive, Experienced operator required
	Intrusive probing/Exploratory Removal		Slightly destructive
	Cross-Hole Sonic Logging		Tubes or drilled holes required for examination and testing
	Impedance Logging		Requires very good test data.
	Parallel Seismic		Boreholes needed next to foundation
	Time Domain Reflectometry*		Requires pre-placement of monitoring wire within new construction for high quality analysis. For constructed elements, a monitoring wire must be positioned alongside member, quality of analysis data drops.
	Gamma-Gamma Logging		Access tubes or drilling required
	Sonic Echo*		Determines depth of piles
	Impulse Response		Determines depth of piles
(Post-tensioned ducts)	Linear Polarization*	SHRP S 324, SHRP S 330	Determines corrosion rate
Delaminations/Voids/Honeycombing	Hammer Sounding*		Simple, effective method for smaller regions
	Chain Drag*	ASTM D 4580	Simple, effective method for large regions
	Impact Echo*		Access to one side needed. Experienced operator req'd. Members less than 6 ft thick.
	Ultrasonic Pulse Velocity		Need access to both sides of member
	Intrusive Probing/Exploratory Removal		Slightly destructive
	Remote Viewing		Requires access holes
	Infrared Thermography*	ASTM D 4788	Expensive. Trained operator. Yields various data regarding internal state (versatile)
	Impulse Response		Data interpretation may be extensive
	Radiography		Requires specially trained operator, expensive
	Ground Penetrating Radar*		Expensive equipment. Experienced operator required. Gives indications of internal defects: embedded metals, voids, honeycombs, delaminations (when moisture is present), etc.
	Ultrasonic Pulse Echo		Gives 1-D view of internal defect. 2-D view possible with proper documentation of locations.
	Spectral Analysis of Surface Waves		Good for layered configurations, Data interpretation may be extensive. Determines stiffness profile through thickness.
	Impact Echo*		Access to one side needed. Experienced operator req'd. Members less than 6 ft thick. Gives 1-D view of defect. 2-D possible if proper documentation of position on member is kept
	Radiography		Proper orientation required to gain needed information. The technique is an x-ray and the position of the beam dictates the view that will be seen on the radiographic film.

Most Appropriate Method(s)

**Table C. Concrete Testing Techniques  
(continued)**

Internal Problem	Test	Standard Test	Comment
Deterioration of post-tensioned systems	Impact Echo*		Access to one side needed. Experienced operator req'd. Members less than 6 ft thick.
	Radiography		Requires specially trained operator, expensive
Deterioration of pre/post-tensioned strands	Magnetic Flux Leakage*		Expensive equipment. Possible subjective data interpretation.
	Radiography		Requires specially trained operator, expensive
Nonuniformity	Petrographic Analysis*	ASTM C 856, ASTM C 457, ASTM C 294, ASTM C 295	Trained individual needed to perform examination.
	Ultrasonic Pulse Velocity	ASTM C 597	Need access to both sides of member
	Windsor Probe	ASTM C 803	Gives relative changes between different zones
	Rebound (Schmidt) Hammer	ASTM C 805	Gives relative changes between different concrete surface condition areas.
	Core Sampling	ASTM C 42	Destructive
Steel Area Reduction	Ultrasonic thickness gage*		Need direct steel contact
	Intrusive Probing/Exploratory Removal		Slightly destructive.
	Radiography		Requires specially trained operator, expensive
Reinforcement Condition/Location	Covermeter (Pachometer)*		Possibly time consuming if large areas are to be analyzed. Inexpensive. Portable.
	Ground Penetrating Radar	ASTM D 4748	Fast, Expensive, Experienced operator required, cannot evaluate condition effectively, gives good indication of location
	Radiography		Requires specially trained operator, expensive
	Magnetic Flux Leakage*		Expensive equipment. Possible subjective data interpretation.
	Time Domain Reflectometry		Requires monitoring wire to be pre-placed in new construction. Pre-placed monitoring wires provide much better data than external position monitoring wires.
	Intrusive Probing/Exploratory Removal		Slightly destructive.

Most Appropriate Method(s)

**Table C. Concrete Testing Techniques  
(continued)**

Chemical Make-up	Test	Standard Test	Comment
Alkali-Silica Reactivity	Petrographic analysis*	ASTM C 856	Trained individual needed to perform.
	Uranyl (uranium) acetate fluorescence	SHRP C 315	Requires protection from Uranyl acetate solution
Carbonation Depth	Phenolphthalein Solution		Need fresh concrete surface to expose to phenolphthalein
	pH meter		Need fresh concrete surface to expose to phenolphthalein
	Litmus paper		Need fresh concrete surface to expose to phenolphthalein
	Petrographic analysis*	ASTM C 856	Trained individual needed to perform test. Yields a variety of data on internal condition/mechanisms of concrete core.
Chloride Content	Acid-Soluble Chloride Determination	ASTM C 1152	Lab test of field specimens
	Water-Soluble Chloride Determination*	ASTM C 1218	Lab test of field specimens
	Specific Ion Probe for Chloride	SHRP S 328	Field test for chloride content
Electro-Chemical Activity	Half-cell potential*	ASTM C 876	Yields areas of varying degrees of corrosion potential
	AC resistance (using 4-probe resistance meter)*		In-situ test of resistivity
	SHRP surface resistance test	SHRP S 327	Effective in sealer evaluation
	Electrical resistivity	ASTM D 3633	Used for membrane pavement system

Most Appropriate Method(s)

**Table C. Concrete Testing Techniques  
(continued)**

Material/Mechanical Properties	Test	Standard Test	Comment
Abrasion Resistance	Abrasion resistance determination by sandblasting <sup>*</sup>	ASTM C 418	Sandblaster must be obtained
Air permeability	SHRP surface airflow method <sup>*</sup>	SHRP S 329	In-situ permeability test
	Schonlin		In-situ permeability test
	Figg air-permeability test		In-situ permeability test
Bond Quality/Strength at Interface of Topping	Pull-off testing	ASTM D4541, ACI 503R	Setup of apparatus may be time consuming
	Impact Echo <sup>*</sup>		Access to one side needed. Experienced operator req'd. Members less than 6 ft thick.
	Spectral Analysis of Surface Waves		Good for layered configurations, Data interpretation may be extensive
	Impulse Response		Data interpretation may be extensive
Cement Content	Cement Content Determination <sup>*</sup>	ASTM C 1084	Sampling required
Compressive Strength	Core sampling <sup>*</sup>	ASTM C 42, ASTM C 39	Slightly destructive
	Windsor probe	ASTM C 803	Not as accurate as core sampling, calibration needed between probe values and actual comp. Strength
Concrete Quality  (Piles, deep, and long members) (Piles, deep, and long members) (Piles, deep, and long members) (Piles, deep, and long members)	Ultrasonic Pulse Velocity <sup>*</sup>	ASTM C 597	Need access to both sides of member
	Rebound (Schmidt) hammer	ASTM C 805	Relationship between strength and rebound number must be determined
	Cross-Hole Sonic Logging		Requires access tubes.
	Parallel Seismic		Requires bore hole into pile/foundations
	Gamma-gamma logging		Requires access tube placement.
	Impact Echo <sup>*</sup>		Gives indication of depth and condition of pile. No tubes placement required.
Density	Specific Gravity of Samples <sup>*</sup>	ASTM C 642	Lab test of field gathered samples
	Back Scatter Radiometry		Need access to one side of member. Less accurate than Direct Transmission
	Direct Transmission Radiometry		Need access to inside of member and opposite faces. Licensed operator
Dynamic Modulus of Elasticity	Resonant frequency of sawed specimens	ASTM C 215	Invasive, samples must be taken
	Ultrasonic pulse velocity <sup>*</sup>	ASTM C 597	Involves interpretation of results to obtain dynamic modulus
	Impact Echo		Involves interpretation of results to obtain dynamic modulus
	Spectral Analysis of Surface Waves		Involves interpretation of results to obtain dynamic modulus
Flexural Strength	Third point loading <sup>*</sup>	ASTM C 78	Invasive, cores must be taken
	Center point loading <sup>*</sup>	ASTM C 293	Invasive, cores must be taken
Freeze-Thaw Resistance	Freeze-Thaw Resistance Determination <sup>*</sup>	ASTM C 666, ASTM C 671	Lab test of field gathered samples
Local or Global Strength	Load Test, deflection, and strain measurements <sup>*</sup>		
	Acceleration, Strain, and displacement measurements		
Moisture Content	Moisture Meters <sup>*</sup>		

**Most Appropriate Method(s)**

**Table C. Concrete Testing Techniques  
(continued)**

Material/Mechanical Properties	Test	Standard Test	Comment
Moisture Content	Back Scatter Radiometry		Need access to one side of member. Less accurate than Direct Transmission
	Direct Transmission Radiometry		Need access to inside of member and opposite faces. Licensed operator
Repair Evaluation (Injection of cracks and voids)	Ultrasonic pulse velocity*		Need access to both sides of member
	Impact Echo Impact Echo*		Access to one side needed. Experienced operator req'd. Members less than 6 ft thick.
	Spectral Analysis of Surface Waves		Good for layer configurations, Data interpretation may be extensive
Resistance to Chloride Penetration	90-day ponding test	AASHTO T 259	Length of total test time may be 4 months
	Chloride Ion Resistance* (Rapid Permeability Test)	ASTM 1202, AASHTO T 277	Total test time is 30 hrs, not as accurate as 90-day ponding
Resistance to Deicing Salts	Deicing Salt Resistance determination*	ASTM C 672	Lab Test of field gathered materials
Shrinkage/Expansion	Shrinkage/Expansion length change of drilled specimens*	ASTM C 215	Invasive, cores must be taken
Static Modulus of Elasticity	Core compression test*	ASTM C 469	Invasive, cores must be taken
Tensile Strength	Splitting tensile test*	ASTM C 496	Invasive, cores must be taken
	Pull-off testing	ASTM D 4541, ACI 503R	In-situ testing, some setup time req'd. Better suited for determining tensile strength of overlays
Water Absorption	Initial surface-absorption test (ISAT)		In-situ absorption test
	Figg water-absorption test*		In-situ absorption test
	Covercrete-absorption test		In-situ absorption test
Water permeability	Clam		In-situ permeability test
	Steinert*		In-situ permeability test

\* Most Appropriate Method(s)

## **Appendix B. Steel Problems and Associated NDE Techniques**

The following section details the steel problems and NDE techniques discussed in the earlier part of this report. The information has been tabularized for easy reference. There are two tables given in this section as follows:

### **Table A. Overview of General Steel Problems and Causes**

This table discusses the general problems encountered in steel structures and relates them to their specific causes.

### **Table B. Steel Testing Techniques**

This table lists the available steel testing techniques. Appropriate cross-references to this report and references to standard tests are given.

**Table A. Overview of General Steel Problems and Causes**

<b>Problem</b>	<b>Possible Causes</b>
Corrosion	
Crevise Corrosion	Confining Geometry, Exposure to salt spray accelerates corrosion process, Ineffective expansion joints, Lack of cleaning (Maintenance)
Deposit Attack	Foreign materials: bird droppings and nests, coal dust in mining areas, grain dust in farming regions, deicing salts
Erosion Corrosion	Flow of abrasive particles over a steel surface (this is not a common deterioration mechanism)
Fretting Corrosion	Lack of lubrication or ineffective lubrication between two steel surfaces under load
Galvanic Corrosion	Attachment of aluminum fixtures to a bridge, contact between galvanized and non-galvanized steels
Pitting Corrosion	Foreign materials like bird droppings, dirt, and deicing salts in conjunction with paint scratches and defects
Stress Corrosion Cracking	Aggressive environment: highly stressed areas and a corrosive environment
Underfilm Corrosion	Breaks and defects in painted surface
Rust Staining	Corrosion of steel likely below painted area
Weld Decay	Field welded areas on bridge structures
Cracking	
Web Cracking near connections	Fatigue/Out-of-plane distortion/Improper details
Flange Cracking (Cover plates)	Fatigue Prone detail
Flange Cracking (No Cover plates)	Intersecting welds between butt welded flange and flange to web weld
Flange Cracking (No Cover plates)	Highly stressed, intersecting welds at connections
Damage	
Misalignment out of plane of member	Impact by over height vehicles, fire
Movement	
Excessive deflection	Overload, Fire damage, Ineffective expansion joints, Inadequate design
Misalignment of supports	Settlement
Misalignment of member on supports	Impact by over height vehicles

(The corrosion portion of this table is based upon information from *National Cooperative Highway Research Program Report 333*, 1990)

**Table B. Steel Testing Techniques**

External behavior	Test	Standard Test	Comment
Cracks	Acoustic Emission*	ASTM E 1106, ASTM E 650, ASTM E 569	Monitors local areas well. Global monitoring requires extensive sensor placement. Experienced operator needed, expensive, time-consuming.
	Ultrasonic Pulse Catch	ASTM A577, ASTM A435, ASTM E587, ASTM A898	Access to only one face needed. Experienced operator needed
	Ultrasonic Through Method*	ASTM A577, ASTM A435, ASTM E587, ASTM A898	Requires access to both sides of the element. Gives no indication of defect depth. Rarely used.
	Magnetic Particle	ASTM E 709	Best method for ferromagnetic material, fast, easy to use, can be used through thin-placed surfaces. Little prep. time is required. Cannot detect subsurface flaws.
	Dye Penetrant	ASTM E 165 AASHTO AWS D1.1	Simple, Inexpensive, Not capable of detecting subsurface flaws
	Visual Inspection		
	Coating Tolerance Thermography		Expensive equipment. Time required to heat from both directions during testing (Rarely used)
	Radiographic Testing	ASTM E 94, ASTM E 1955	Requires access to both sides of the element. Gives no indication of defect depth. Relatively inexpensive <\$100/hour.
	Computed Tomography	ASTM E1570	Expensive equipment. Can give indications of conditions through cross-section (Rarely used)
Corrosion	Visual Inspection/Remote Monitoring*		
	Section loss monitoring		Measurement from as-constructed conditions
	Coating Tolerance Thermography		Capable of identifying corrosion below paint. Expensive equipment. Time required to heat from both directions during testing. (Rarely Used)
Damage/Misalignment	Visual Inspection*		
	Measurement/Surveying*		Monitoring to determine changes from as-built condition

\* Most Appropriate Method(s)

## Appendix C. Index of NDE Test Methods

### A. Test Methods for Concrete Structures

The following is a description of some of the test procedures mentioned previously in the guidelines as they apply to concrete bridge components.

#### 1. 90-day Ponding

This test measures a concrete surface's resistance to chloride intrusion. The test entails ponding a 3% sodium chloride solution on the surface of a core or small slab for a period of 90 days. The concrete is then sampled at various depths with the chloride ion content determined by lab tests. The total test time including lab tests may reach close to 4 months.

**Relevant References:** AASHTO T-259, AASHTO T-260

#### 2. Abrasion Resistance

This is a lab test to determine relative resistance to wear. The test is meant to simulate traffic wear on a concrete surface. A sandblaster is used to provide the rapid abrasive environment. This test requires the sandblaster apparatus and relevant dust collection devices. The amount of material removed from the specimen under the spray is determined by filling the holes with modeling clay and measuring the amount of clay required to fill the hole. This is largely a technique to compare different concrete mixes' resistance to abrasion.

**Relevant References:** ASTM 418

#### 3. AC Resistance (using 4 probe resistance meter)

This method is capable of determining the resistivity of in-situ concrete surfaces. The rate at which the corrosion reaction can proceed is governed by the amount of oxygen available to the reaction, the alkaline state of the concrete in the area, and the resistivity of the concrete. Electrical resistance depends upon the microstructure of the paste and the moisture content of the concrete. The measurement of the resistivity of the concrete is useful in conjunction with a half-cell potential test. The test setup uses four equally spaced electrodes placed in drilled holes. The outer electrodes are connected to an alternating current. The inner electrodes are connected to a voltmeter. The apparent resistivity,  $\rho$ , is given by the following expression:

$$\rho = \frac{2 \pi s V}{I}$$

Where:  $s$  = spacing of probes,

$V$  = measured voltage between inner electrodes, and

$I$  = current between outer electrodes

A spacing of 2" (50mm) for the probes is commonly considered to be efficient for most concrete mixtures. Currently, a distinct relationship between concrete resistivity and corrosion risk does not exist. However, a high resistivity may give an indication of a very slow corrosion rate if the half-cell results show corrosion.

**Relevant References:** <sup>(64)</sup>

#### 4. Acid Soluble Chloride Determination

Acid soluble chloride determination is a laboratory procedure used to determine the chloride content of field gathered specimens. This test determines the amount of chloride soluble to acid.

**Relevant References:** ASTM C 1152, ACI 222R-96

## 5. Acoustic Emission

The Acoustic Emission technique is a monitoring technique. Acoustic emission, by definition, is the generation of transient elastic waves during the rapid release of energy from localized sources in the material. This wave propagates spherically from the initial location eventually reaching the outer surface. The waves, or emissions, are detected and converted into electrical signals. The detection of these signals occurs through the coupling of several piezoelectric transducers located on surfaces throughout the structure. Parameters such as crack type, orientation, and energy can be found theoretically and compared with test data. This technique is more common in steel structures, but research and corresponding data analysis techniques are being developed for concrete structures.

**Relevant References:** ASTM E 650, ASTM E 569 (these are with regard to sensor placement)

## 6. Back Scatter Radiometry

Backscatter Radiometry can be classified as a nuclear method. Radiation is introduced into the concrete and measurements are taken from the same side. This is known as a radiometric technique. There are two forms of radiometric techniques, differentiated by the way the gamma rays are measured. Backscatter Radiometry measures the reflected gamma rays.

The backscatter techniques are suitable for applications where a large number of measurements are required. Backscatter measurements are affected by the top ½" (40mm) to 4" (100 mm), so this method is best suited for measurement of the surface zone of a concrete element such as monitoring the density of bridge deck overlays. The radiation source and detector are placed on the same side of the sample. The difference between this method and the direct transmission method is that the detector receives gamma rays scattered within the concrete rather than those that pass through the concrete. The scattered rays are lower energy than the transmitted rays and are produced when a photon collides with an electron in an atom. Part of the photon energy is imparted to the electron, and a new photon emerges, traveling in a new direction with lower energy. The amount of scattering is based on the concrete density.

A calibration curve is needed prior to testing using backscatter techniques. This technique does not require hole drilling.

**Relevant References:** ACI 228.2R-98 pg.17, ASTM C 1040

## 7. Break-off Test

A Break-Off (B.O.) test is performed to estimate the in-place concrete strength. The in-place concrete is placed, compacted, and cured in a different manner than the concrete cylinder specimen. Determination of accurate in-place strength is critical in form removal and prestressed or post-tensioned force release operations.

The B.O. test consists of breaking off an in-place cylindrical concrete specimen at a failure plane parallel to the finished surface of the concrete element. The break-off stress at failure can then be related to the compressive or flexural strength of the concrete using a predetermined relationship that relates the concrete strength to the break-off strength for a particular source of concrete.

The test specimen is created by means of a disposable tubular plastic sleeve, which is cast into the fresh concrete and then removed at the planned time of testing, or by drilling the hardened concrete at the time of the B.O. test. The test specimen has a 55mm (2.17 in.) diameter and 70mm (2.76 in.) height.

The equipment is safe and simple, and the test is fast to perform, requiring only one exposed surface. The B.O. test does not always need to be planned in advance of placing the concrete because drilled B.O. test specimens can be obtained.

**Relevant References:** ACI 503R, ASTM C1150

## **8. Cement Content Determination**

The standard test designation covers the determination of Portland-cement content of a sample of hardened concrete. This is a laboratory test of field gathered specimens.

**Relevant References:** ASTM C 1084

## **9. Center Point Loading**

The standard test designation covers the determination of flexural strength of a sample concrete beam. The sample could be gathered from the field, if possible, or the lab could mix and cure a comparable sample to field conditions.

**Relevant References:** ASTM C 293

## **10. Chain Drag**

The chain drag method is a simple and inexpensive means of determining delaminated areas on horizontal concrete surfaces. The change in sounds from one region to another gives an indication of the delaminated region. The chain drag method is a faster and larger version of the hammer sounding method.

**Relevant References:** ASTM D 4580

## **11. Chloride Ion Resistance (Rapid permeability test)**

This test method is based on the 90-day ponding test. The test entails sampling a core or forming a 4" diameter cylinder in the lab. The sample's top surface is maintained in a 3% chloride solution, like the 90-day ponding test. A 60-volt DC electrical current is placed across the samples length and the specimen is monitored for a 6-hour period. The chloride ions migrate toward the end not in the solution. The current is recorded every 30 minutes and the data is plotted versus time. The area under the curve is a measure of the chloride permeability.

**Relevant References:** AASHTO T 277, ASTM 1202

## **12. Clam Test**

This method involves measuring the flow of water into the concrete surface under a fixed pressure. A specially designed cap is glued to the concrete surface and a micrometer-screw piston provides pressurized water. A pressure gauge in the chamber measures the water pressure. To perform the test, the chamber is filled with water, the micrometer screw is turned so as to maintain a constant water pressure of about 22psi (150 kPa) above atmospheric pressure, and the movement of the piston is recorded at periodic intervals for a 20 to 30-minute period. The area of the cylinder multiplied by the distance traveled by the piston gives the volume of water that penetrates the concrete. The concrete surface is typically damaged from this procedure.

**Relevant References:** ACI 228.2R-98

## **13. Computed Tomography**

Computer Tomography is a radiographic method that utilizes computer imaging to reconstruct a view of a cross-sectional plane of an object. This can utilize X-ray and gamma radiation to image the interior defects of an object. Computer Tomography uses a computer to reconstruct the image from the penetrating radiation exiting on the opposite side into the detector-array by taking information from many locations. This method is similar to medical CAT scans.

#### **14. Core Sampling**

Core sampling is a tried and true method of determining concrete properties. The sampling of cores and concrete beams and the testing of these specimens to determine concrete strengths are covered in ASTM standards.

**Relevant References:** ASTM C 42 (sampling), ASTM C 39 (compressive testing), ASTM C 469 (modulus, Poisson's ratio determination)

#### **15. Covercrete Absorption Test**

This test is designed to account for absorption throughout an 2" (50mm) deep hole. There is a cap with a gasket placed over the hole with a tube passing through it and it is connected to a reservoir. The cap contains a second tube connected to a horizontal capillary to measure absorption from 8" (200mm) above the hole. The tube to the reservoir is shut off and the movement of the meniscus in the capillary is measured between 10 and 11 minutes after initial contact with water. The technique is moisture sensitive and drilling is required.

**Relevant References:** ACI 228.2R-98 pg. 32

#### **16. Covermeter (Pachometer)**

Covermeters are magnetic devices that are used to determine location of reinforcement and, if the size of the reinforcement is known, the concrete cover. These devices can typically measure concrete cover with; 1/4" at 0 to 3" from the surface. Calibration of the Pachometer may be needed in situations where metallic particles and additives may influence the readings. Excavating the concrete, measuring the cover, and calibrating the machine achieve this calibration.

**Relevant References:** ACI 228.2R-98 pg. 28

#### **17. Cross-Hole Sonic Logging**

Cross-hole sonic logging is a method used to determine condition of deep, long concrete members. This method requires placement of either metal or plastic tubes or drilling of core holes after the concrete has set. This technique is an offshoot of ultrasonic pulse velocity. A transmitter is fed down one tube and a receiver is fed down the other. Care is taken that the transmitter and receiver are located at the same depths as the two are fed down the tubes as measurements are taken every 1/2" (10mm) to 2" (50mm). The ultrasonic pulse velocity is related to the density and dynamic elastic constants of the concrete. At regions where the speed of pulse is slower, the concrete is either damaged, or a void likely exists. The negative aspects of this technique are the need for drilling holes and lack of information obtained outside the ring of tubes.

**Relevant References:** ACI 228.2R-98 pg. 15

#### **18. Deicing Salt Resistance Determination**

This is a laboratory testing technique that is used largely to determine the effect of mix parameters on resistance to scaling from deicing salt exposure. This testing technique requires the use of a freeze-thaw refrigeration apparatus.

**Relevant References:** ASTM C 672

#### **19. Direct Transmission Radiometry**

Direct Transmission Radiometry is classified as a nuclear method. Radiation is passed through the concrete and measured from the other side. There are two forms of radiometric techniques, differentiated by the way the gamma rays are measured. Direct Transmission Radiometry introduces a radioactive source on one side of the member and measures the intensity of the gamma rays that have passed through the member.

This technique is well suited for heavyweight concrete or roller compacted concrete.

In direct transmission radiometry, the source is located on one side of the member and the detector on the other. When radiation passes through the concrete, a portion scatters by free electrons, a smaller amount is absorbed by atoms. The amount of scattering depends on the concrete density. The amount of absorption depends on the chemical makeup of the concrete. A calibration curve is needed for the instrument. This is done by making test specimens of different densities and determining the output and developing the curve. The technique is capable of determining the in-place density of concrete members.

**Relevant References:** ACI 228.2R-98, ASTM C 1040

## **20. Electrical Resistivity**

The methods listed are useful in sealer or membrane evaluation. Tests are generally inexpensive and rapid. Environmental effects can disturb testing procedures and results. These techniques require a dry surface. Environmental effects can disturb the testing procedures and results.

**Relevant References:** SHRP S 327, ASTM D 3633

## **21. Figg Air- Permeability Test**

This test uses the same setup as the Figg water absorption test, except that a vacuum pump is used in place of a water source. The hole is pressurized to a prescribed value and the valve is closed. The time the hole takes to reach a given pressure value is an indication of the air permeability of the concrete.

**Relevant References:** ACI 228.2R-98 pg. 33

## **22. Figg Water Absorption Test**

This test consists of drilling a 5.5mm diameter hole 30mm deep. The hole is cleaned and a foam insert is pushed 20mm below the surface. The rest of the hole is sealed with silicone rubber. A needle is inserted into the hole with a series of capillaries connected. These capillaries are set up to achieve a water head of 100mm. The time needed to travel 50mm on a horizontal capillary is measured in seconds. This value is the absorption index.

**Relevant References:** ACI 228.2R-98 pg. 32

## **23. Freeze-Thaw Resistance Determination**

The referenced techniques are laboratory techniques utilized to determine concrete's resistance to freezing. The first reference is a more exploratory technique into the effects of different mixes on freeze-thaw durability. The second discusses the critical dilation point, the point at which concrete can no longer resist freezing.

**Relevant References:** ASTM C 666, ASTM C 671

## **24. Gamma-gamma Logging**

Gamma-gamma logging is similar to cross-hole sonic logging. The main difference being the source used to gather information. In gamma-gamma logging, a radioactive source is introduced. This method is typically used in deep foundation scenarios. As is the case in cross-hole sonic logging, tubes are needed to send the instrumentation down the member. This method is capable of determining the integrity of piles and large foundations. This method requires 2" diameter pre-placed tube. The results are available only for those areas directly across from one another.

**Relevant References:** ACI 228.2R-98 pg. 20

## **25. Ground Penetrating Radar**

Radio Detection and Ranging (RADAR) uses electromagnetic waves (radio waves or microwaves) to determine concrete thickness, locating voids and reinforcing bars,

and identifying deterioration. The most attractive features are: its ability to penetrate into the subsurface and detect unseen conditions; its ability to scan large surface areas in a short period of time; and its high sensitivity to subsurface moisture and embedded metal.

This method uses an antenna that is either dragged or attached to a survey vehicle that transmits short pulses of electromagnetic energy into the surveyed material. A portion of the energy is reflected back to the antenna when an interface between materials of dissimilar dielectric properties is encountered. The antenna then generates an output signal that contains information on what was reflected, how quickly the signal traveled, and how much of the signal was attenuated.

This is a fast but expensive technique. The radar is emitted in a cone shape and the area tested is therefore limited to this shape also.

**Relevant References:** ASTM D4748, ASTM D6432, and ASTM D6087

## **26. Half-Cell Potential**

Half-Cell Potential method is an electrical method that evaluates the corrosion activity taking place in the steel reinforcement. As corrosion takes place there is a flow of electrons and ions. At active sites on the bar, called anodes, iron atoms lose electrons and move into the surrounding concrete as ferrous ions. This is called half-cell oxidation reaction. When the remaining electrons in the bar combine with the water and oxygen in the concrete, they form rust. When there is no flow of electrons or ions, the bar is not corroding. There is a negative charge left in the bar after the ferrous ions move to the concrete. Half-cell potential method detects the negative charge and thus provides an indication of corrosion activity. This method is not applicable to epoxy-coated bars. The equipment is lightweight and portable. Corrosion is likely when the potentials are less than  $-350\text{mV}$ .

**Relevant References:** ASTM C 876

## **27. Hammer Sounding**

Hammer sounding is simple and inexpensive means of determining delaminated areas. The change in sounds from one region to another gives an indication of a deteriorated region.

## **28. Impact Echo**

A mechanical impact introduces a stress pulse into the member. It is used to test concrete structural members. It helps determine flaws in plate-like structural members, beams, columns, and hollow cylindrical members. It also measures crack depth.

Impact Echo uses P and S waves as follows:

P-waves are dilatational or compression waves. They are associated with the propagation of normal stress and particle motion parallel to propagation direction.

S-waves are shear or transverse waves. They are associated with shear stress and particle motion is perpendicular to the propagation direction.

The P and S waves propagate into the object along hemispherical wave fronts. The waves travel away from the point of impact along the surface. The waves are reflected by internal interfaces or external boundaries. When the echoes return to the surface, a transducer with a data-acquisition system records their displacements. For thin members, a frequency analysis of displacement is quicker and easier than a time-domain analysis. Frequency analysis is merely a stress pulse generated by an impact that undergoes multiple reflections between test surface and the reflecting interface (flaw or boundaries).

**Relevant References:** ASTM C1383, ACI 228.2R-98 pg. 8

## 29. Impedance Logging

This technique is typically utilized for testing of piles. Impedance logging combines two different techniques, sonic echo and impulse response. A mechanical impact is introduced at the top region of the pile. As the stress wave travels through the pile, it is affected by the geometry or condition of the pile itself. These changes are reflected back to the top of the shaft. Data is gathered and analyzed, relating the data from the sonic echo test with the impulse response test.

**Relevant References:** ACI 228.2R-98 pg. 14

## 30. Impulse Response

In the impulse response technique, a controlled force vibration is introduced into the top of a pile. The vertical response of the shaft is recorded by geophone velocity transducers and compared to the continuously monitored introduced vibration. The plot of geophone particle velocity divided by vibrator force versus frequency gives an indication of shaft integrity.

**Relevant References:** ACI 228.2R-98 pg. 13

## 31. Infrared Thermography

Infrared Thermography is based on the principle that subsurface anomalies in a material affect heat flow through that material. Localized differences in surface temperature are caused by these changes in heat flow. Thus, by measuring surface temperature under heat flow, one can determine the location of the subsurface anomalies.

It is based on two principles:

A surface emits energy in the form of electromagnetic radiation.

Non-uniformity in concrete affects heat flow through the concrete.

The first principle is based on the Stefan-Boltzmann law,  $R = e\sigma T^4$  where:

$R$  = rate of energy radiation per unit area of surface,  $W/m^2$  ;

$e$  = the emissivity of the surface;

$\sigma$  = the Stefan-Boltzmann constant,  $5.67 \times 10^{-8} W/m^2 K^4$

$T$  = absolute temperature of the surface K.

The wavelength of the emitted radiation depends on the temperature. As temperature increases, the wavelength becomes shorter. Sensors that produce electrical signals in proportion to the amount of incident radiant energy detect the radiation. The output of the infrared sensor can then be converted to temperature.

The second principle tells us that anomalies associated with poor concrete, namely, voids and low density, decrease the thermal conductivity of the concrete by reducing energy conduction properties without substantially increasing the convection effects.

This method exploits two main heat transfer mechanisms: conduction and radiation. Sound concrete is more thermally conductive than low density or cracked concrete. The best times to test are shortly after sunrise or sunset. This is recommended because heat flow inward to the test specimen's surface will show up as hot zones above flaws and will show up as cold zones when the heat flow is outward of the specimen.

Infrared thermography has been used for detecting subsurface anomalies within and below concrete elements. This method has been applied to the identification of internal voids, delaminations, and cracks in concrete structures such as bridge decks. This type of evaluation can be used with and without asphalt overlays.

Advantages of infrared thermography are that it allows large areas to be tested rather quickly, equipment emits no radiation, and it is an area testing technique which gives a two dimensional image of the test surface instead of a point or line testing method.

One major disadvantage is that the depth or thickness of a void cannot be determined. We are unable to tell if the anomaly is near the surface or down in the area of the reinforcing bars.

When used with other types of testing, infrared thermography is an efficient way to quickly check for flaws in the concrete. Other methods such as ground penetrating radar can be used to find out flaw depth and thickness.

**Relevant References:** ASTM D4788, ACI 228.2R-98 pg. 35

### 32. Initial Surface Absorption Test (ISAT)

A circular cap with a minimum surface area of 5,000 mm<sup>2</sup> is sealed to the concrete surface. A reservoir attached to the cap is filled with water so that the water level is 200 mm above the concrete surface. The cap is connected to a horizontal capillary positioned at the same height as the water in the reservoir. At specified intervals (10min, 30min, 1hr, 2hr) from the start of the test, the valve below the reservoir is closed and the rate at which water is absorbed into the concrete is measured by the movement of water in the capillary attached to the cap. This test helps determine the permeability of concrete.

**Relevant References:** ACI 228.2R-98 pg. 31

### 33. Intrusive Probing/Exploratory Removal

Intrusive Probing and Exploratory Removal, though destructive, can yield useful information regarding causes of cracking, extent of cracking, or corrosion.

### 34. Linear Polarization

Using electrolytic test cells, the polarization resistance technique can determine corrosion rate. The technique measures the change in the open-circuit potential of a short-circuited electrolytic cell when an external current is applied to the cell. For a small perturbation about the open circuit potential, a linear relationship exists between the change in voltage and the change in current per unit area of bar surface. This is the polarization resistance,  $R_p$ :

$$R_p = \frac{\Delta E}{\Delta i}$$

Where  $\Delta E$  = Change in voltage,  
 $\Delta i$  = change in current per unit area

There is an established relationship between corrosion rate of the anode and the polarization resistance:

$$i_{\text{corr}} = \frac{B}{R_p}$$

Where  $i_{\text{corr}}$  = corrosion current density in Area (cm<sup>2</sup>),  
 $B$  = a constant based on polarization curves  
 Taken as 0.026V for steel in concrete  
 $R_p$  = Polarization in  $\Omega/\text{cm}^2$

**Relevant References:** ACI 228.2R-98 pg. 28, SHRP S 324, SHRP S 330

### **35. Litmus Paper**

*See phenolphthalein.*

### **36. Magnetic Flux Leakage**

Magnetic Flux Leakage is the measurement of changes in the path of magnetic force lines, or flux, near a ferromagnetic material, such as steel, with discontinuities or defects. In the presence of a magnetic field, ferromagnetic materials align their electric dipoles with the external field. A stronger field yields more aligned dipoles. There is a saturation point where all dipoles are aligned and no further alignment is possible. The amount of the alignment of these dipoles, and subsequently the flux, depends on the intensity of the applied magnetic field. It is therefore possible to increase the intensity of magnetic flux leakage to overcome some of the inherent limitations in terms of sensitivity, signal-to-noise ratio, etc. The applied magnetic field needs to be large enough to overcome problems due to noise, distance between the magnetic source and ferromagnetic field, and the masking effect of the large quantities of steel typically found in prestressed and reinforced concrete members. When an external magnetic field is applied to reinforced or prestressed concrete members, the flux within the steel remains unchanged until it must leave the steel to travel back to the south pole of the magnet. If the flux encounters a flaw, such as corrosion or broken and fractured steel, the flux will “leak” out at these areas and not travel to the south pole of the magnet. The amount of leakage can be measured using Hall-effect sensors. The crystals of the sensors react to the presence of an external magnetic field when excited by developing a voltage difference across the two parallel faces of the crystal, this is known as the “Hall Effect.”

### **37. Moisture Meter**

Moisture meters can determine in-situ moisture content of concrete members. There are several meters commercially available.

### **38. Parallel Seismic**

Parallel Seismic requires the drilling or pre-placement of a small access hole next to the foundation. A receiving probe is placed at the top of the tube. A hammer impact is initiated as close as possible to the top of the foundation and the received signals are collected. The probe is lowered into the borehole and the hammer impacts are repeated. The borehole must be lined with a plastic tube to retain water, which acts as a couplant in this scenario. The technique is capable of showing discontinuities in the foundation or pile at lengths along the member. Once the first anomaly is hit, the technique cannot reach further below it.

**Relevant References:** ACI 228.2R-98 pg. 16

### **39. Petrographic Analysis**

Petrographic examination requires the remove of concrete cores. There are ASTM standards regarding this removal. Petrographic analysis is a detailed examination of concrete that can determine the following deterioration mechanisms: freeze-thaw resistance, sulfate attack, alkali-aggregate reactivity, aggregate durability, and carbonation. Highly specialized personnel, at an increased cost, perform this analysis.

**Relevant References:** ASTM C 856, ASTM C 457, ASTM C 294, ASTM C 295

### **40. pH**

*See phenolphthalein.*

#### 41. Phenolphthalein

Acid base indicators, in the typical case phenolphthalein, change color depending on the pH level of that they are contacting. Phenolphthalein changes to a deep pink or red color when it encounters pH levels greater than 10. The solution remains colorless when in contact with pH levels less than ten. The exact pH value is not determined, only the general indication of the alkaline or acidic nature of the concrete. Phenolphthalein is typically sprayed. The spraying of the concrete surface must be on a fresh surface. These indicators are typically used in conjunction with those testing techniques that remove a concrete sample from the bridge, core sampling and petrographic analysis for example. Litmus paper can also be used to determine the alkalinity of concrete

**Relevant References:** <sup>(65)</sup>

#### 42. Pulloff Test

The pulloff technique requires bonding of an aluminum disc to the surface of the concrete or concrete overlay. The disc is attached to a jacking apparatus that pulls the disc by bracing against the concrete surface. This technique's applicability is more for tensile strength of concrete overlays, but the technique can be used to determine a general indication of the tensile strength of concrete through bonding directly to the concrete surface. *See also: Pullout Test, Break-off Test*

**Relevant References:** ASTM D 4541, ACI 503R

#### 43. Pullout Test

The pullout test measures the force required to pull an embedded metal insert with an enlarged head from a concrete specimen or structure. The measured ultimate pullout load is used to estimate the in-place compressive strength of the concrete. The pullout test is widely used during construction to determine if the concrete is strong enough for form removal, application of post-tensioning, or termination of cold weather protection to proceed. The pullout test subjects the concrete to a slowly applied load and measures an actual strength property of the concrete. However, the concrete is subjected to a complex three-dimensional state of stress, and the pullout strength is not likely to be related simply to uniaxial strength properties. By the use of correlation curves the pullout test can be used to make reliable estimates of in-place strength.

The disadvantage of the pullout test is that it requires embedding the metal insert at the time of construction. As such, it may not be used as a NDE tool for routine inspections of in-place structures.

**Relevant References:** ASTM C900

#### 44. Rebound (Schmidt) Hammer

This test measures the surface hardness by allowing a spring controlled hammer to collide with the surface of the concrete, thus it is considered a rebound test. After collision with the concrete surface, the distance of rebound of the calibrated hammer is measured on a scale and recorded. An empirical correlation has been established between strength and rebound number. The Schmidt hammer weighs about 4 lbs. and is suitable for both laboratory and fieldwork. Surfaces can be tested at any angle but the instrument must be calibrated in that position.

One main limitation of this test is calibrating the hammer. Test cylinders are needed from the day the surface was poured. The rebound number is then taken after the test cylinders provide a well-distributed and reproducible average. Although the Schmidt hammer provides an inexpensive, simple and quick method of obtaining an indication of concrete strength, it only has an accuracy of  $\pm 15\text{-}20\%$  for specimens

that have been cast, cured and tested under conditions for which calibration curves have been established.

**Relevant References:** ASTM C 805

#### **45. Remote Viewing**

Remote viewing requires the use of video equipment (fiber optics, monitors, cameras, etc.) to determine the condition through an access hole. This hole could possibly be the hole used for another technique. This technique provides the capability of viewing the concrete through its section without removing a complete core. The equipment may be somewhat expensive.

#### **46. Resonant Frequency of Sawed Specimens**

This method measures the fundamental transverse, longitudinal, and torsional frequencies of concrete prisms and cylinders for the purpose of calculating dynamic Young's modulus of elasticity, the dynamic modulus of rigidity, and dynamic Poisson's ratio. This is a test of primarily lab prepared specimens.

**Relevant References:** ASTM C 215

#### **47. Radiography**

Radiography provides an image of the interior of concrete by showing the light colored dense areas that appear as radiation passes through the concrete and reinforcement. Voids and grouting of post-tensioning ducts can be identified as dark spots on the photograph because they are unable to block the radiation from special photographic film.

A radiation source is placed on one side of the test specimen and a beam of radiation is emitted. The density affects the beam and the result is shown on the film. Operators must be licensed and highly skilled. Although the X-ray equipment is bulky and expensive, it is a frequently used method.

**Relevant References:** ACI 228.2R-98 pg. 19, ASTM E94

#### **48. Schonlin Test**

This method does not require a hole to be drilled, but rather a 50mm diameter chamber of known volume placed on the surface. A vacuum pump is then used to create a pressure less than -99 kPa. The valve then closes and time is taken between -95 kPa and -70 kPa. Having known the volume we are able to find the permeability index.

**Relevant References:** ACI 228.2R-98 pg. 33

#### **49. SHRP Surface Airflow Method**

The surface airflow method measures the airflow into a vacuum plate placed on a concrete surface. The plate is fitted with a rubber gasket that is held in contact with the concrete surface. This method can be used to determine the relative permeability of the near surface portion of the concrete member. Moisture can affect the results and drying must be done if heavily exposed to rainfall prior to test. The test can also be sensitive to surface roughness.

**Relevant References:** SHRP S 329, SHRP S 330

#### **50. SHRP Surface Resistance Test**

This technique determines the changing electrical resistance between two conductive paint strips on the concrete surface. This is used as an indication of the effectiveness of penetrating sealers applied to the concrete. The paint must be cured with a propane heater or equivalent. The surface is dried for a short time. The test area is wetted for a short time and excess water is removed. The resistance is

measured after 4 minutes and is an indication of water penetration or expulsion. The test can also be sensitive to surface roughness.

**Relevant References:** SHRP S 327, SHRP S 330

### **51. Specific Ion Probe for Chloride**

This method uses a drilled concrete powder sample from the field. The powder is placed in solutions and eventual readings are taken using a chloride specific ion probe. The technique can give an indication of overall total chloride content.

**Relevant References:** SHRP S 328, SHRP S 330

### **52. Specific Gravity of Samples**

The test method is a laboratory determination of density, absorption, and voids in concrete. The test requires drying ovens, and other applicable laboratory testing apparatus.

**Relevant References:** ASTM C 642

### **53. Spectral Analysis of Surface Waves**

This method involves determining the relationship between the wavelength and velocity of surface vibrations as the vibration frequency is varied. Digital signal processing is used to develop the relationship between wavelength and velocity.

An impact is used to generate a surface wave (R-wave) and two receivers are used to monitor the motion as the R-wave propagates along the surface. The received signals are sent to a spectral analyzer where they are processed and used to infer the stiffness values of the underlying layers.

The spectral analysis of surface waves method is used to determine mechanical characteristics of asphalt and concrete pavements and concrete structural members. The method can also measure changes in the elastic properties of concrete slabs during curing, detection of voids, and assessment of damage.

**Relevant References:** ACI 228.2R-98 pg. 10

### **54. Sonic Echo**

Sonic-echo method is primarily used to determine deep foundation integrity or length evaluations. This method is also known as seismic-echo or PIT (Pile Integrity Test).

This method uses a small impact delivered at the head of the deep foundation (pile or shaft), and measures the time taken for the stress wave generated by the impact to travel down the pile and to be reflected back to an accelerometer coupled to the foundation head. The times from impact to reception are recorded by either an oscilloscope or by a digital data acquisition device that records the data on a time basis. If the length of the foundation is known and the transmission time is measured, then the velocity through to foundation can be calculated. Information about the quality of the concrete from this information can be deduced.

This method requires no pre-placed tubes, is easily portable, and rapid. The method does confuse necking and bulging, and cannot measure diameter. It is also unable to determine defects in shafts greater than 30 meters.

**Relevant References:** ACI 228.2R-98 pg. 12

### **55. Steinert Test**

The Steinert Test uses the guard ring principle to attain an approximation of unidirectional flow under pressure. A cap is placed over the testing area. This cap utilizes two concentric chambers separated by a circular rubber seal. The outer edge

of the cap is glued to the concrete surface. The concentric chambers are filled with water and eventually pressurized to 90psi (600 kPa) with compressed air. The flow under the inner chamber is nearly unidirectional, making the recording of flow as a function of time easy to interpret.

**Relevant References:** ACI 228.2R-98 pg. 32

## **56. Third Point Loading**

The standard test designation covers the determination of flexural strength of a sample concrete beam. The sample could be gathered from the field, if possible, or the lab could mix and cure a comparable sample to field conditions.

**Relevant References:** ASTM C 78

## **57. Time Domain Reflectometry**

Time domain reflectometry requires either a monitoring wire to be placed within the concrete member or placement of a monitoring wire for as-built members. Placement of the wire alongside the member yields less accurate measurements than the monitoring wire placed alongside the reinforcement. This technique is capable of determining deterioration of post-tensioned ducts and reinforcing strand deterioration. The use of this technique is largely in the educational realm, but the field applicability appears to be satisfactory.

**Relevant References:** <sup>(66)</sup>

## **58. Ultrasonic Pulse Echo**

The Pulse-Echo method eliminates the requirement of double side access in through transmission. The transducer sends a pulse into the object that is reflected by flaws or interfaces. The pulse is then monitored as it is received by the same transducer or a receiver near it and is sent to an oscilloscope as a time-domain waveform. The round-trip travel time of the pulse can then be obtained. If the wave speed in the material is known, the travel time can be used to determine the depth of the reflecting interface. This method is applicable to a limited member size and performance in concrete with larger aggregate is not known. This method is capable of determining defect depth and locating delaminations and voids in thin members.

**Relevant References:** ACI 228.2R-98 pg. 12

## **59. Ultrasonic Pulse Velocity**

The through transmission method of testing concrete is based on measuring the travel time over a known path length of a pulse of ultrasonic compression waves. The technique is more commonly known as the ultrasonic pulse velocity method.

The speed of propagation of stress waves depends on the density and the elastic constants of the solid. We are looking for these variations in density to find potential subsurface problems. By determining the wave speed at different locations in the structure, we are able to evaluate the uniformity of the concrete. The compression wave speed is determined by measuring the travel time of the stress pulse over a known distance.

The shortest time is found from a straight path through a uniform mix. Travel times get longer as the pulses have to go through inferior concrete or around a large crack or void. If there is a large air interface from a void, the ultrasonic pulse is interrupted and does not make it through the medium to the receiver. This technique is used to determine relative concrete strength and condition, location of voids, and location of other subsurface anomalies.

**Relevant References:** ASTM C 597, ACI 228.2R-98 pg. 6

## **60. Ultrasonic Thickness Gage**

Ultrasonic Thickness gages are commercially available. This technique requires direct contact with the underlying reinforcement. This can give a better indication of section size and strength of the member.

## **61. Uranyl Acetate Fluorescence**

Uranyl Acetate Fluorescence is utilized to determine alkali-silica reactivity in concrete. The only definitive evidence of alkali-silica reactivity is the presence of the products of the alkali-silica reaction. These products are visible under ultraviolet light when sprayed with a Uranyl acetate solution. The greater the intensity of the yellowish color, the greater the concentration of alkali in the concrete.

**Relevant References:** SHRP C 315

## **62. Void Determination**

This is a laboratory test to determine the extent of a possible air-void system in concrete. The technique involves the use of microscopic equipment to determine various issues regarding the concrete including: air content of the concrete surface, void frequency, spacing factor, and paste-air ratio of the air-void system.

**Relevant References:** ASTM C 457

## **63. Water Soluble Chloride Content**

Water-soluble chloride determination is a laboratory procedure used to determine the chloride content of field gathered specimens. This test determines the amount of chloride soluble to water.

**Relevant References:** ASTM C 1218, ACI 222R-96

## **64. Windsor Probe**

The Windsor probe is a penetration test that provides an indication of the compressive strength of a concrete sample. The test consists of driving a .25-inch diameter by 3.125-inch long probe into concrete by means of a precision powder charge. The depth of penetration provides an indication of the compressive strength of the concrete. Although the manufacturer provides calibration charts, the instrument should be calibrated for the type of concrete and the type and size of aggregate used.

Equipment is comprised of a powder-actuated gun or driver, hardened alloy probes, loaded cartridges, a depth gauge for measuring penetration of probes, and other related equipment.

Probe test results vary and are not expected to give accurate values of concrete strength. The ability to check the quality and maturity of in-situ concrete is the test's greatest attribute. It provides a means of assessing strength development with curing.

**Relevant References:** ASTM C 803

## **B. Test Methods for Steel Structures**

The following is a description of some of the test procedures mentioned previously in the guidelines as they apply to steel bridge components:

### **1. Acoustic Emission**

Acoustic Emission testing is used to “listen” to cracks as they form. As a crack grows, it emits tiny amounts of elastic energy that propagate outward from the source as an acoustic wave. Sensors are placed on the surface and detect and measure these acoustic waves. These measurements provide information as to the location and rate of crack growth.

Piezoelectric transducers are set up on the surface to sense the energy of the acoustic waves. Digital counters, computer filtering, and graphic tape recorders are used to process and record the sounds.

Sensors require clean smooth surfaces free of thick coatings. Sensors can detect cracking of .0001 inch in length. This system is easily transportable and the transducers are considered inexpensive and can be permanently attached to bridge structures, offering potential long-term and remote monitoring applications.

**Relevant References:** ASTM E1106, ASTM E650, and ASTM E 569

### **2. Coating Tolerance Thermography**

Coating tolerance thermography utilizes a source that introduces heat flow in the plane of the steel plates of a typical bridge structure. The source is placed to both sides of a suspected crack area, one at a time, introducing heat from both sides of the crack. A camera captures these thermal images associated with both directional heating. The images undergo a data manipulation, the signals are subtracted from each other, and areas of increased heat are indications of defective areas. Consult the apparatus manual for details regarding its operation.

### **3. Computed Tomography**

Computer tomography (CT) is a radiographic inspection technique that uses a computer to construct a 3-dimensional image by reconstructing several 2 dimensional images taken at various positions around an object. The CT image is the result of triangulation from many different directions. Defects are identifiable since the radioactive source scatters differently as it passes through regions of different density. These regions of differing densities are likely the defective areas.

**Relevant References:** ASTM E1570

### **4. Dye Penetrant Testing**

Liquid Penetrant Testing (PT) is a widely used NDE technique. The test material is coated with a visible or fluorescent dye solution. Excess dye is removed from the surface, and then a developer is applied. The developer acts like a blotter and draws penetrant out of imperfections, which are open to the surface. With visible dyes, the vivid color contrast between the penetrant and the developer makes the "bleed-out" easy to see. With fluorescent dyes, an ultraviolet lamp is used to make the bleed-out fluorescent, thus allowing the imperfection to be seen readily. Visible and fluorescent liquid penetrant examination reveals surface discontinuities on materials having non-porous qualities. PT consists of portable, economical equipment that can be easily used for inspections. A basic kit consists of a small grinder to remove rust inhibitors, a part cleaner, the dye penetrant, and a developer to facilitate the bleed out of excess dye from a discontinuity. It is fast, simple, and can cover large areas of inspection. Again, this test can only be performed on flaws that are free from debris and open to the surface. This method works well with both ferrous and nonferrous materials.

**Relevant References:** ASTM E165, AASHTO AWS D1.1

## **5. Magnetic Particle Testing**

Magnetic Particle Testing (MT) involves introducing a magnetic field in a ferromagnetic material and dusting the surface with iron particles (either dry or suspended in liquid) over the area being tested. The presence of surface imperfections will be indicated by distortion of the magnetic field, which causes the iron particles to concentrate near areas of flux leakage.

**Relevant References:** ASTM E709

## **6. Radiography**

Radiography provides a picture of the interior of steel by showing the light colored dense areas that appear as radiation passes through the concrete and reinforcement. Voids and other defects can be identified because of the discontinuity in their densities at the defect region. These appear as dark spots on the photograph because they are unable to block the radiation from special photographic film.

A radiation source is placed on one side of the test specimen and a beam of radiation is emitted. The density affects the beam and the result is shown on the film.

Operators must be licensed and highly skilled. Although the X-ray equipment is bulky and expensive, it is one of the most frequently used methods.

**Relevant References:** ASTM E94, ASTM E1955

## **7. Ultrasonic Testing**

Ultrasonic testing has been used to determine internal defects in steels for years.

In ultrasonic testing an ultrasonic pulse is introduced into a material and the changes the pulse undergoes are an indication of the internal condition of the steel. Ultrasonic testing utilizes piezoelectric transducers to typically introduce stress pulses in to the material and measure the response of the material to the stress pulse. Ultrasonic testing can be oriented in several ways. The through method can be used to determine the density of the steel material between transducers. Areas of different densities from the majority of the steel are likely defective. The pulse-echo technique can be utilized to determine vertical flaw depth in steels. Angled transducers can be utilized to send stress pulses into confined areas and determine internal characteristics. Data acquisition can also be utilized to gather both two-dimensional and three-dimensional images based on the degree of data acquisition used.

Experience is the best means of ensuring proper utilization and interpretation of ultrasonic testing results.

**Relevant References:** ASTM A577, ASTM A435, ASTM E587, and ASTM A898

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