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CURRENT BRIDGE DECK REHABILITATION PRACTICES: USE AND EFFECTIVENESS

By

Melvin Ramcharitar, B.Eng.

Ryerson University, 2002

A thesis

presented to Ryerson University

In partial fulfillment of the

Requirements for the degree of

Master of Applied Science

in the program of

Civil Engineering

Toronto, Ontario, Canada, 2004

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ABSTRACT

CURRENT BRIDGE DECK REHABILITATION PRACTICE: Use and Effectiveness

Melvin Ramcharitar
2004, MASc., Department of Civil Engineering, Ryerson University

Approximately thirty to forty percent of all bridges across North America have some form of deterioration on them. Many organizations/agencies across North America are investing significant amounts of money on repairing and rehabilitating their bridges. The reason being, these bridges are deteriorating due to heavy use (overloading from today's oversized trucks), old age (many built in late 1950s and 1960s) and environmental and chemical attacks (deicing salt applications during the winter season).

The purpose of this thesis concentrated on one area, namely bridge decks. To better understand how these organizations/agencies were dealing with bridge deck deterioration, a survey containing thirteen questions was developed and sent out throughout North America, to Department of Transportation, Ministry of Transportation, Municipalities, Bridge Authorities and Consultants.

The survey was made up of six parts, each focusing on different areas during a bridge rehabilitation/repair operation. Areas looked at were: Condition Surveys, Concrete Removal, Rehabilitation Techniques, Environmental Impacts and Service Life.

ACKNOWLEDGEMENTS

I would like to thank my two faculty supervisors, Dr. M. Lachemi and Dr. A. Hossain for sharing their expertise knowledge and guidance during the development of this thesis. A special thanks to Dr. M. Lachemi, whom for the past five years have helped me focused and be in the right path with his encouragement, support, wisdom and understanding, I am indebted to you professor.

I would also like to thank all of the Ministries of Transportations, Department of Transportations, Bridge Authorities, Canadian Consultants and City Municipalities who responded to a survey that was sent out to them.

Lastly, a special thanks also goes out to my family for their extended support and encouragement not only throughout the development of this thesis but for the past twenty four years of my life. Thank You.

DEDICATION

To My Family

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NOTATION

A	Polarized Area
B	Constant, for actively corroding steel in concrete 25 mV, for passive situations 50 mV
C_p	Compression wave propagation velocity
d	Density
E	Steel Potential
f	Frequency
F	Faraday Constant
I	Current
i_c	Corrosion rate
m	Actual rebar loss
M	Atomic Weight
p	Corrosion Penetration
R_p	Polarization Resistance
$r(t)$	Reflected Waveform
$s(t)$	Transmitted Signal
t	Time
T	Thickness
T_A	Time delay for signal to propagate from asphalt surface to concrete surface
T_C	Time delay for signal to propagate from concrete surface to rebar layer
v	Velocity
z	Ionic charge

ϵ	Dielectric Constant
ρ	reflection coefficient
ρ_A	reflection coefficient of air/asphalt interface
ρ_C	reflection coefficient of asphalt/concrete interface
ρ_R	reflection coefficient of steel reinforcement

1.0 Background and Defects of Bridge Decks

1.1 Introduction

Many States and Provinces across North America invest significant amounts of money on repairing and rehabilitating their infrastructures, namely bridges. These bridges are deteriorating due to heavy use, old age and environmental and chemical attacks. Many of these bridges were built using ordinary and reinforced concrete in the late 1950's and throughout the 1960's. According to Hassanain (2003), approximately 30% to 40% all bridges across North America have some form of deterioration on them. Canada has 80,000 bridges of which 40,000 are over 35 years old and many of them had a designed life of 50 years (Lounis, 2003). The United States has approximately 578,000 highway bridges and approximately 42 percent or 238,000 of them are classified as deficient by the Federal Highway Administration (FHWA) (Ahlskog, 1990). The FHWA estimated in 1987, that it would have cost approximately $92 Billion between the periods of 1987 to 2005 to have corrected all existing and accruing deficiencies (FHWA, 1989).

In many bridge rehabilitation projects over the years, because of funding constraints, inadequate condition evaluation, and lack of quality rehabilitation construction practices, has resulted in the construction of poorly planned repairs. This has resulted in durability problems and poor performance of concrete. These repair failures can be attributed mainly to material selection (chosen material may not be compatible with existing residual concrete) and inadequate field practices.

The durability of concrete is determined by its quality and the ambient conditions for which it was designed for. When concrete is designed to withstand the conditions for which it was designed for it is said to be durable (Xanthakos, 1996). The quality of concrete is mainly determined by its water to cement ratio (w/c). The lower the w/c, the higher the durability of the concrete, provided the materials used are sound and properly proportioned, consolidated and cured. Air entraining is also added to improve the concrete resistance to freezing and thawing.

In a study carried out by Rostam (1991), it was concluded that when an organization/agency carries out regular maintenance on a bridge deck, it results in lower operating costs to keep the structure in service, provided correct assessment and correct interventions are made at the correct time. Rostam also concluded that the costs for repairing a damaged bridge deck is much higher than the cost for installing some form of protection measures (cathodic protection, anode mesh system, corrosion inhibitors etc.) on the bridge deck while it still remained undamaged. Rostam also added that the cost for repairing a damaged bridge deck can be as much as five to ten times higher than the costs for early preventative maintenance. The most common defects found in bridge decks are described in the following sections.

1.2 Effect of Salt

The most common types of deicing salts used are sodium chloride (NaCl) and calcium chloride ($CaCl_2$); however sodium chloride is more cost effective of the two (Yehia and Tuan, 1999). When salt is in its solid state, it does not attack the concrete, but when present in a solution form with the aid of water, they can react with the hardened concrete. During the winter, many organizations and agencies across North America require roads and bridges to be at a high level of service during a storm or overnight freezing (Mason et al., 2001). Therefore, deicing salts are routinely applied to decks to prevent ice formation. These salts when in solution, release chloride ions (Cl^-) which penetrate through the concrete's cracks and pores, eventually coming into contact with the reinforcing steel. Penetration of chloride ions in concrete can lead to severe corrosion of the steel reinforcement (Rostam, 1992). When water and oxygen are present, an electric current develops between various parts of the bars causing corrosion. When corrosion occurs, high tensile stresses are induced on the surrounding concrete due to expansion (i.e. rust formation on the bars). This results in hairline cracks being developed and with time they become enlarged as a result of freezing and thawing or due to the action of traffic (FHWA, 1979). Since salt is a deicing agent it causes freezing shock on the concrete surface, this results in thermo cracks, delamination and scaling. Salt is hygroscopic which makes it more difficult to dry up when on the concrete surface (Rostan, 1992).

1.3 Overloading

Today's trucks are substantially heavier, travel much faster, and induce greater impact forces on bridge decks and they account for a much larger percentage of the traffic flow. These load increases have not only damaged the decks (cracking, erosion, potholing etc.), but also have contributed to reducing the fatigue life of deck girders. The most detrimental effect heavy trucks have on bridges is the increase in stresses they apply on bridges, which often exceed allowable levels. Due to the fact that it is very costly and disruptive to strengthen bridges, many highway authorities across North America have accepted a limited amount of overstress; 10-15% overstress for prestressed concrete beams and 5% overstress for steel girders (FHWA, 1979).

1.4 Scaling

This type of defect occurs where there is gradual decomposition of cement paste (flaking away of the cement paste) beginning at the surface and progressing downward as shown in Figure 1.1. It usually occurs when deicing chemicals are used on bridge decks and when the deck slab concrete is not properly air entrained. Scaling is caused by a build up of osmotic and hydraulic pressures produced when water freezes in the concrete. These pressures increase until they become critical. Since no entrained air voids are present to relief these pressures, scaling results (Kosmatka and Panarese, 1988). Salt is hydroscopic (moisture absorbing) and attracts water. This results in a concrete that's more saturated increasing the potential for freeze-thaw deterioration. The extent of scaling depends upon the amount of deicer used and the frequency of application (Sayward, 1984). Scaling can be reduced, sometimes prevented by using low water to cement ratio mix, moderate to high cement content, air entrainment, proper finishing and curing (Whiting, 1989). Scaling is usually found in older structures and in bridges covered with asphalt that lacks waterproofing (Xanthakos, 1996).

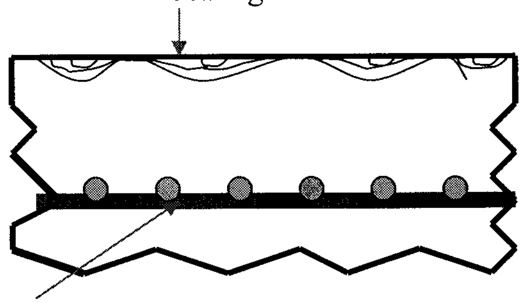


Figure 1.1 Typical Scaling Detail

1.5 Cracking

A crack is usually defined as an incomplete separation of the concrete into two or more parts with or without a space between them (Minor et al., 1988). Figure 1.2 shows a typical illustration of a crack detail. This type of defect usually occurs after scaling. It can be transverse, longitudinal, diagonal or random. Cracking occurs when the tensile stresses becomes greater than the tensile strength of the concrete (MacGregor and Bartlett, 2000). Transverse cracking usually occurs over the top of the steel bars, whilst longitudinal cracks are common between prestressed box beams. Random cracking can be caused by overloading a structure, improper curing methods, material defects and differential foundation movement. The significance of cracking depends on the structure type, crack origin and whether the width and length increase with time. Cracks are categorized into several types as discussed below.

- Plastic Shrinkage cracks results from rapid drying of concrete while in the plastic state (Manning and Bye, 1983). These cracks are wide and shallow but are spaced at regular intervals and may form a definite pattern. Plastic shrinkage cracks can be prevented by using a monomolecular evaporation retarding film, water fogging, wind breaks or casting concrete when evaporation is low (Krauss and Rogalla 1996).

- Drying Shrinkage cracks occurs from the drying of restrained concrete after it has hardened. They are usually finer and deeper than plastic shrinkage cracks and have random orientations.
- Settlement cracks can have any shape and width ranging from hairline cracks (typically found above reinforcing bars) to wide cracks in supporting members (piers, abutments). These cracks are dangerous as they affect the load carrying capacity of the bridge.
- Structural cracks results from variations between assumed and actual stress levels if the structure is not properly reinforced (Minor et al., 1988).
- Reactive aggregate cracks results from either alkali silica or alkali carbonate reactions between the mineral aggregates and the cement paste. These cracks increase with time due to the action of freezing and thawing and the presence of moisture. This type of cracking cause considerable damage by causing abnormal expansion and loss of strength.
- Corrosion-induced cracks also called delamination results from the corrosion of steel reinforcement. Their width increases with time when corrosion is allowed to continue. These cracks are generally located directly above or below the reinforcement (Minor et al, 1988).

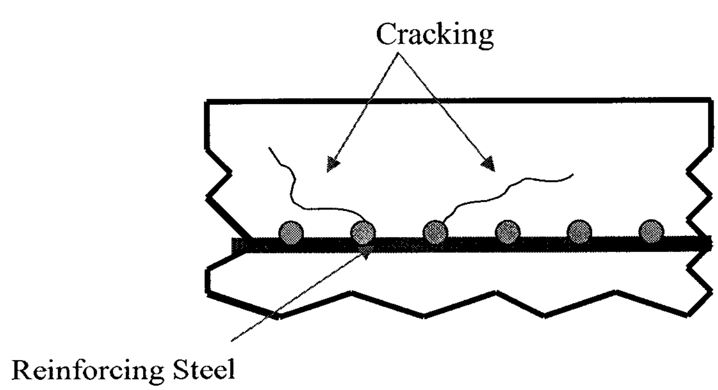


Figure 1.2 Corrosion Induced Cracking Detail

1.6 Delamination

Concrete Delamination in bridge decks usually occurs when there is debonding between the reinforcing steel and the adjacent concrete. Debonding is caused by the disruptive stresses arising from the corrosion of reinforcing steel (Figure 1.3). These stresses arise due to the increase in volume around the reinforcing steel caused by rust formation. Delamination is also called corrosion induced cracks because they are formed in the same manner. Delamination is also often referred to as debonding of concrete, because the stresses produced during rust formation (corrosion process) causes the concrete to crack. Delamination of concrete usually occurs at the upper layer of the reinforcing steel. These internal cracks are precursors of spalling and the concrete at delamination will eventually loosen and spall (Gilstad, 1993). Chapter 2 discusses in detail various surveys used to detect concrete delamination.

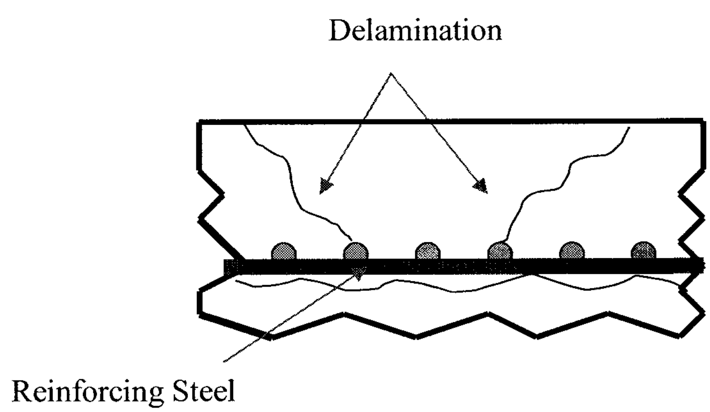


Figure 1.3 Typical Delamination Detail

1.7 Spalling

This type of defect usually occurs at the top reinforcing steel. It is related to concrete delamination and is accelerated with the increase use of chlorides causing the concrete to "popout" (Figure 1.4). Spalling is considered to be a very dangerous type of defect. The rate of development and severity depends on factors such as the depth of concrete cover, porosity of concrete, volume and intensity of traffic, ability of the deck to deform without cracking, frequency of chloride application and number of freeze/thaw cycles. When spalling occurs throughout the entire structure, this reduces the load carrying capacity of the structure. Spalling weakens the structure, exposes the steel reinforcing and disrupts the riding quality of the deck (Minor et al., 1988). To prevent or reduce spalling from occurring, measures such as reducing the application rates of deicers being applied and better waterproofing should be used.

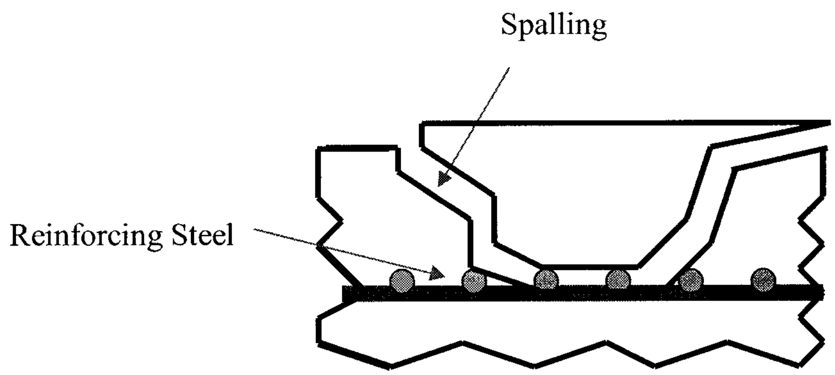


Figure 1.4 Typical Spalling Detail

1.8 Chloride Contamination

In North America, many states and provinces use deicing salts as a means of melting snow and improving road conditions during winter conditions. These deicing salts can be in a solid state (rock salt) or a liquid state (salt brine - solution of rock salt and water). Either form releases chloride ions onto the surface of the bridge deck. These chloride ions can only penetrate the deck surface if there is sufficient moisture present as shown in Figure 1.5. The rate of penetration of chloride ions as outlined by Emmons (1993) depends on the following:

- The amount of chlorides coming into contact with the concrete;
- The permeability of the concrete;
- The level of moisture present; and
- Cracks

With time, the chloride ions in the presence of water will eventually reach the reinforcing steel causing the steel to corrode. As rust develops there is an increase in volume around the reinforcing steel causing stresses in the concrete. These stresses cause the concrete to expand producing cracks and areas of spalling.

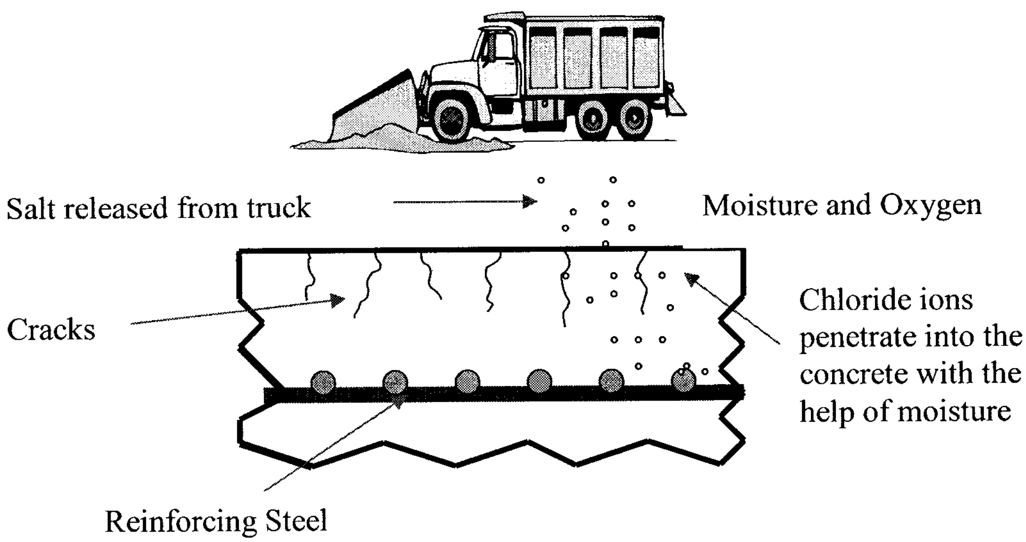


Figure 1.5 Typical Chloride Contamination Detail

1.9 Honeycombing and Air Pockets

These defects usually occur at the time of construction. Air pockets results when fresh concrete is not properly consolidated. Honeycombing occurs when there are spaces between the coarse aggregates particles. Honeycombing occurs when the cement mortar has not filled these spaces, this is caused by improper consolidation and/or leakage of mortar between sections of the formwork (Xanthakos, 1996). An illustration of honeycombing and air pocket formation is shown in Figure 1.6.

The primary causes of honeycombing as outlined by Emmons (1993) are as follows:

- When concrete forms are not constructed properly, there can be leakages at the joints of the forms. Whenever there are leakages it results in cement paste being wasted. Cement paste is essential in fresh concrete in order to have good workability, develop good bonding and to ensure proper consolidation.

- Improper mix designs would also lead to voids being formed in the fresh concrete. When the cement content is to low, the resulting concrete mix tends to segregate as there isn't enough cement to bind the aggregates together. So in order to minimize honeycombing and air pocket formation, the mix should contain a sufficient amount of fine aggregates to fill the voids in concrete. Fresh concrete should be workable and agents such as superplasticizers and retarders can be used to improve the workability of fresh concrete.
- Improper placement of fresh concrete can also lead to unwanted voids in fresh concrete. When placing fresh concrete measures should be taken to avoid excessive free fall as this causes segregation. Vibrators should be used to ensure the concrete is properly consolidated.

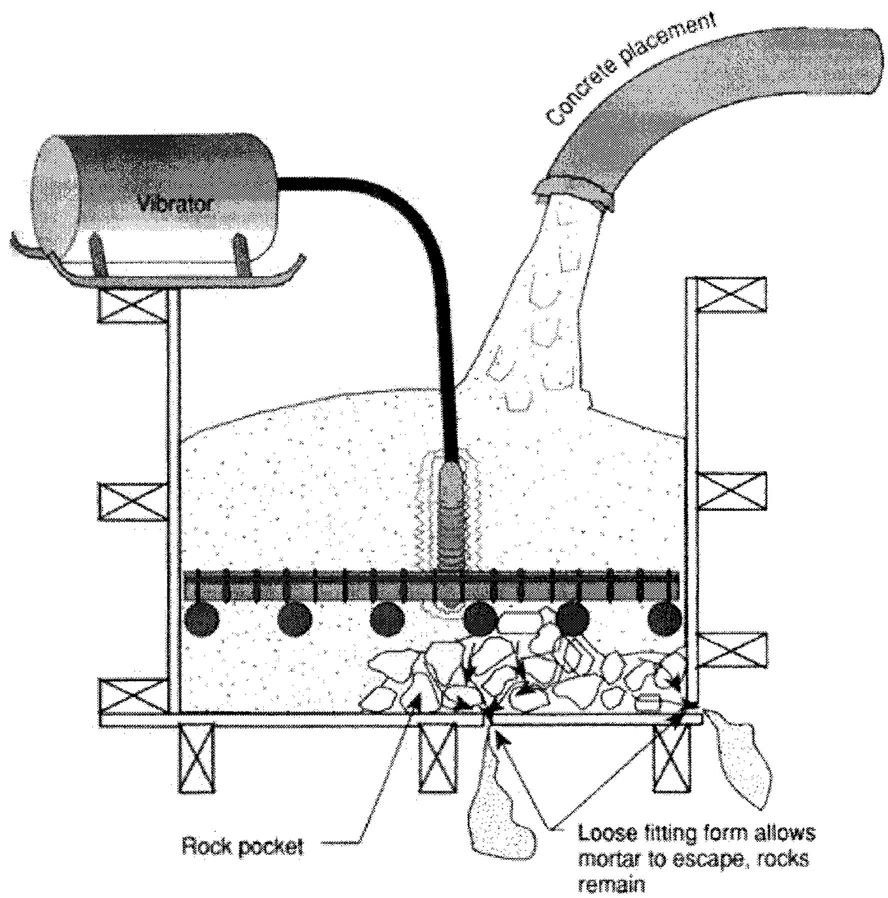


Figure 1.6 Honeycombing of Concrete (Emmons, 1993)

1.10 Frost Damage

Frost damage initially starts off as scaling and it is then followed by loosening up of the surface aggregates. This type of damage is most prevalent on horizontal surfaces. Frost damage on vertical surfaces is often associated with spalling and progressive cracking. Frost damage commonly occurs on poor quality or poorly compacted concretes and concretes containing shrinkable aggregates (these aggregates are readily saturated and vulnerable). Concrete durability largely depends on the attack of frost and deicing salts (Setzer, 1992).

In a study carried out by Zech and Setzer (1989) they calculated the amount of frozen water in concrete. They concluded that between -15°C to -25°C most of the pore water is unfrozen at low water/cement ratios. This is mainly the reason why low water/cement ratio concrete is more resistant to freeze and thaw cycles. When frost is combined with deicing salts, they lower the freezing point and this causes severe damages (Setzer, 1992). This resulting differential temperature induces secondary tensile stresses, which in combination with stresses already in the concrete exceed the tensile strength and spalling occurs. In order to reduce the action of frost damage, concrete used on bridge decks should have a low water/cement ratio and be properly cured. This ensures the pore water remains unfrozen up to -25°C.

1.11 Objective of Thesis

The main objective of this thesis is to develop a database outlining what various organizations/agencies across North America are doing in terms of bridge deck rehabilitation work. The targeted organizations/agencies included mainly Ministries of Transportation in Canada and Department of Transportations in the United States and also a few Canadian consultants and bridge authorities who are also familiar with the processes involved during a bridge rehabilitation project.

Before carrying the thesis main objective, a review of current practices dealing with bridge deck rehabilitation was done by performing a literature review. After reviewing what current practices are used, a survey was developed as outlined in Chapter 5 of this thesis. This survey was sent out to over one hundred individuals working for over fifty different organizations/agencies. This survey was made up of six different parts, each part focusing on a different area during a bridge rehabilitation operation.

The first part of the survey focuses on questions related to condition surveys. The second part of the survey focused on questions related to the removal of deteriorated concrete. The third part of the survey focused on questions related to concrete repair and rehabilitation. The forth part of the survey focused on questions related to concrete compatibility. The fifth part of the survey focused on questions related to proper finishing and curing of concrete. The sixth and final part of the survey focused on questions pertaining to the service life of a bridge deck.

1.12 Thesis Outline

The content of this thesis is arranged according to the following major sections:

(i) Chapter 2 focuses on the assessment and evaluation of bridge decks. This chapter will include details about what goes on during both a visual survey and detailed condition survey of a bridge deck. These surveys are generally carried out to determine if the deck requires immediate repairs or if these repairs can be done at a later date. For each of the two type of survey mentioned, a summary of the common tests and techniques involved as well as advantages and disadvantages will be discussed.

(ii) Chapter 3 focuses on the various types of concrete removal techniques commonly used by these organizations/agencies. Once a rehabilitation technique is chosen, the next step on a rehabilitation project is to outline what types of concrete removal technique will be ideally suited for that specific project.

(iii) Chapter 4 focuses on the various rehabilitation methods that are used today on many bridge decks across North America. This chapter outlines various techniques which are ideally suitable for various working environment and time constraints applications. This chapter will also look at the impact a rehabilitation project has on both motorists and on the Environment.

(iv) Chapter 5 focuses on the development of the questionnaire. The questionnaire contained six parts and each part is discussed in detailed explaining why these six parts were specially chosen. An explanation is also given on the expected outcome.

(v) Chapter 6 discusses the results obtained from carrying out this questionnaire. Each question in the questionnaire is being analyzed.

(vi) Chapter 7 gives conclusions and recommendations obtained after analyzing the survey and available literature.

2.0 Assessment and Evaluation of Bridge Decks

2.1 Introduction

Before a bridge rehabilitation project begins, a significant amount of time is spent on assessment and evaluation. The assessment and evaluation stage is one of the most important stages before construction begins as it determines which of the bridges are in need of immediate repair and which can remain in service for a few years before they require repairing (patching) or full scale deck replacement.

Concrete bridge decks are designed with the intention they will be in service for decades. Unfortunately, damages such as concrete deterioration and rebar corrosion often occur way before their intended planned service life. These damages are often hidden inside the deck and not manifested onto the surface until it is at an advanced stage (Huston et al., 2000). In order to minimize these damages a precise assessment and evaluation of these bridge decks are required.

For this research, assessment and evaluation focus mainly on the deck elements such as:

- Wearing Surface: This provides riding surface for traffic and is placed on top of the structural slab.
- Structural Deck: This is part of the structural superstructure system and its main function is providing load-carrying capacity. For this project the main focus is on reinforced concrete slabs.
- Other deck elements such as the sidewalks, curbs, steel railing or parapet walls; are not considered in this study.

The assessment and evaluation of bridge decks are usually done by carrying out two types of investigations; visual inspection surveys and detailed condition surveys. These surveys are usually carried out by an experienced bridge inspector (in most cases a licensed professional engineer with at least five to ten years of experience). Depending on the size of the structure, the inspector may have as much as four technicians or sometimes teams of crews. The bridge inspector as outlined by Xanthakos (1996) is responsible for the following when conducting both the visual and detailed surveys:

- Familiarity with basic design and construction features of the bridge.
- Ability to analyze loads in order to determine structural capacity assessments.
- Ability to detect deficiencies, assess its seriousness and make recommendations.
- Ability to recognize problem areas, so preventive maintenance can be carried out.

2.2 Visual Inspection Surveys

Typical Visual Inspection investigations are made up of three surveys and are discussed in detail in the following sections.

2.2.1 Visual Observation Survey

A visual observation survey is a non destructive evaluation technique used during bridge deck inspections to assess their material and performance defects. This type of survey is usually carried out very quickly, so the extent of deterioration is usually estimated not measured. The main objective of this survey is to detect possible structural defects such as scaling, cracking, delamination, spalling and other visual signs of deterioration (Litvan, 1982). Bridge decks treated with deicing agents or located in salt environment are carefully examined because of their vulnerability to chemical attack. Since visual observations are usually done by an experienced inspector. Another purpose of this survey is to identify, as accurately as possible the cause of concrete delamination.

A large portion of bridges requiring immediate repair are located in densely populated areas or have a high Average Annual Daily Traffic (AADT). In these populated areas, they are required to provide a high LOS (Level of Service) for the public during the winter months. In order for this to occur, salt (NaCl) application rates tend to be 2-3 times higher that what is normally applied in smaller cities and rural areas (Mason et al., 2001). This results in higher concentration of chloride (Cl) ions being available to attack the reinforced concrete deck slabs (Beazley et al., 1996). Corrosion of the steel reinforcement may sometimes be detected by rust stains on the concrete surface. This should not be confused with ferrous sulfide inclusions in the aggregate fraction or with rusting of the tire wires (Xthanthos, 1996).

Whenever a bridge deck is protected by an asphalt surface or other types of wearing surfaces, these wearing surfaces tend to hide concrete deterioration and other defects until they become well advanced. In order to prevent this, these surfaces are carefully examined and regularly inspected and any signs of deterioration properly noted and recorded (Litvan, 1982).

Where deterioration is suspected, the surface may be spot checked by removing small sections for observation and inspection. Whenever the deck is covered with a wearing surface the underside of the deck must also be examined for signs of deterioration such as rust stains, cracks and water spots (Litvan, 1982) as shown in Figure 2.1.

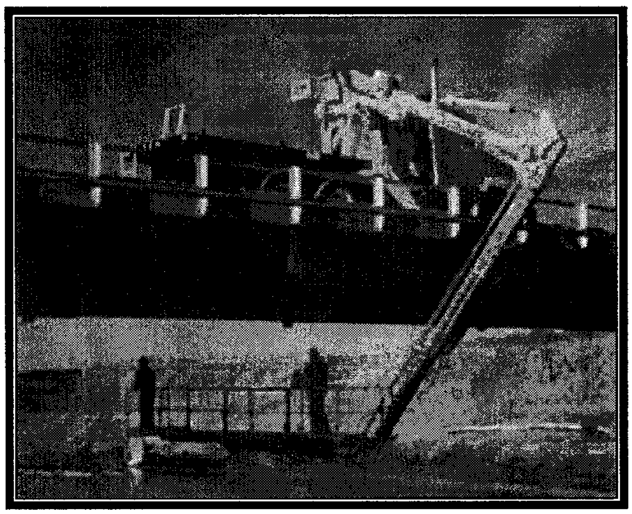


Figure 2.1 Bucket Snopper used to examined underside of the deck

The advantages of visual inspections are that they are quick, practical, not as expensive as a detailed condition survey and non destructive. In a recent study carried out by Phares et al., (2000), they concluded that visual inspection is the most common form of non destructive evaluation used during bridge inspection.

However, there is a big disadvantage as it does not reveal what is happening below the surface of the deck. This type of inspection does not reveal hidden defects or deterioration in concrete such as delamination, rebar corrosion and low concrete cover to reinforcing steel.

2.2.2 Concrete Surface Deterioration Survey

This survey is used together with the visual observation made by an inspector to give an indication where problems are occurring on the deck surface. For this survey, the area and locations of patches, spalls, exposed reinforcement, honey-combing, wet areas, scaling and other defects and deterioration are all measured and recorded, using hand drawings and checked with the aid of tables such as 2.1 (Crack width measurements) and 2.2 (Scaling measurements) to determine there classifications.

Crack widths are measured using a crack comparator. The size and location of cracks are recorded with respect to grid lines as outlined on existing structural drawings. The depths of the cracks are measured using Feeler gauges or using fine wires (MTO, 1996). On asphalt covered decks, the condition of the asphalt and cracks wider than 3 mm shall be recorded. Sealed cracks are also measured and recorded.

Scaling on the other hand should be reported with respect to location and severity. Delaminations are not generally visible except where they are shallow and show discolorations. Spalls are easily recognized, and the reinforcement is often exposed within the spalled zone.

Concrete Surface Deterioration Surveys are relatively cheap as no specialty equipment is required. However, it is very time consuming and depending on the location of the bridge and daily traffic flow on the bridge this survey might not be carried out unless the bridge is closed for several hours.

Table 2.1 Scale for describing crack width as outlined by the Transportation Research Board (1979)

Types of Cracks	Width (mm)
Hairline	< 0.1
Narrow	0.1 - 0.3
Medium	0.3 – 0.7
Wide	> 0.7

Table 2.2 Types of scales with respect to depth and appearance as outlined by the Transportation Research Board (1979)

Types of Scales	Depth (mm)
Light	0.5 mm, coarse aggregate not exposed
Medium	5 – 10 mm, coarse aggregate exposed
Heavy	10 – 25 mm, coarse aggregate projecting from surface
Severe	> 25 mm, loss of coarse aggregate particles

2.2.3 Delamination Surveys

Concrete delamination is detected by striking the surface using tools such as hammers, steel rods or chains and noting the change in sound being emitted. The chain drag method is sometimes used during visual inspections because of its relative low cost and it can be done pretty quickly, however it is not the most accurate method to measure delaminations.

Generally there are two types of chains available for the chain drag as outlined by Litvan (1982). The first type consists of four to five segments of 25 mm links of chain about 450 mm long. The second type consists of a chain that weighs approximately 2.2kg/m with 50 links.

When this method is done on concrete decks, it depends on the inspectors experienced in being able to accurately detect delamination using his/her ear after striking the surface of the concrete. The chain is dragged along the surface of the concrete in a swinging motion, resulting in a ringing sound. When there is delamination encountered, a “dull” sound is produced.

Delamination surveys are more reliable when done on exposed concrete decks as opposed to when they are done on asphalt covered decks only. It is recommended that the chain drag method not be carried out on asphalt covered decks. When the concrete surface is covered with an asphalt surface, the delamination test can be difficult to do. The “dull” sound produced on an asphalt surface can also be due to the loss of bond between the concrete deck slab and the asphalt overlay. To correct these problems, suspected areas along is removed and the delamination test is then carried out. If the bridge deck is part of a major arterial road, then this option will not be considered as it is very time consuming. So the best alternative would be to perform a condition survey involving a radar test because it is quick and has very little traffic disruptions.

2.3 Detailed Condition Surveys

Detailed condition surveys are done after a visual inspection survey has been completed. A typical detailed condition survey consists of one or a combination of various techniques. Condition Surveys can be categorized into two types; non destructive evaluations and destructive evaluations. Each of which will be discussed in the following sections.

2.3.1 Non Destructive Evaluations

Non destructive evaluations are the most economical and effective techniques available to assess the condition of bridge decks (Loulizi and Al-Qadi, 1997). A Non Destructive Evaluations is defined as any test or method that yields information regarding the quality of a structure (or a portion of it) that does not impair the serviceability of the element or structure (Poston et al., 1995). This section is made of five common non destructive tests used by many organizations and agencies across North America.

2.3.1.1 Corrosion Potential Surveys

There are currently two common types of Corrosion Potential surveys which can detect if corrosion is occurring in steel reinforcing bars. They are described in the following sections:

2.3.1.1.1 Half Cell Potential Survey

Stratfull et al. (1957) were the first pioneers who developed the half-cell potential technique for use on decks damaged by chloride induced corrosions. In 1977, the half-cell potential technique became adopted by ASTM as a standard test method C 876.3 (Novokshchenov, 1997). It has since been revised in 1980 to C 876-80 (ASTM C 876, 1980). Since the 1980s, the half-cell potential method has been widely used in identifying corrosion occurring in the reinforcing steel because of its simplicity and cost effectiveness (Broomfield et al., 2002).

This survey measures the degree of corrosion between the embedded steel reinforcement and the surrounding concrete. In this method, corrosion activity is measured by comparing the electrical potential between a rebar and a reference electrode (usually a copper/copper sulfate cell, in contact with the concrete surface) (Vassie, 1991). Usually these readings are taken around the areas of suspected delaminations. The connection is made by directly connecting the positive lead of the voltmeter to the reinforcing steel bar. The other lead of the voltmeter is connected to the reference electrode. The reference electrode is inserted in a water filled hole of approximately 5 mm in diameter drilled through the asphalt coating as shown in Figure 2.2. Figure 2.3 illustrates a schematic diagram of the measurement process of the corrosion rate during half-cell potential measurement.

The main purpose of the half cell potential test is to detect corrosion and related damages (mainly delamination) occurring in bridge decks. This test is designed to identify locations along the deck where corrosion activity is most likely occurring and also outlining areas where there might be delaminated concrete. Studies carried out by Polder (2001) and Mc Carter et al., (1995) have shown that the measured resistivity values are closely related to the actual corrosion rate occurring in the structure when repairs are undertaken.

The advantages of the Half-Cell Potential test are as follows:

- When this technique is carried out on dry decks properly, it can be quite successful in identifying locations on the structure which are suffering from corrosion attack.
- Half cells can also be embedded into the concrete to monitor the performance of the reinforcement, when subjected to contaminants such as deicing salts, dust,

debris etc. Table 2.3 illustrates the results that can be achieved using this type of survey.

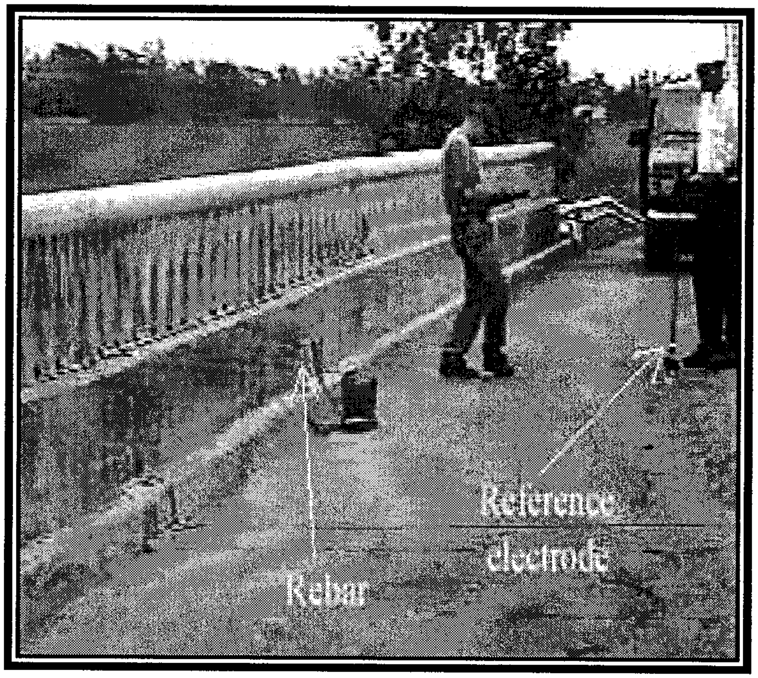


Figure 2.2 Half Cell Potential test (Rhazi, 2000)

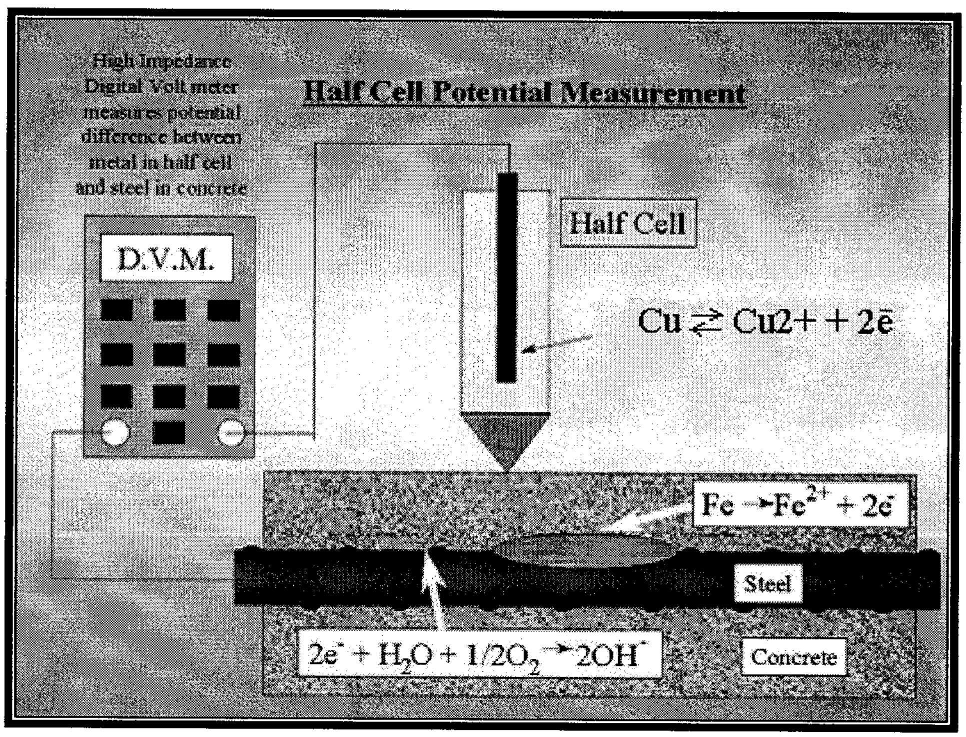


Figure 2.3 Half Cell Potential Measurement (Broomfield et al., 2002)

Table 2.3 ASTM Standards on Half Cell Potentials (ASTM C 876, 1980)

Potentials	Comment
< - 0.2 V	>90% probability no corrosion is occurring in reinforcing
-0.2 to -0.35 V	Uncertain if corrosion is taking place
>-0.35 V	>90% probability corrosion is occurring in reinforcing

The disadvantages of the Half-Cell Potential test are as follows:

- Reinforcing steel must be continuous between the points of measurement and the ground connection to the voltmeter (Rhazi, 2000).
- Half cell potential surveys can only be done on dry decks. The temperature should not be less than 10°C (Minor et al., 1988).
- Obtaining accurate measurements are time consuming.
- When this test is carried out, the bridge is usually closed from anywhere between seven to twelve hours, depending on the size of the deck surface to be surveyed.
- Half-cell potential mapping gives no information about the rate at which corrosion may be occurring in the structure (Millard et al., 2001).
- Gu et al., (1996) have shown that under circumstances such as different moisture content, temperature and oxygen and/or chloride concentration, the data (resistivity measurements) obtained during the half-cell test can be misinterpreted and lead to inadequate conclusions. Gu et al., (1996) believed it is more appropriate to use more than one corrosion evaluation techniques to assess the condition of the bridge decks.

As a result of these disadvantages, this method can not be used on heavily travel roads because closing the bridge for 7-12 hours would be impossible in order to carry out this test. Also it is important to note that this type of survey should not be done on decks using epoxy coated steel. Epoxy coated reinforcement has very low electrical conductivity, so readings will not be very accurate (Litvan, 1982). Also, there are concerns that the half-cell potential test may damage the epoxy coating covering the rebars (MTO, 1996). Other surveys such as chain drag, impact echo, ground penetrating radar or concrete coring can be done. Bridge inspectors use other methods such as ground penetrating radar (GPR) or impact echo techniques because these methods are all less time consuming.

2.3.1.1.2 Linear Polarisation Measurement Survey

Also known as linear polarisation resistance (LPR), this is a survey that has been growing in popularity over the last few years because of its main advantage in directly measuring corrosion rates of steel reinforcement in concrete structures. The half-cell potential technique can only determine if the structure is corroding but cannot calculate the speed at which corrosion is taking place.

The corrosion of steel in concrete is an electrochemical process, which involves an exchange of ions between passive (cathodic) and active (anodic) locations on the surface of a steel reinforcing bar as shown in Figure 2.4. LPR is usually conducted in aqueous solutions (mainly water) on small, uniformly corroded specimens (Broomfield et al., 2002). Performing this test in solution ensures a good contact is made between the half-cell and the concrete surface.

In order to properly evaluate the corrosion rate of the bridge deck it is important to define the polarized area (A) of the rebar. This is achieved by applying an auxiliary electrode for current confinement. LPR measurements are normally made using a three-electrode system, as shown in Figure 2.5. A half cell electrode (usually copper/copper sulphate cell) enables the corrosion potential and over-potential of the steel to be measured between the steel reinforcing bar and an auxiliary electrode placed on the surface of the concrete to relay and confine the current. The auxiliary electrode is situated around the half-cell and is maintained at the same potential as the half-cell. The function of the guard ring is to constrain the electric field from the auxiliary electrode because of the size of the corroding rebar (Broomfield, 1997). LPR measurements are effective in directly evaluating the rate of corrosion activity and in giving a good indication of the rate of deterioration occurring in a reinforced concrete structure.

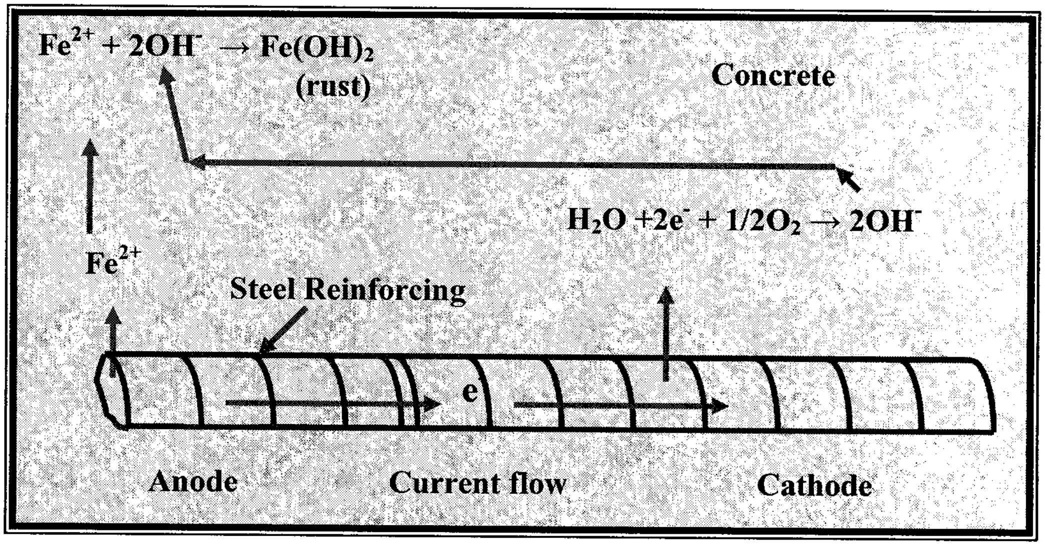


Figure 2.4 Corrosion Process of Steel in Concrete (Millard et al., 2001)

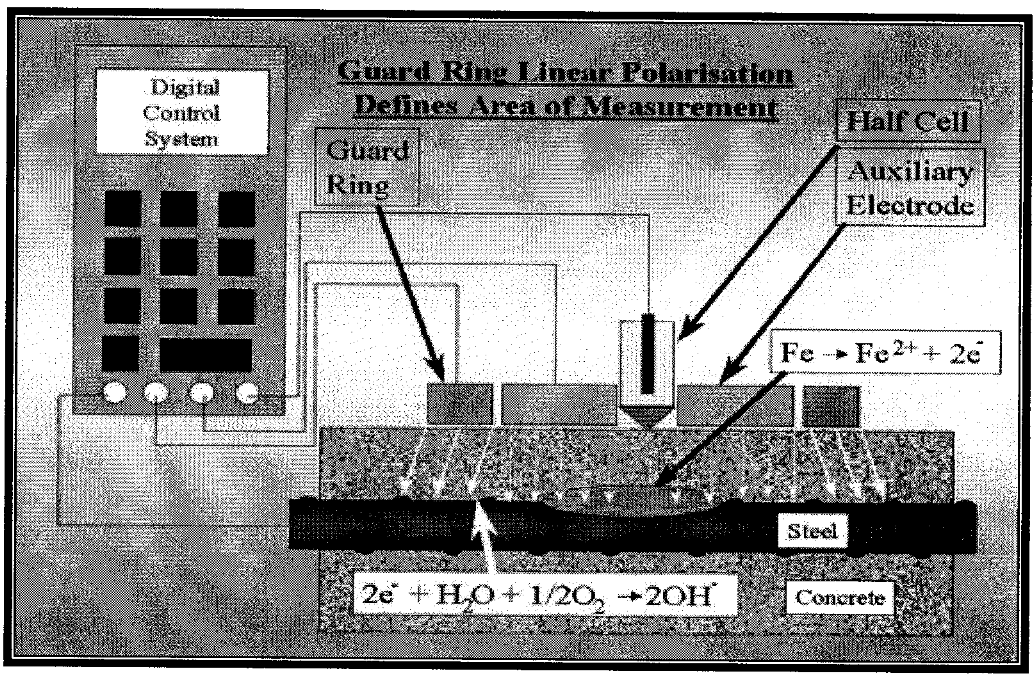


Figure 2.5 Linear Polarisation Measurement (Broomfield et al., 2002)

In order to measure the corrosion rate, the polarization resistance (R_p) must first be calculated. The polarization resistance (R_p) is inversely proportional to the corrosion current (I). The corrosion current (I) is directly proportional to the corrosion rate (i_c) (Flis et al., 1993). This is usually represented using the Stern Geary equation for determining the corrosion rate (i_c) (Stern and Geary, 1957).

$$i_c = B/R_p \qquad (1)$$

B is a constant, usually 25 mV for actively corroding steel in concrete (Hladky et al., 1989) and 50 mV for passive situations (Alonso and Andrade, 1988). Table 2.4 summarizes typical corrosion rates of steel in concrete. The polarization resistance (R_p) is usually determined experimentally. Usually, a small polarization overpotential (ΔE) between 10 and 30 mV is applied to the steel bar from the surface of the concrete and the resulting response is measured after a time (approximately 30 seconds) to ensure equilibrium has been established. The resulting current (ΔI) is recorded and shown in Figure 2.6 (Milliard et al., 1991).

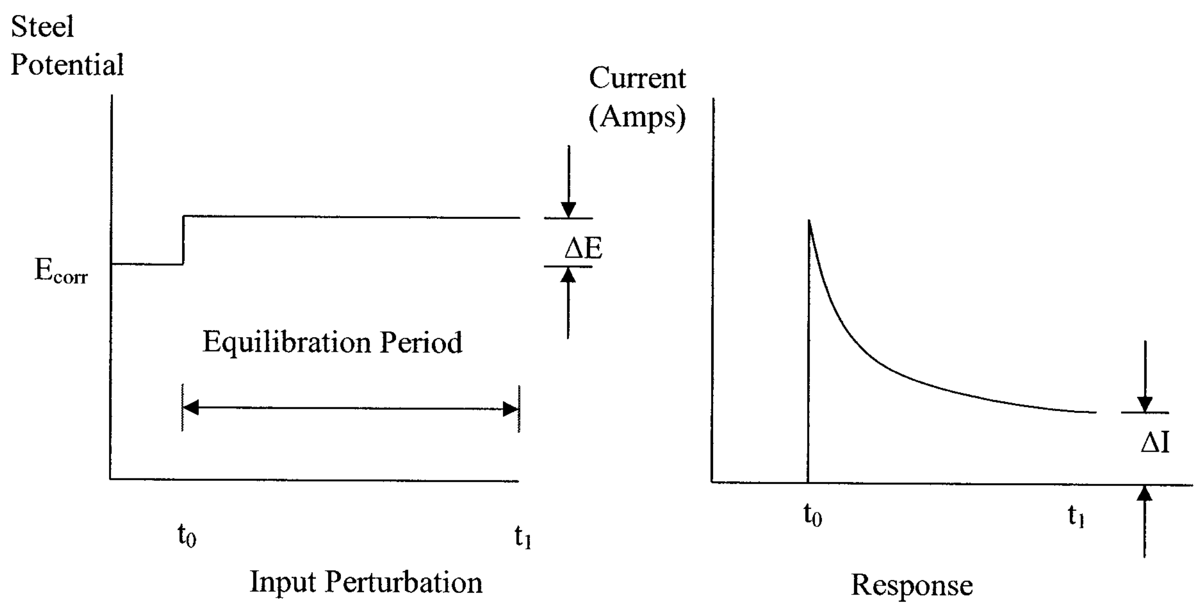


Figure 2.6 Potentiostatic LPR Measurement (Milliard et al., 1991)

The polarization resistance R_p can be found using the following equation:

$$R_p = \Delta E/\Delta I \qquad (2)$$

Once the polarization resistance (R_p) is known the corrosion rate (i_c) can be calculated using Equation (1). Once the corrosion rate is known, an estimation of the actual rebar loss (m) can be made using Faraday's Law (Broomfield et al., 1997)

$$m = MIt/zF \qquad (3)$$

Where M is the atomic weight (56g for Iron (Fe)), I is the current which is obtained by (i_c/A) (A/cm^2), A is the polarized area of the rebar subjected to overpotential, t is the time in seconds, z is the ionic charge (+2 for Fe) and F is Faraday constant, 96500 A.s.

Once the actual mass loss is known by using Equation (3) an estimation of the corrosion penetration (p) (cm/s) can be made using the following:

$$p = m/d \qquad (4)$$

Where d is the density of Iron (Fe) 7.95 g/cm^3

Table 2.4 Typical corrosion rates for steel in concrete (Milliard et al., 2001)

Rate of Corrosion	**Polarisation Resistance (R_p) (Ωcm^2)**	**Corrosion current density (I) ($\mu A/cm^2$)**	**Corrosion Penetration (p) ($\mu m/yr$)**
Very High	2.5-0.25	10-100	100-1000
High	25-2.5	1-10	10-100
Low/Moderate	250-25	0.1-1	1-10
Passive	>250	<0.1	<1

The advantages of LPR are as follows:

- Very quick method (very short measuring time).
- Small perturbation required in order to carry out the test (minimal disturbance of interface) (Flis et al., 1993).
- Simple method and low equipment cost (Flis et al., 1993).

- Once the test is carried out accurately it gives a good indication of the corrosion rate occurring in the steel reinforcing (Milliard et al., 2001).

The disadvantages of LPR are as follows:

- Polarization resistance (Rp) measurement has to be carry out properly and accurately otherwise this test will not be accurate (Flis et al., 1993).
- Surface contact between concrete and electrode should be in aqueous solution (usually water) and the polarized surface area should be known (Scully, 2000).
- The electrochemical current at the fixed potential should be constant during the entire polarization time (Qian et al., 2003).
- Since corrosion is a dynamic process it can fluctuate both up and down with changes in ambient environmental conditions (temperature change). If the degree of fluctuation is not known, or if ambient fluctuations are not compensated, then the reading obtained during the LPR measurements may not necessarily be accurate when evaluating the corrosion risk of a structure (Millard et al., 2001).

2.3.1.2 Concrete Cover Survey

A Pachometer (covermeter) is used to determine the location and thickness of concrete covering the embedded steel reinforcement. The covermeter was developed based on the principle that the presence of the steel reinforcing located in the concrete will affect the field of an electromagnet (McCann and Forde, 2001). Covermeters are electromagnetic, they consists of two coils positioned on an iron cored inductor. When alternating electric current is passed thorough one of the coil, it generates a magnetic field which propagates through the concrete and interacts with the reinforcing steel. The interaction will be due to either or both of two physical properties of steel: its magnetic permeability and/or its electrical conductivity (McCann and Forde, 2001). The interaction causes a secondary magnetic field to propagate back to the iron cored inductor where it is detected by a second coil as shown in Figure 2.7. This change in inductance depends on the type of reinforcing steel, the quantity and the distance between the steel and coil. The quantity (mass) of steel can be determined once the bar diameter is known. If the bar diameter is not known, it can be obtained by making a small hole in the concrete. With the type and quantity of steel known and constant, the distance between the rebar and the coil (concrete cover) can be determined from this change in inductance (Popel et al., 1995). Alldred (1995) has worked on developing a Covermeter which is capable of determining the rebar's diameter when it is not known, instead of using destructive techniques.

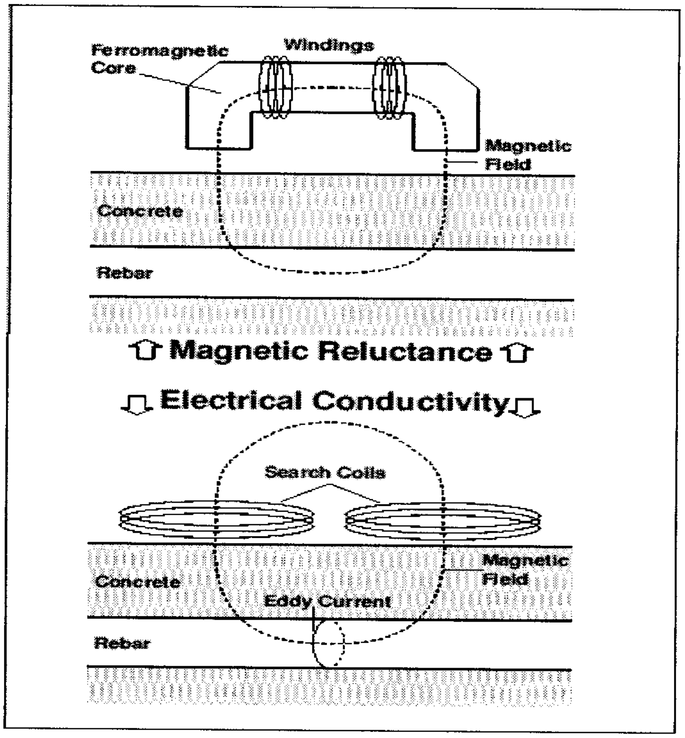


Figure 2.7 Eddie Current Generation (Alldred, 2002)

In order for accurate readings to be obtained the concrete cover is determined by rolling the covermeter across the surface of the concrete. These measured values are transferred to a laptop or a microcomputer built into the covermeter. Typical results are shown in Figure 2.8. The highest point on the curve shows the position of the rebars and these peaks corresponds to the concrete cover (Popel et al., 1995). When these peaks are detected, the covermeter creates a visible signal, thus enabling the operator to determine the exact position of the rebars.

The covermeter is a very efficient tool for locating top mat reinforcing bars. They can locate some bottom bars but it is not very accurate due to magnetic field shielding (Popel et al., 1995) and instrument limitation. The covermeter can measure concrete cover up to 100 mm in depth.

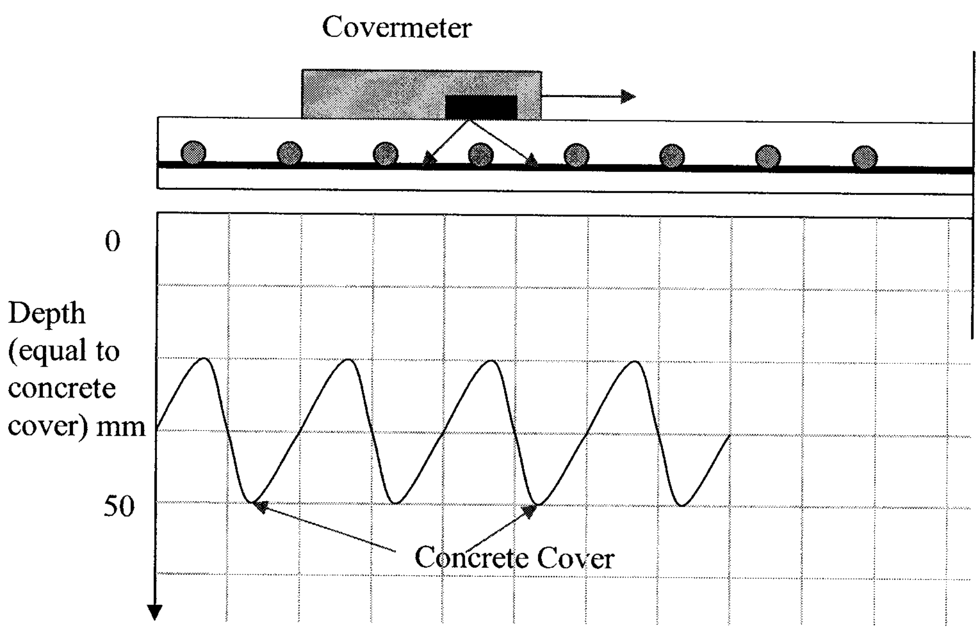


Figure 2.8 Diagram of Concrete Cover Measurement (Popel et al.,1995)

The advantages of covermeters are as follows:

- When this test is carried out accurately, with the bar diameter known, it gives a good indication of the concrete cover.
- This test can be done relatively quickly with minimum disruptions to traffic.

Some disadvantages of covermeters are as follows:

- They required destructive tests, to remove the concrete cover to determine the bar diameter, if the bar diameter is not known.
- Covermeters containing a built in microcomputer are expensive (Alldred, 2002).

2.3.1.3 Deck Evaluation using Radar

These are more commonly called ground penetrating radar (GPR) surveys and are commonly referred to as an electromagnetic investigation method. GPR uses electromagnetic (EM) waves, and it is said to be one of the most promising techniques for inspection, quality control and early deterioration detection (Loulizi and Al-Qadi, 1997). The principle of this technique is simply transmitting EM waves into a non magnetic body (Cardimona et al., 2000) and receiving the reflected signal from that body. This technology was first used by German researchers in the early 1900s for detecting buried objects (Loulizi and Al-Qadi, 1997). It has since been developed over the last thirty years for use on bridge decks to detect rebar locations and/or concrete delaminations (Cooke et al., 1993).

There are generally two types of GPR systems; air coupled systems and ground coupled systems. Air coupled systems or horn antennas as they are commonly called, are used approximately 150-500 mm above the concrete surface. A ground couple system consists of a transceiver (a device that's transmits and receives electromagnetic signals) that is in full contact with the ground. Air coupled antennas are suited for driving speed (>50 km/hr) measurements whilst ground coupled antennas are suited for deeper penetrations and object detection where survey speed is not critical (Maser, 1996).

Air coupled systems are ideally suited for bridge inspections because they minimize traffic disruptions. Air coupled systems are mounted on vehicles as shown in Figure 2.9. These antennas emit signals into the structure and they have the ability to retrieve data readings from the deck slabs. When these tests are conducted it usually takes several longitudinal passes over the bridge deck for the radar antennae to completely emit signal throughout the entire deck surface. Each antennae pass is normally 2 to 3 feet in width (Daniels, 1996). This technique gathers a high volume of data acquisition in a short period of time, with minimized traffic flow obstructions.

GPR can emit and retrieve electromagnetic waves at depth of 500 mm or more. It primarily depends on the antenna used and the concrete type (Popel et al., 1995).

Different antennas emit and record different ranges of frequencies. High frequencies antennas result in high resolution data, but they have low penetration depth. Low frequency antennas have greater penetration depths; however they have lower resolutions (Hugenschmidt, 2002). Many GPR systems used today have antennas with frequency ranging from approximately 50 MHz to 1.5GHz. For bridge inspections, usually antennas with frequencies above 1 GHz are used (Daniels, 1996).

Figure 2.9 Air coupled GPR System

GPR's electromagnetic waves are used to detect echoes and reflections in areas within the concrete where delamination is suspected. This is illustrated in Figure 2.10. If the concrete is sound and there is no concrete delamination occurring, then no reflections or echoes will be observed on the monitor. The reflected signal or echoes from a rebar reinforcement and different internal layering can be directly associated with the amount of deterioration. Lower signal strength generally indicates more deck deterioration is present (Cardimona et al., 2000).

Delaminated areas are detected as reflected energy caused by the changes in material properties, they are recorded and analyzed (Daniels, 1996). After the data has been gathered, it is followed by data processing and interpretation (the information is extracted from the radar and converted into a meaningful format) (Hugenschmidt, 2002) (Figure 2.11).

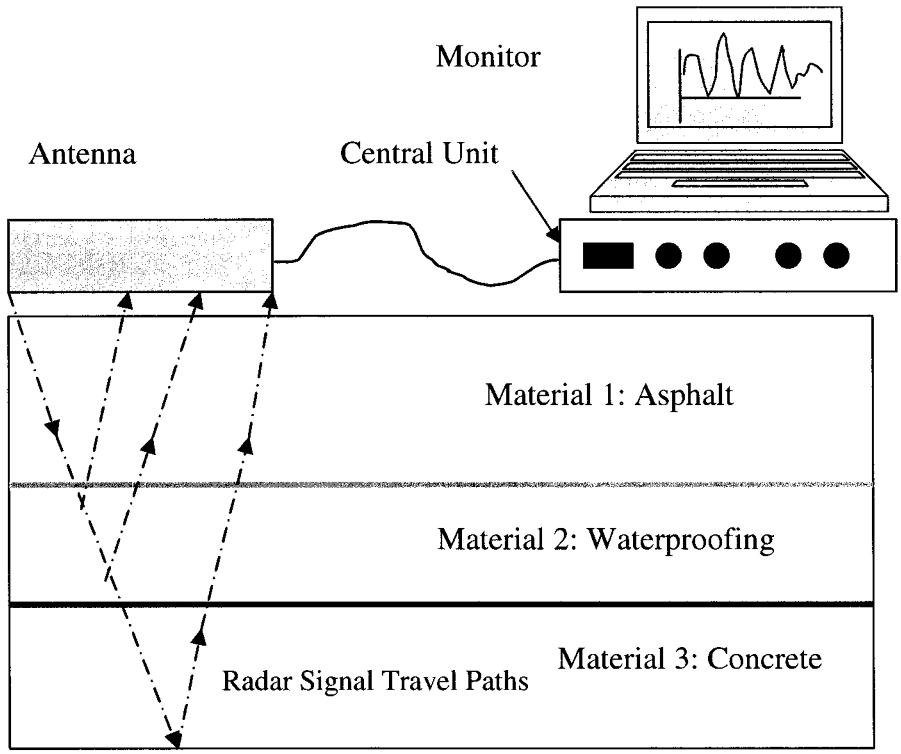
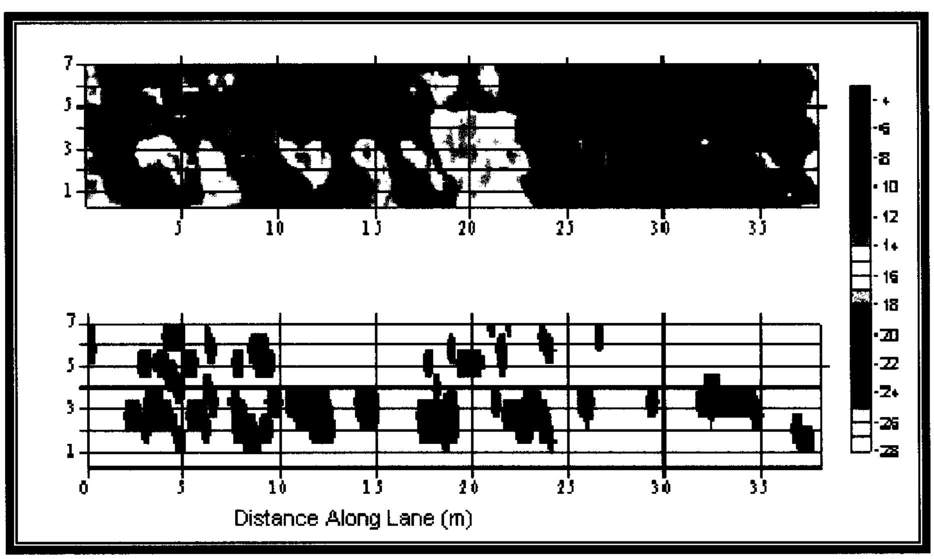


Figure 2.10 Schematic Illustration of a GPR System



Top Map: Plan view of predicted deterioration
Red = Most deteriorated, Blue = Least deteriorated
Bottom Map: Black areas = actual areas of delamination

Figure 2.11 Typical Example of Extracted data from the GPR Survey

GPR is also a very useful technique to use on asphalt covered bridge decks. It can determine the asphalt layer thickness (T) quite accurately. The use of GPR for asphalt thickness evaluations is based on measurements of time difference (t), between different layer reflections (air, asphalt, waterproofing membrane, concrete) and the radar wave velocity (v) (mm/ns) in each layer (Maser, 1996). If these two are known, the thickness of the asphalt layer (T) can be calculated using the following equation.

$$T = vt/2 \qquad (5)$$

Where t is the round trip travel time between layer reflections (ns). The radar velocity in each layer is calculated from knowing each layers dielectric constant (Є).

$$v = 150/\sqrt{Є} \qquad (6)$$

The dielectric constant of an asphalt layer relative to the previous layer (air) can be calculated by measuring the amplitude of the waveform peaks corresponding to the reflection from the interfaces between layers (Barnes and Trottier, 1999). Since air is the first layer, it has a known dielectric constant of 1 (Maser, 1996). In general the dielectric constant Є and the reflection coefficient ρ are related by

$$Є = [(1-\rho)/(1+\rho)]^2 \qquad (7)$$

A similar approach is used to calculate the concrete cover on a concrete deck slab. In order to determine the concrete cover on asphalt covered decks the entire reflected waveform must be examined. Figure 2.12 shows a typical waveform. The equation for this waveform is as follows:

$$r(t) = \rho A\, s(t) + \rho C\,(1-\rho A^2)\, s\,(t-2T_A) + \rho R\,(1-\rho C^2)\,(1-\rho A^2)\, s\,(t-2T_A - 2T_C) \qquad (8)$$

Where s(t) is the transmitted signal shown in Figure 2.13. ρA is the reflection coefficient of the air/asphalt interface; ρC is the reflection coefficient of the asphalt/concrete interface; ρR is the effective reflection coefficient of the rebars; T_A is the time delay for the signal to propagate from the asphalt surface to the concrete surface; and T_C is the time delay for the signal to propagate from the concrete surface down to the rebar layer (Chung et al., 1990).

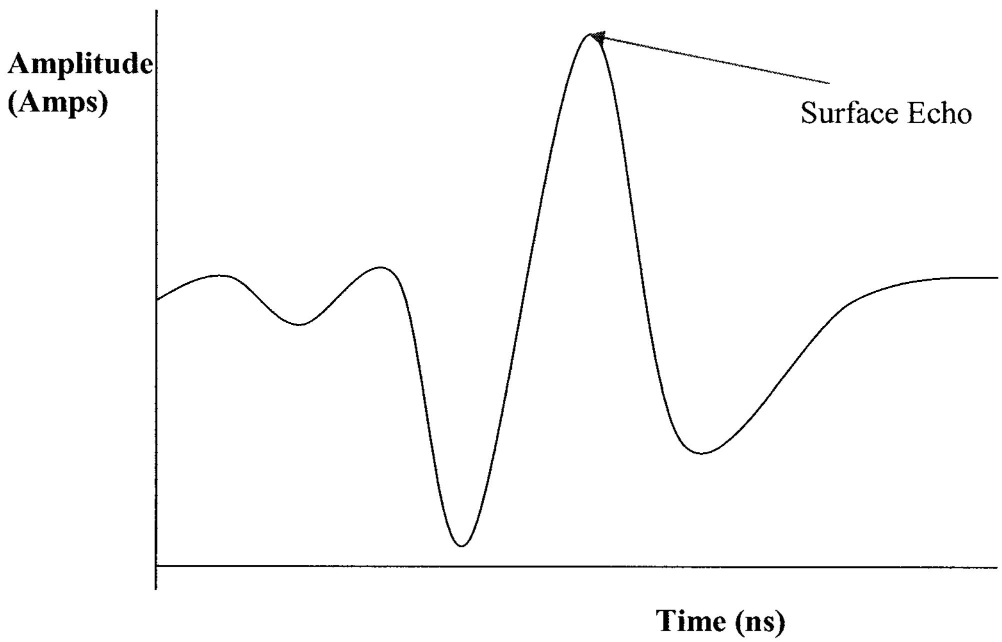


Figure 2.12 Transmitted Impulse Radar Waveform

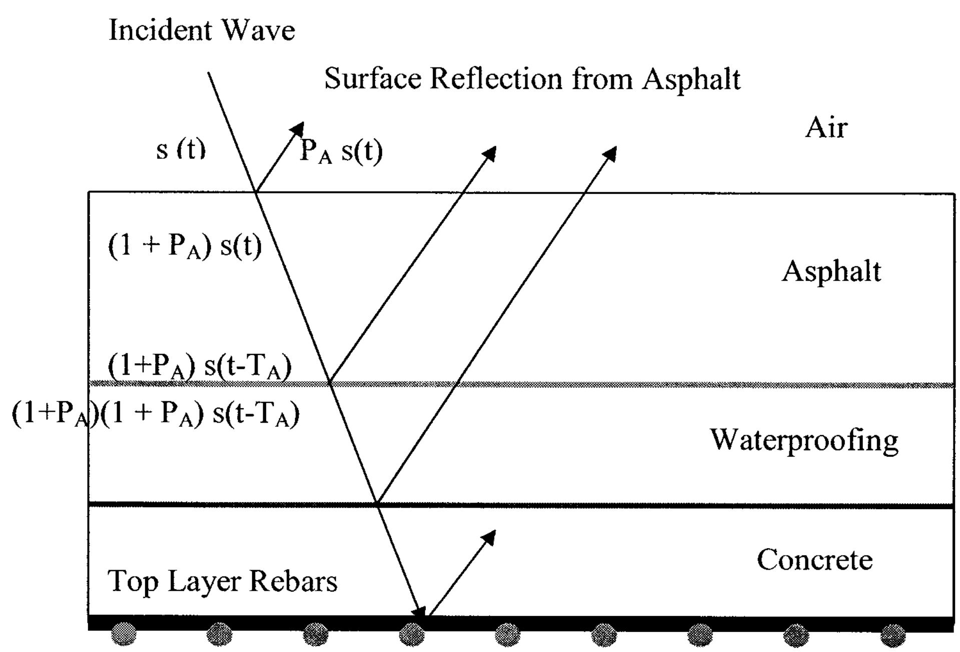


Figure 2.13 Reflections from Bridge Deck Surface (Chung and Carter, 1993)

The dielectric constants can be calculated by measuring the time delays and amplitudes of the peaks from the reflected signal (illustration shown in Figure 2.14). The thickness of the asphalt layer or concrete cover over the reinforcement can be calculated by combining equations (5) and (6) (Chung and Carter, 1993).

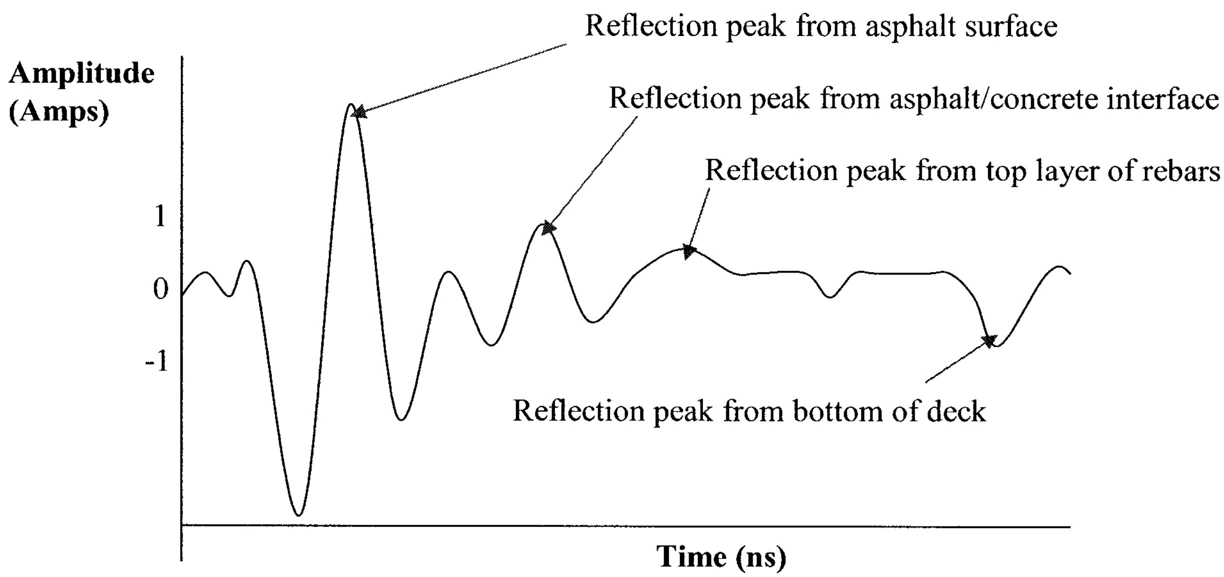


Figure 2.14 Reflection from a Typical Bridge deck (Chung and Carter, 1993)

In an investigation carried out by Chung et al. (1990), they compared the radar measured asphalt thickness with the actual thickness using core samples. Chung et al. (1990) confirmed an accuracy of ±5 to ±8 %. Chung et al. (1990) also concluded that when the asphalt thickness is less than 30 mm, the surface reflection and asphalt/concrete boundary interface reflection interfere with each other, reducing the accuracy of this technique.

The advantages of GPR Systems are as follows:

- GPR systems take measurement very quickly, they don't disturb traffic flow and it can be done at regular intervals to keep monitoring the structure (Barnes and Trottier, 2000).
- This method can determine overlay thickness and concrete cover thickness and is important in bridge management systems. Knowing there thickness aids in planning effective rehabilitation techniques. It also gives an indication into the structures remaining service life (Loulizi and Al-Qadi, 1997).

The disadvantages of GPR are as follows:

- This method acquires data quickly, but processing this data is time consuming if done manually. It is recommended to use a specialized data processing software or program (Maser, 1996).
- If the antenna frequency is less than 1 GHz, it will not be powerful enough to detect air void delaminations occurring in bridge decks (Maser 1989). It is recommended to use higher frequency antennas (1-6 GHz), with shorter wavelengths and higher resolutions.
- This method is only moderately successful in its ability to estimate the percentage of deck area that is deteriorated (it gives very mixed results in terms of its ability to accurately locate deterioration quantities on a deck and in defining their boundaries (Halabe et al., 1997).

2.3.1.4 Impact Echo Technique

The Impact Echo (IE) technique was originally developed for analyzing defects in foundations (Davis and Dunn, 1974), but since the 1980s it has been developed for use on concrete slabs (Pederson and Senkowski, 1987). The IE method can be used to locate cracks, voids, honeycombing and delaminations in concrete bridge decks (Sansalone and Streett, 1997).

The IE technique is used when a detailed analysis of the bridge deck is required, as opposed to using the chain drag method which is quick and inexpensive and only gives an opinionated assessment which can be inaccurate. The IE technique is very time consuming and expensive, but it gives accurate results, making it a very accurate method in determining areas of delaminating concrete (Scott et al., 2002). The IE technique is a stress-wave method. It is performed on a point by point basis by using a small impulse hammer to hit the surface of the deck at random locations and recording the reflected energy obtained (McCann and Forde, 2001) as shown in Figure 2.15. This stress pulse propagates into the object along a spherical wave fronts as compression (P) waves and shear (S) waves (Sansalone et al., 1987). These P and S waves are reflected by internal cracks and voids and the boundary of the objects being tested (Carino and Sansalone, 1990). In addition, there is also a surface (R) wave which travels along the surface away from the impact point (Carino and Sansalone, 1990).

When the concrete surface is impacted, a transducer mounted on the same concrete surface receives a longitudinal wave reflection if there are discontinuities within the concrete (Scott et al., 2002). Whenever the receiver is placed to close to the impact point, the displacement waveform is dominated by displacements caused by P wave's arrivals (Sansalone et al., 1987).

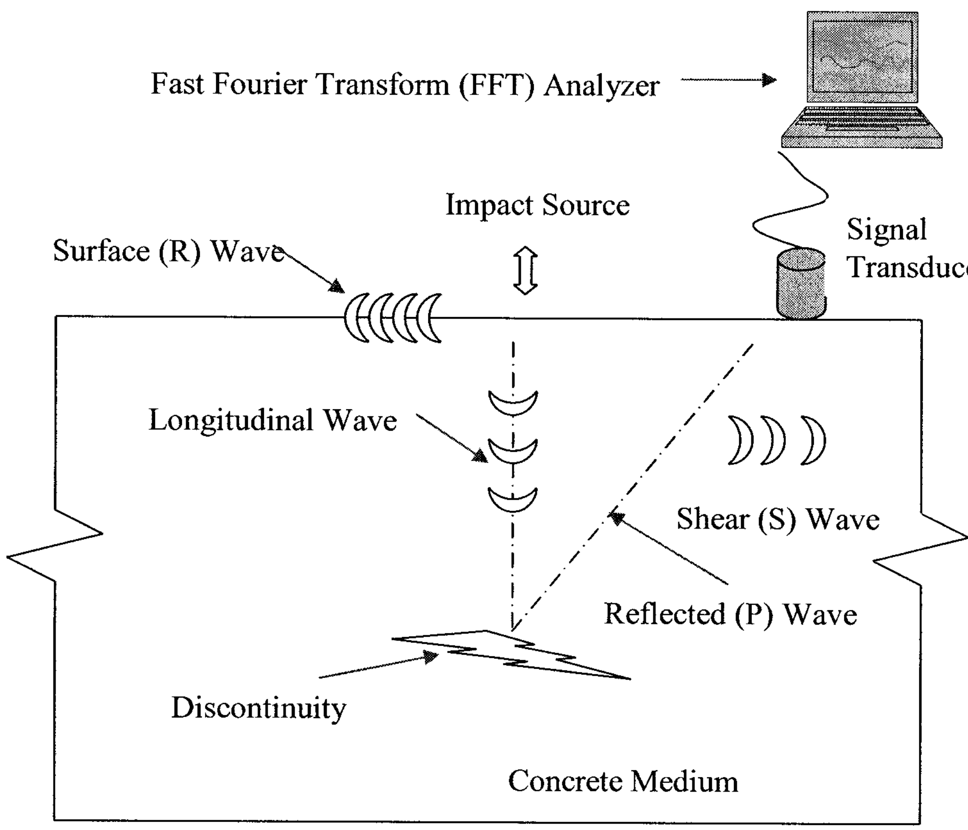


Figure 2.15 Schematic Illustration of the Impact Echo Technique

The IE system is made up of three components. The impact source, usually a hammer is used, a receiver which operates at a nominal resonance frequency of approximately 1 kHz and a waveform analyzer usually a portable notebook computer with a data acquisition card (Scott et al., 2002).

A spectral analysis using a Fourier transform algorithm such as a fast Fourier transform (FFT) is performed on the reflected P waves to determine the internal characteristics of the concrete at that particular location (Scott et al., 2002). The FFT analysis converts the time domain signal into a frequency domain function. A typical frequency (f) domain spectrum is shown in the Figure 2.16. This spectrum is evaluated to determine the location of discontinuities or to determine the thickness (T) of the material.

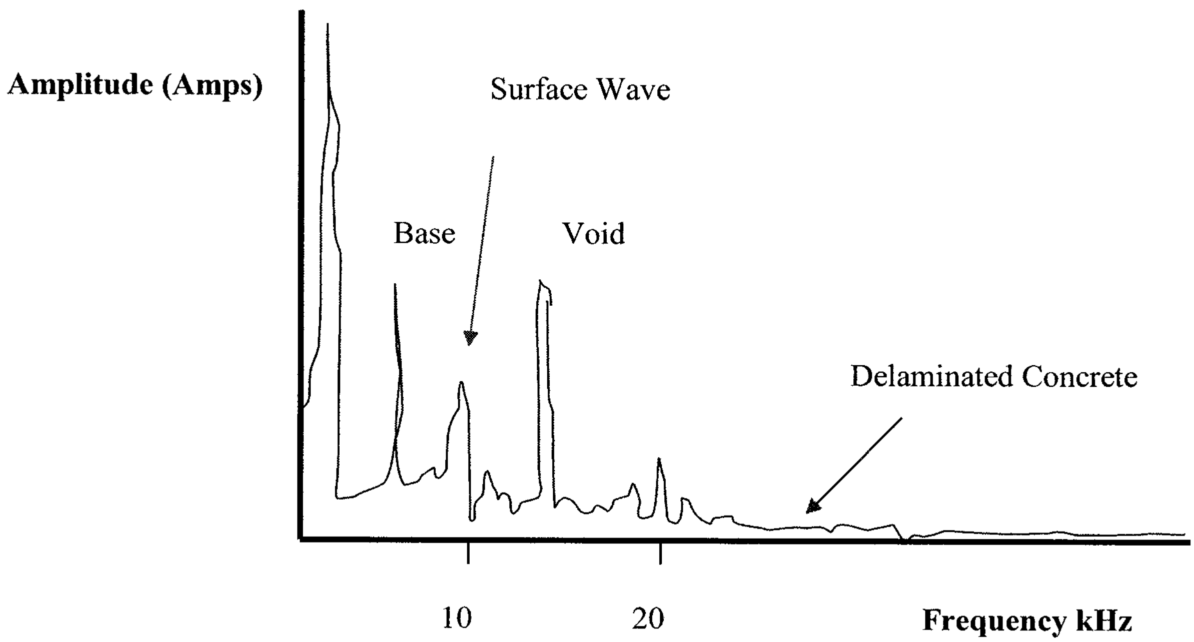


Figure 2.16 Frequency Spectrum Obtained after Impact with Hammer

Whenever a signal is emitted through the material, stress waves propagate at finite velocities, this is known as the compression wave propagation velocity (C_p). The thickness of the material is calculated using the following equation:

$$T = C_p/2f \qquad (9)$$

The advantages of the IE method are as follows:

- This system has been used successfully in the laboratory as well as on site detecting defects in concrete (Sansalone et al., 1986).

- It is a very accurate method if done properly. It gives a good indication of the amount of concrete deterioration that is occurring in the structure.

The disadvantages of the IE method are as follows:

- It is very time consuming, which makes it almost impractical to be used on major arterials unless there are lanes closures or complete closure of the bridge deck for several hours.
- Sometimes there is a reduction in frequency of the impact echo signal. This is due to the "crumbling" of the concrete surface. This results in longer contact time and lower frequency (Martin and Forde, 1995).

2.3.2 Destructive Evaluations

Before destructive testing can begin, core samples are required. Cores are taken from bridge decks and should have a diameter of at least three times the coarse aggregate size (ASTM C42, 1986). Core drilling usually weakens the bridge deck (Minor et al., 1988). Determining which areas to drill for core samples is critical as core samples are limited. Usually, the bridge inspector in charge uses his/her expertise (visual inspection) and some non destructive techniques to determine which areas should be cored for possible signs of deterioration. It is a good idea to perform a covermeter survey as discussed in section 2.2.1.2, to determine the concrete cover over the top mat of reinforcing steel to avoid coring through it. However, when an area along the bridge deck has a high corrosion potential (indicated by the results from the corrosion potential survey), cores are taken through the steel reinforcing to observe condition of the rebar even though the concrete may still be sound (MTO, 1996). It is important to note that the number of cores depends on the result of the corrosion potential survey.

After the coring operation has been completed, the concrete cores are taken to the laboratory and tested. Concrete cores are used to carry out, three main tests; air void content, compression strength test and chloride ion test. Air void content test is usually carried out when the deck shows signs of scaling. If the results yield an air content exceeding 3%, the concrete is considered to be properly air entrained (Minor et al., 1988). Generally two types of tests are available to determine the air void content. ASTM C 457 tests for microscopical deterioration of air void content (ASTM C 457, 1986). ASTM C 642 tests for specific gravity, absorption and voids (ASTM C 642, 1986).

The compressive strength test is used to identify the high and low strength areas and the concrete variability throughout the deck. Usually, an area of high strength indicates good sound concrete, whilst an area of low strength yields poor delaminated concrete. The ASTM C 42 test is used on concrete core samples to determine the concrete compressive strengths (ASTM C 42, 1986).

For the chloride ion test, the concrete cores are analyzed at various depths (25, 50 and 125 mm) to determine the amount of chloride penetration into the concrete. This test also gives a good indication of the concentration of the chlorides at these various depths. There are three tests that can be used to determine if chloride ion are penetrating on a concrete surface. They are follows:

- AASHTO T 259 is a test that determines the resistance of the concrete specimens to chloride ion penetration (AASHTO T 259, 1986).
- ASTM C 672 tests for concrete resistance to deicing salts (ASTM C 672, 1986).
- AASHTO T 277 tests for permeability of conventional Portland cement and special concrete such as latex modified, polymer concrete etc, (AASHTO T 277, 1986).

Concrete cores are also visually inspected to determine the amount of delamination present and scaling of the cement matrix and aggregate as a result of the coring process. Additional cores are normally taken where deterioration is expected but not visible with the naked eyes i.e. near curbs, areas of poor drainage, in areas of high corrosion potential, in areas of delamination and at cracks in the asphalt surface (MTO, 1996).

3.0 Concrete Removal Techniques

3.1 Introduction

Chapter 3 outlines various demolition methods and removal techniques commonly used in bridge rehabilitation projects throughout North America. The choice of which technique/method to use usually depends on the following factors:

- Financial;
- Time limit (Project speed);
- Strength and Quality of concrete;
- Shape, size and accessibility of the structure;
- Amount of concrete to be removed;
- Environmental concerns, including noise, dust, vibrations and debris;
- Worker safety and Public safety;
- Possible recycling of concrete; and
- Removal, transport and disposal of debris.

In many urban cities, the main concern when carrying out a rehabilitation project is preserving good public relations during traffic disruptions. Keeping lanes open during repair/removal or performing a speedy demolition/removal to prevent traffic problems on roadways running below the structure are all factors considered in the decision making process of which choice of repair/removal technique to use. Also, in these urban areas restriction on noise, dust or vibration may also be imposed on the project.

3.2 Types of Concrete Removal Techniques

There are three common types of concrete removal techniques available today, which are described in the following sections.

3.2.1 Hand Held Pneumatic Breakers (HHPB)

Hand Held Pneumatic Breakers are the most widely used and well established tools for removing contaminated or/and deteriorated concrete. They are light weight and have excellent maneuverability which benefits workers in removing damaged concrete from small, isolated areas and from vertical and overhead surfaces on all bridge structural elements (Emmons, 1993). They are generally used on cracked, spalled or delaminated concrete, and on chloride contaminated concrete when the depth of removal is known.

HHPBs can be powered by a variety of energy sources, including pneumatic pressure, hydraulic pressure, gasoline engine or electric motor. Table 3.1 summarizes some of the advantages and disadvantages of pneumatic breakers powered by various energy sources.

Table 3.1 Advantages and Disadvantages of Breakers powered by different energy sources (Vorster et al., 1992)

Energy Sources	Advantages	Disadvantages
Hydraulic	Higher Production	Primarily for demolition
	Greater Impact Energy	Damages residual concrete
		Requires hydraulic compressor
Gas/Electric	Self Contained	Greater weight
	Suitable to sites with limited access	More complicated than Hydraulic
		Higher Cost
Pneumatic	Light weight	Requires air compressor
	Durable	

HHPBs are used more often because they are more effective and economical than any other breakers powered by other energy sources. HHPBs can be made in a variety of weights. A 9 kg hammer is used for small chipping used for light duty applications. Small chipping hammers are limited to less than 20 kg so that they may be safely used on vertical or overhead surfaces (Emmons, 1993). However breakers of 45 kg are manufactured, but they are used for projects requiring high production rates. The weight of the breakers is governed by the quality requirements of the job and the weight which can be handled by the operator with ease and safety on horizontal surface (Vorster, et al., 1992).

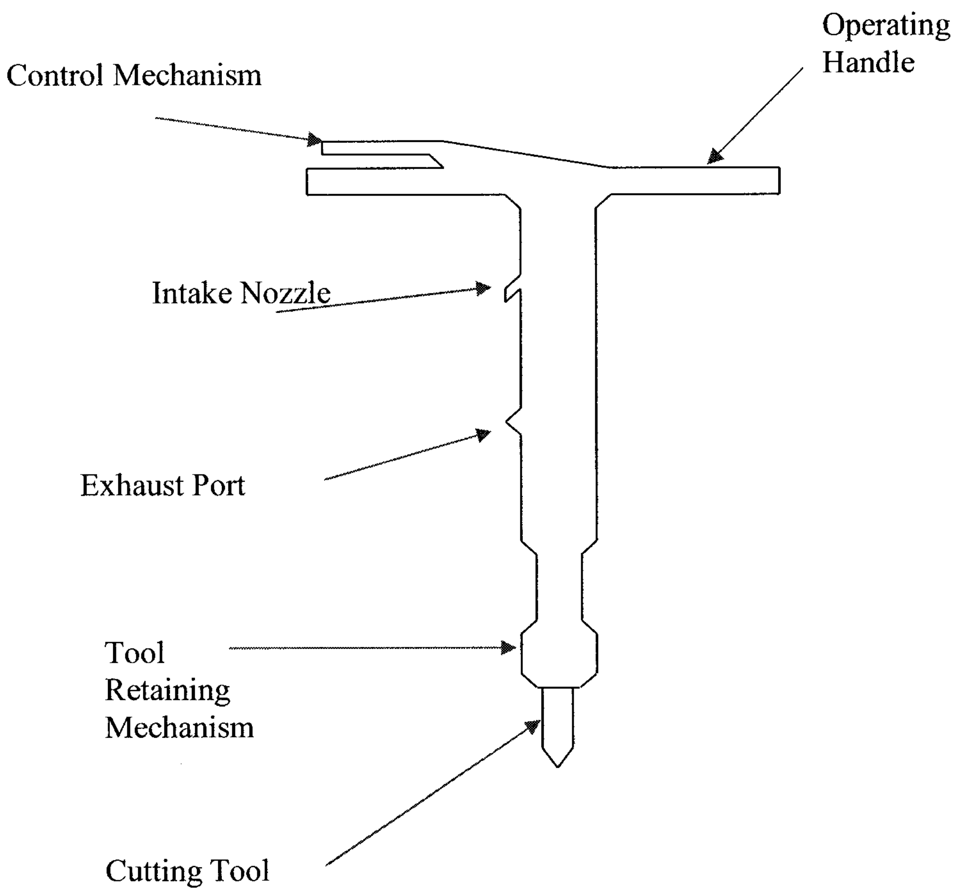


Figure 3.1 Hand Held Pneumatic Breaker

3.2.1.1 Uses of Hand Held Pneumatic Breakers

HHPBs are generally utilized to varying degrees in all types of bridge rehabilitation projects. The project type for which breakers are most effectively utilized is determined by the location of the concrete to be removed, the location of the bridge, the speed of the project and the availability of manpower.

HHPBs are commonly used on projects involving patching, the removal of concrete from bridge structural elements that are not accessible to larger equipments and in small areas from between, around and below the reinforcing steel bars. They are well suited to operate in congested urban areas with high traffic volumes. In these areas, they have a big advantage over other methods in terms of set up time and space. Since pneumatic breakers are often of small size and have a high degree of mobility, they are suited to projects that have a limited working window. However, they have the disadvantage of being very labour intensive.

3.2.1.2 Deterioration Type and Extent

HHPBs are one of the few methods available for removing concrete from vertical and overhead surfaces (beams, girders, and piers) (Vorster, et al., 1992). Usually a small HHPB weighing 20 kg is used for these types of applications; however the weight of the breaker will have an effect on the operator performance when working on vertical or overhead surfaces. HHPBs also have the advantage of being able to selectively removing only damaged/contaminated concrete. This is usually effectively accomplished when a skilled operator can distinguish between various levels of concrete deterioration by the resistance of concrete to the breaker (Vorster et al., 1992). This allows the operator to selectively remove only concrete which is deteriorated and minimizes the removal of sound concrete.

3.2.1.3 Material/Area/Depth to be Removed

HHPBs are primarily used for removing concrete that is cracked or delaminated. They can also be used to remove contaminated as well as sound concrete. They are also used in support of other removing techniques such as milling and splitting (Mallet, 1996). HHPBs are generally used to remove small areas of concrete; this small removal area is the primary factor defining its working environment. Larger removal equipment used for techniques such as milling and hydrodemolition are restricted to only removing concrete from areas which have a definite boundaries and dimensions as dictated by the shape and size of the machines (Vorster et al., 1992). HHPBs have no limitations and are capable of removing small and irregular sections of pavement. The small cutting tool allows the HHPB to effectively remove concrete from confined spaces such as around, between and under reinforcing steel in the concrete matrix.

3.2.1.4 Quality Control

In order to ensure the success of a project, targets/objectives must be established and maintained to ensure that the HHPBs operators achieve their desired level of quality. There are five main quality targets/objectives to ensure success, they are as follows:

3.2.1.4.1 Removal of Deteriorated Concrete

If the operator is not experienced enough, he/she may not be able to effectively remove deteriorated concrete, and as a consequence the actual quantity of concrete removed may vary from the estimated quantity resulting in a loss for the contractor (Vorster, et al., 1992).

3.2.1.4.2 Residual Concrete Damages

When the HHPB is used to fracture and remove the deteriorated or contaminated concrete, the impact forces may produce microcracks in and immediately below the surface of the residual concrete. These cracks will in time accelerate the deterioration of the residual concrete, resulting in weak bonds being formed between the residual concrete and new overlay/patch material. To avoid this, the breaker weight has been limited to 16 kg and there impact angle of the surface of the concrete be between 45 and 60^{o} (Strategic Highway Research Program, 1992).

3.2.1.4.3 Reinforcing Steel Damages

The impact force generated from the HHPBs to fracture the concrete is even strong enough to damage the steel reinforcing or the bond between the concrete and the steel ((Vorster, et al., 1992). When inspected, if the cross-sectional area of the reinforcing bar is substantially reduced (by corrosion or gouging caused by the breaker) then the entire damaged section must be replaced. All the steel reinforcing bars must be cleaned and free of chloride ions, this is accomplished using either a wire brush or doing sandblasting.

3.2.1.4.4 Surface Characteristics

The HHPB produces a surface which is rough and irregular in shape. This has the advantage of allowing good bonding between the old and new surfaces. However, because of its irregular shape, the surface cannot be opened to traffic until it is resurfaced.

3.2.1.4.5 Environmental Concerns

Careful consideration is required to ensure the HHPB causes minimal impact on the surrounding environment. The main environmental concerns are usually dust, noise and flying debris.

3.2.2 Milling

Milling is a capital intensive method (that is very little manpower is involved) which uses high production machines to strip contaminated and deteriorated concrete from above the reinforcing steel. This removal technique is ideally suited for bridge deck removal of large volumes of concrete above the reinforcing steel. If the removal of concrete below the reinforcing is required, HHPBs are required.

The heavier machines are able to exert a greater down force to keep the machine grinding whilst increasing production. Milling machines can run on either tyres or tracks which are powered by a hydrostatic motor. There is a big advantage when using tracks, it has the ability to carry heavier loads and exert increased traction. They also distribute the milling machines weight over a larger area and this reduces the possibility of exceeding point load limits on any particular location. Today, the majority of milling machines are equipped with a conveyor system for removing the debris and discharging it into loader trucks. Figure 3.2 illustrates a typical example of a milling machine.

From Figure 3.2, the main function of the cutting mandrel is it is a cylindrical metal drum mounted horizontally on the underside of the milling machine. It holds the cutting teeth used to break the concrete. These cutting teeth are made of Carbide-Tungsten and are approximately 76 mm long. The milling machine accuracy depends on the stiffness of the cutting teeth (Lee and Ko, 2001).

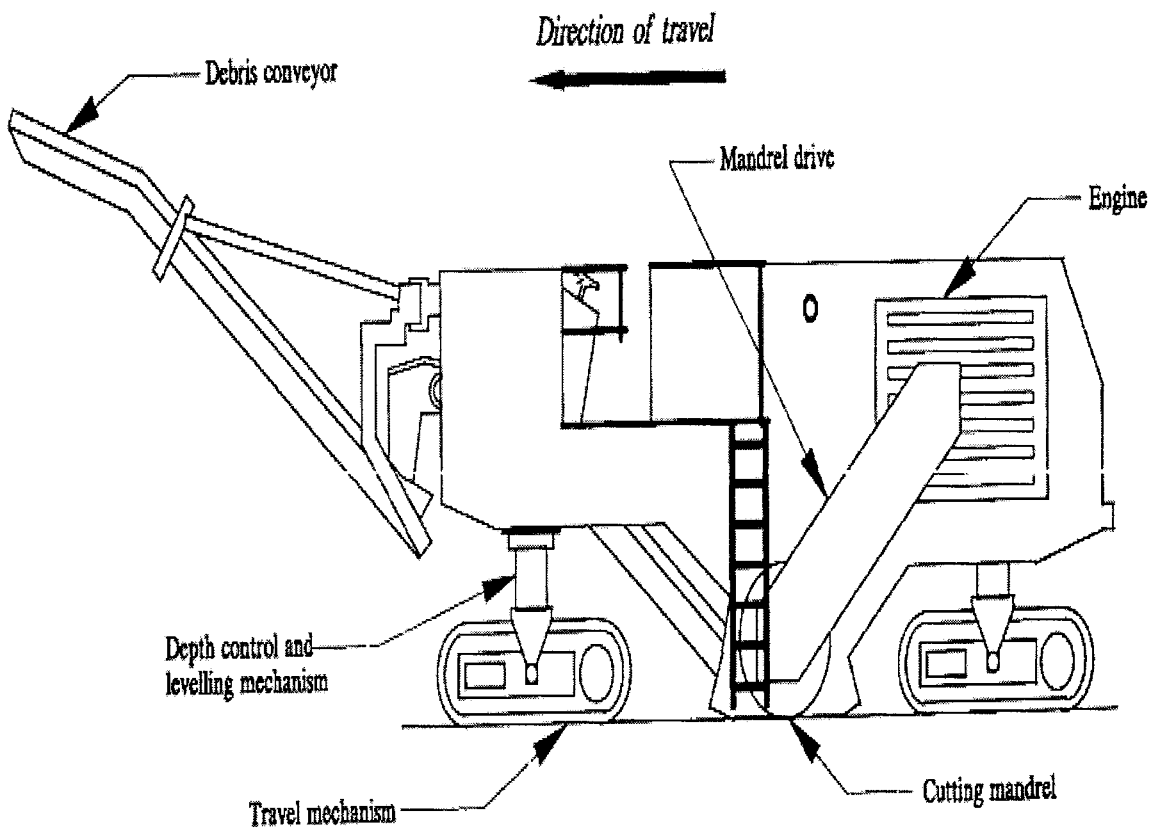


Figure 3.2 Milling Machine (Vorster, et al., 1992)

These machines can be altered to any depth above the reinforcing steel. Operators can control the depth of cut by adjusting the height of the machine as a whole or they can adjust the height of the cutting mandrel relative to the machine. Generally, the larger and heavier milling machines are able to achieve the required depth of cut in one pass, but the lighter, smaller and less powerful machines may require several passes (Martellotti, 1945). Figure 3.3 illustrates the cutting action of the milling machine.

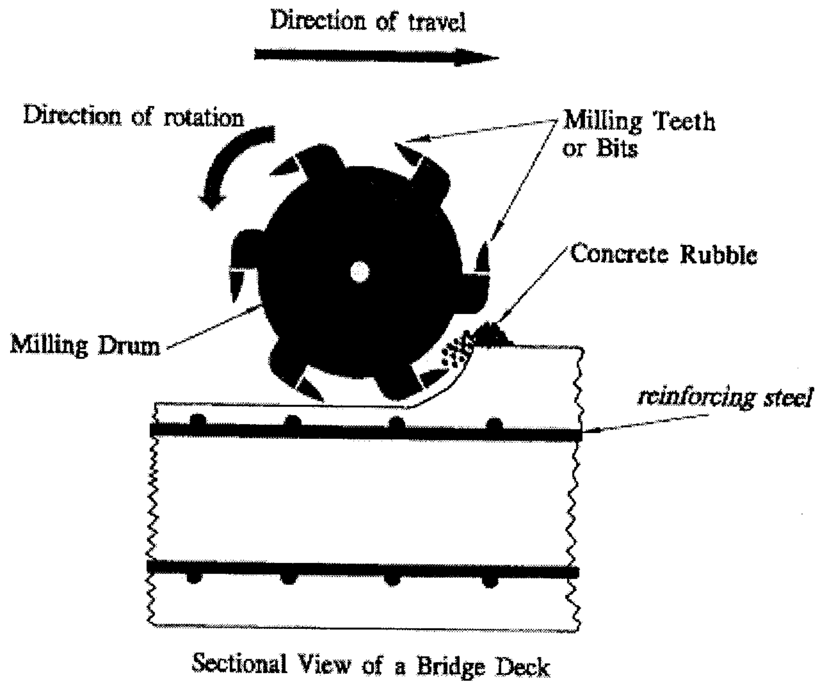


Figure 3.3 Illustration of how the Milling Process Occurs (Vorster, et al., 1992)

3.2.2.1 Uses of Milling Machines

These machines are used on decks when the entire surface must be removed above the reinforcing steel. When a larger area is milled at once, greater equipment utilization and reduced mobilization costs are achieved (Kline et al., 1982). These machines are not designed for projects requiring small area of removal. The location of the project is an important factor determining the speed of the project (Vorster, et al., 1992). For urban projects, operating hours are often restricted and work maybe permitted only at night. Rural jobs have better access and fewer limitations on operating hours but may be more costly in terms of mobilization.

3.2.2.2 Materials/Area/Depth to be Removed

In order to ensure accurate depth of removal any previously placed asphalt overlay must be removed prior to the concrete removal operation. Milling machines can remove both types of overlay material. It is generally much easier for milling machines to break the bond between cement paste and aggregate than to fracture the aggregate (function of the pneumatic breaker) (Sutherland and Devor, 1986). The area to be removed is the main factor affecting milling machines efficiency and productivity. When the machine works continuously, it results in high production rates and low unit costs.

3.2.2.3 Quality Control

This section will focus on five issues concerning requirements which ensure that the desired quality is achieved.

3.2.2.3.1 Machine Size

If the machine is off the wrong size to perform the work, it can result in a product of inferior quality. When a machine is too small for a required project, it will not have the necessary production rate to be economical and can also damage the residual concrete if the machine is pushed beyond its capability. Care and precaution must be exercised when using larger machines, as their weight and vibrations can easily damage the remaining deck or other structural components quite easily.

3.2.2.3.2 Residual Concrete Damages

When milling machines are used there is always a small chance they can produce a layer of damaged concrete with small cracks ranging in lengths between 12 and 18 mm due to vibration (Vorster, et al., 1992). These microcracks reduces the strength of concrete, lower the bond between concrete and steel and reduce the bond between old and new overlay materials.

3.2.2.3.3 Reinforcing Steel Damages

Generally, when a project requires milling close to the top reinforcing steel, the machine should proceed slowly and under close control of the operator. If the machine catches a bar, it pulls it out of the concrete, resulting in damages to the machine's cutting teeth and cause extensive damages to the deck. To avoid such a disaster, it is more cost –effective to use milling machines to remove concrete to a reasonable depth and then use HHPBs to remove any remaining damaged concrete.

3.2.2.3.4 Surface Characteristics

After the milling process, it leaves the residual concrete with a rough textured finish which can be opened to traffic immediately or covered with overlay material. The grid pattern produced by milling allows the new overlay material to interlock and bond very well with the old material.

3.2.2.3.5 Environmental Concerns

This removal technique generates a lot of dust. Debris removal operations often obstruct the vision of both the operator and motorists. Noise is another main concern in heavily populated areas so caution is exercised to keep it at a desired level.

3.2.3 Hydrodemolition

Hydrodemolition was first used in 1979 in Italy (Warner, 1998). It was first introduced in Canada and in Sweden in 1984 (Warner, 1998). Hydrodemolition is a relatively new technology that uses complex equipment to produce and direct a high pressure water jet to erode the cement matrix between the concrete aggregate (Vorster et al., 1992). This technique is very effective in attaining high rate of production while selectively removing deteriorated or contaminated concrete and also in cleaning the reinforcing steel as well as preparing the surface for an overlay (Silfwerbrand, 1990).

This technique requires two distinct components; a power unit and a demolishing unit. The power unit consists of a drive engine, high pressure pump, water filters, water reservoir tank and other ancillary equipments. The water supplied to the power unit is passed through a series of filters before it is stored in a reservoir tank. The purpose of these filters is to remove solids from the water to prevent excessive wear on the system.

The demolishing unit used for bridge decks is a microprocessor controlled wheeled vehicle shown in Figure 3.4. The microprocessor controls the movement of the nozzle to ensure precise control of the water jet to provide consistent quality (Vorster et al., 1992).

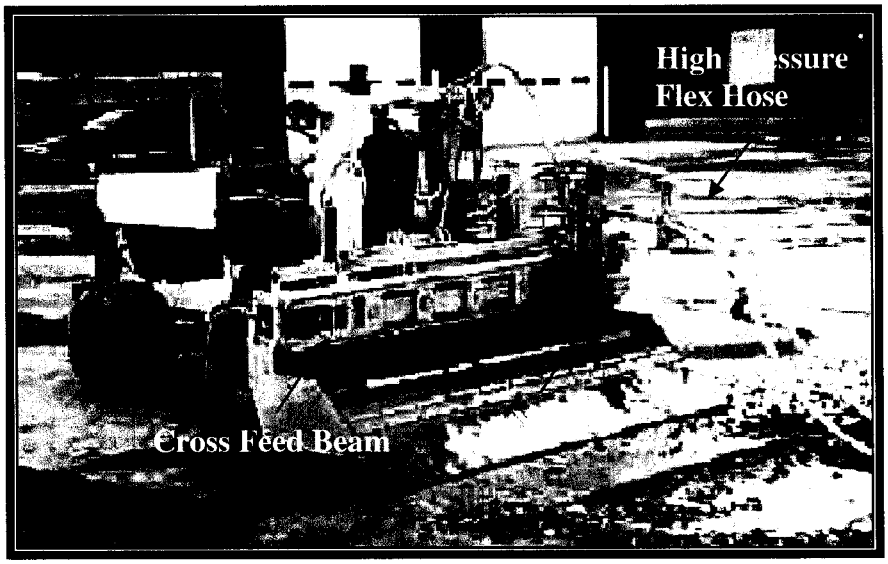


Figure 3.4 Hydrodemolition Demolishing Unit

High pressure water is delivered from the power unit to the nozzle by a high pressure flexible hosing. The water delivery nozzle is attached to a trolley which transverse back and forth along a cross-feed beam at a programmed rate. The nozzle is rotated or oscillated at a constant programmed frequency. At the end of the programmed cycle, the entire demolishing unit advances a set distance (Vorster et al., 1992).

3.2.3.1 Uses of Hydrodemolition

For decks containing large quantities of contaminated or deteriorated concrete, hydrodemolition is ideally suited to remove concrete not only from above and around the reinforcing steel but below as well. Hydrodemolition requires that a portion of the bridge be completely close to traffic for an extended period of time. Although the production rates is high and enables concrete to be removed quickly, the surface produced is not suitable to be re-opened prior to patching or overlaying which sometimes takes several days to be completed (Ingvarsson, 1988).

3.2.3.2 Material/Area/Depth to be Removed

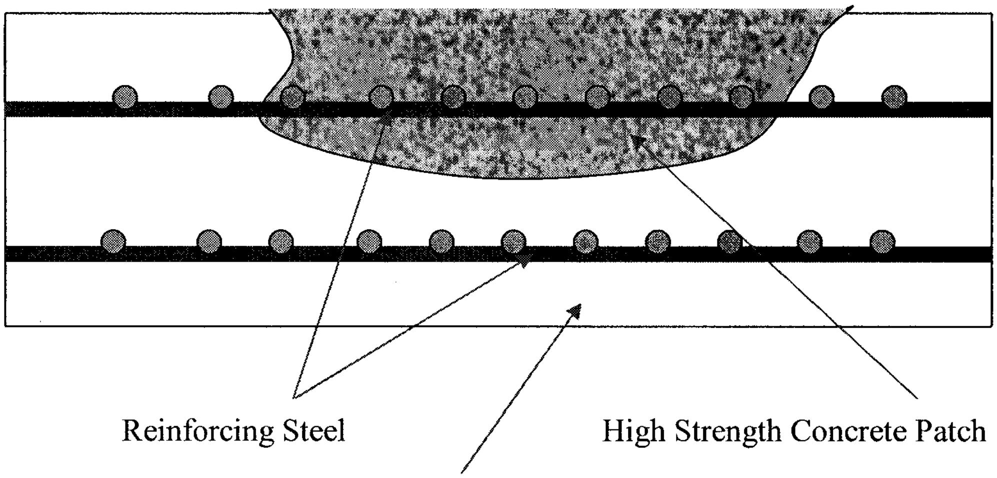
For hydrodemolition to be effective, the strength, uniformity of strength and aggregate size of the concrete all need to be determined. These material properties also determine the resultant surface profile obtained from hydrodemolition (Warner, 1998). The area which may be removed is physically limited by the machine's geometry and operation methods (i.e. the cross-beam width and limit switch settings will determine the width of the cut, and number of passes required).

3.2.3.3 Removal and Clean up of Debris

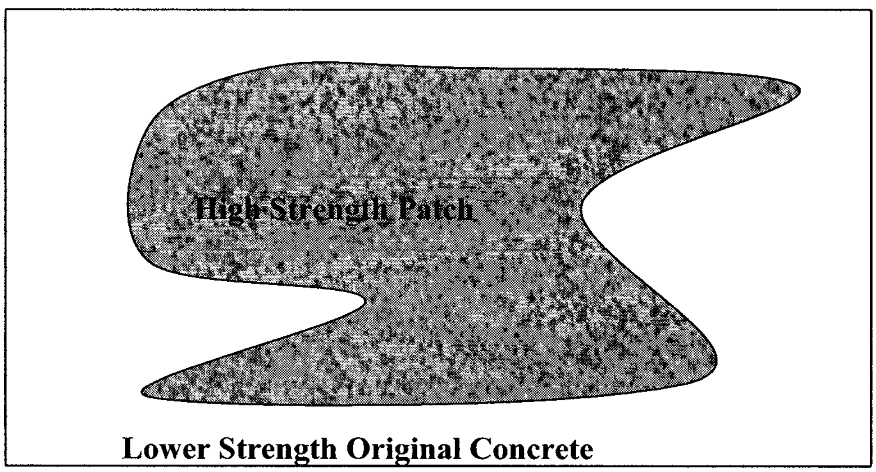
After the process of hydrodemolition, the demolished concrete together with other waste products form a combination of rubble, slurry and run off water. Vacuuming or manually shoveling the coarse particles and flushing the slurry and fine particles away, with fresh water are two methods of cleaning up. It is important to note that the slurry is not allowed to dry and adhere to the deck because it will result in poor bonding being produced from the new and old overlays. If the slurry does adhere, then the deck must be water blasted or sandblasted to provide a clean, bondable surface (Warner et al., 1998).

3.2.3.4 Quality Control

The main quality concern when using hydrodemolition is ensuring that the machine is correctly calibrated to remove concrete to the desired depth. When performing hydrodemolition on a patch containing high strength concrete (HSC) and normal strength concrete (NSC), precise depth of control will not be possible because of the difference in compressive strengths between the two types of concrete. If the system is calibrated to remove HSC, when it encounters NSC (around the perimeter of the high strength patch for example) it will remove excessive amounts of it. This is illustrated in Figure 3.5.



Sectional View



Plan View

Figure 3.5 Concrete Patch containing both Normal and HSC

Hydrodemolition is capable of removing concrete from around and below the reinforcing steel bars without causing damage to the bars, or damaging the concrete to steel bond. However, there is the problem of rebar shadowing because the reinforcing steel bars shield the concrete directly beneath them from impingement by the water jet.

In a study carried out by Momber (2000), it was concluded that the primary cause of concrete failure is due to the propagation and intersection of existing microcracks. Momber (2000) also concluded that crack growth is controlled by the interaction between cracks and the aggregate grains. Whenever hydrodemolition's high speed water flow is not controlled, microcracks will develop and grow in the interfaces between the matrix (containing the microcracks) and aggregate grains. This occurs due to the stresses from the water pressure exceeding the critical material (concrete) values (Momber and Kovacevic, 1994). When several single cracks join, it leads with time to the generation of fine grained erosion debris (Momber and Louis, 1994). According to Momber (2000), if hydrodemolition's water pressure is not controlled, the generation, propagation and intersection of cracks will occur.

Another concern when using hydrodemolition is the amount of water required. This water must be properly handled and disposed off because the slurry produced is usually very alkaline and must be neutralized before it can be released into the storm sewers (Warner, 1998). However, there has been progress and effort made by Morin and Tuttle (1997) to recycle the water used in hydrodemolition. The water recycled must be very clean and free from suspended solids (concrete debris).

3.3 Other Removal Techniques

The previous section discussed commonly used techniques used by many major cities/agencies across North America. However, these techniques primarily depend on the project size and location. Other removal methods that will be discussed in this section are as follows: machined-mounted demolition attachments, sawing and cutting, and splitting.

3.3.1 Machine-mounted Demolition Attachment

For this type of removal technique, there are usually two types of machine-mounted demolition attachment; hydraulic hammer and crushers. Hydraulic hammers are vehicle mounted on excavators, backhoe-loader and skid steer loader as shown in Figure 3.6. They are rated in terms of impact energy measured in Joules. The vehicle mounted breaker tool design is similar to that of the hand-held breaker except that they are mechanically operated and considerably larger.

Figure 3.6 Machine Mounted Hydraulic Hammers

Smaller hammers are adequate for demolishing bridge deck pavements, slabs, and unsupported concrete, whilst the larger hammers are for thicker, highly reinforced concrete members such as piers and abutments (Abudayyeh et al., 1998).

The advantages of hydraulic hammers are as follows:

- High production rate and greater mobility,
- Operable in inclement weather because operators are shielded inside the excavator or back hoe cab.
- Reduced physical stress on operating personnel in comparison with hand held hammers (Abudayyeh et al., 1998).
- The debris produced is recycled by placing it at desired locations in open water to serve as a fish attractive reef (Ryder, 1979).

The disadvantages of hydraulic hammers are:

- Generation of large amounts of noise.

- Generation of large amounts of dust and vibration.
- Restriction in areas of limited space.

Crushers demolish concrete or cut reinforcement by applying opposing forces on either side of a concrete member as shown in Figure 3.7.

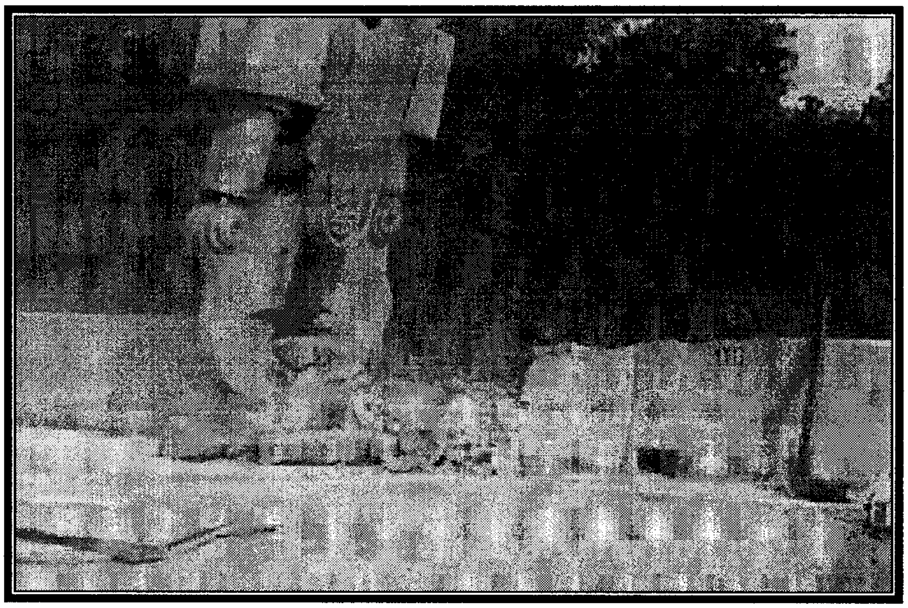


Figure 3.7 Concrete Removal Using Concrete Crusher (Sprinkel, 2000)

A typical crusher maximum force can exceed 3113.6 KN (Koski, 1993). A concrete crusher size ranges from small hand held units weighing 39 kg to larger vehicle mounted units weighing 3364 kg (Transportation Research Board, 1991). Crushers have jaw-like attachments which are used for specific applications. They can have large cracking jaws which are used to remove large sections of concrete. They can have shear jaws used to cut through concrete and reinforcement. Lastly, they can also have pulverizing jaws used to separate concrete from reinforcement.

In a recent study carried out by Sprinkel (2000), Sprinkel compared the use of a 12 kg HHPB, a 1014 Joules backhoe ram (hydraulic hammer) and a universal processor 50 crusher. When the back hoe ram hammer and the crusher was compared to a 12 kg HHPB in removing the deck and parapet concrete, they resulted in a reduction time of 94% and a reduction is cost of 59% for the hoe ram and 58% for the crusher. But yet, many Department of Transportation around North America still do not employ the use of these alternatives because they are still concerned about the damages that may to the residual sound concrete left in place (Sprinkel, 2000).

For Sprinkel's study, each piece of equipment was allowed to remove 3.1 m sections of the parapet and exterior edge of the deck. From this, the contractor involved estimated the time required and total cost for the removal. In addition, strategic cores were taken along the remaining deck to monitor the damage to the residual concrete. These cores were subjected to the following tests; compressive strength, tensile bond pullout strength, and permeability to chloride ions. After the analysis, Sprinkel concluded that all 3 equipment did not damage the residual concrete in any way and there is a tremendous reduction in project time and costs when using the backhoe ram hammer and crusher (Sprinkel, 2000).

The advantages crushers are as follows:

- No dust and vibrations are produced.
- Low noise level.
- This equipment had great mobility which results in high production rates.
- Operable in Inclement Weather.
- Utilization for loading debris into trucks for removal.
- Rapid and safe cutting of reinforcement.
- Effective ability to separate concrete from steel allowing for recycling of both materials.

3.3.2 Concrete Sawing and Cutting

For this type of technique, two methods will be discussed; blade saws and diamond wire cutting. Blade saws are generally used to cut bridge decks into large pieces which can be lifted out with the use of a crane. Wet cutting diamond blades are the most common type used to cut concrete. Water is required during cutting to cool the blade.

Diamond wire cutting is a technology that was developed in Italy in 1974 approximately 28 years ago (Abudayyeh et al., 1998). The most common type consists of industrial diamonds electroplated to a steel bead that is strung into a wire rope. In order for the cutting to begin, a 25 – 50 mm hole is drilled through the concrete and the wire is passed through. The wire is then reconnected together using a steel coupling and placed on the drive wheel. Water is also required to cool the wire and wash away the slurry.

The advantages of these methods as outlined by Hulick and Beckman (1989) are as follows:

- No dust and no vibration are generated.
- Sawing leaves clean straight edge.
- Diamond wire is effective regardless of the thickness of the cut or amount of reinforcement.
- Reduced volume of debris is produced.

Hulick and Beckman (1989) cited the following disadvantages of these methods:

- Diamond blade saws are limited in the depth of cut they can make (diamond wire systems are not).
- Problems can be encountered using diamond blade saws cutting parallel, and coming into contact with the reinforcement.
- Diamond wire systems are expensive

3.3.3 Concrete Splitting

For this technique, two methods are available: mechanical and chemical splitters. Mechanical splitting is done using hand held tools that apply hydraulic pressure to concrete causing it to fragment (Transportation Research Board, 1991). This process involves drilling a 25 – 50 mm hole, then the splitting tools shim plates are inserted into the hole, and hydraulic pressure is applied. The force produced range from 1100 to 3650 KN.

This method also has several advantages, such as:

- No vibrations are generated.
- Little dust produced and this technique is relatively inexpensive.
- Residual sound concrete remains undamaged.

Mechanical splitting also has some disadvantages, including:

- This technique is very time consuming.
- It requires pneumatic breaker to expose reinforcement for cutting.

Chemical splitting use expansive agents (consisting of Calcium Oxide, CaO) that increase in volume when properly mixed (Hinze and Brown, 1994). These agents are placed in holes in a pre determined pattern, once the agents expand the concrete splits.

Advantages of this method include:

- Non explosive, so it can be carried out at any time.
- There is no vibration or noise produced.
- It is a very safe method to remove concrete and it is also very safe on the operator as well.

Disadvantages of this method are:

- This method is very expensive to carry out.
- This method is also very time consuming.

3.4 Bar Cleaning

So far, this chapter has focused on concrete removal techniques. After the concrete (cover concrete in this case) is removed, if the residual concrete is in good condition, then an overlay can be done soon after. However, if the concrete is required to be removed at or below the reinforcing bars, after concrete removal these bars will have to be carefully examined for acceptable levels of contamination and corrosion, before an overlay can be done.

After inspection, if the bars are in good condition or have been repaired, they must be cleaned thoroughly to remove any rust, chloride ions or any other unwanted materials/agents which was able to build up over time (Emmons, 1993). This will prevent or slow down the corrosion process from continuing on the old concrete, and being transferred to the new concrete.

There are three commonly used techniques for bar cleaning and they are described as follows:

- Sand Blasting
- Wire Brushing
- Hydrodemolition (water blasting)

Sandblasting uses sand particles under compressed air to clean the exposed bars to remove cement deposits and rust leaving a bare metal finish. Wire Brushing uses rotary wire bristle brushes to clean the expose rebar. Due to the size of the deck in some bridges, these brushes are usually pneumatically or hydraulically driven, and mounted on a small construction vehicle. Hydrodemolition, as mentioned earlier in this chapter, is unique, because it can remove concrete as well as clean the reinforcing bars at the same time. Providing the reinforcing bars are in good condition, new overlays can be placed within hours after this technique is used. Hydrodemolition uses sand propelled by high pressure water, thus creating an abrasive fluid able to clean the rebar adjunct or parallel to the removal operations.

4.0 Bridge Deck Rehabilitation Techniques/Methods

4.1 Introduction

Once all of the deteriorated concrete has been removed, the next step in repairing/rehabilitating the bridge deck would be to choose an appropriate rehabilitation technique. This Chapter will focus on discussing the various choices of rehabilitation techniques available, which can be used on bridge decks.

4.2 Common Defects and Deterioration

There are nine common defects found on bridge decks: effect of salt, overloading, scaling, cracking, delamination, spalling, chloride contamination, honeycombing/air pockets and frost damage. Each of these has been discussed in detailed in Chapter 1, section 1.1 through to 1.9.

4.3 Factors Affecting the Selection of Appropriate Rehabilitation Techniques

In addition to what has been mentioned in section 4.1, there are also six more major factors which influences the selection an appropriate rehabilitation technique; they are described in the following sections.

4.3.1 Cost Analysis

Generally the results from the condition survey will suggest more than one method of rehabilitation for a given structure. In order to choose the most cost effective rehabilitation strategy a cost analysis of all the proposed methods is carried out. The analysis should also include the option for replacement of part or all of the structure where there is extensive deterioration or where the structure requires strengthening to carry applied loads. The analysis also includes a Do Nothing option. This is added in the event the structure has minor deteriorations and only some minor repair/rehab is required. This may not be cost effective due to high mobilization costs, site may be difficult to access or working on the decks may cause traffic disruptions (MTO, 1996).

4.3.2 Importance of Structure

This factor is determined by traffic volume at the site, the importance of the highway/roadway, availability of alternative routes (if any) and size of the structure. All of these indirectly affect the total cost of the rehabilitation. Sometimes a structure may be classified as a heritage bridge, meaning it will be repaired or rehabilitated even though it may be more economical to replace it with a new one (MTO, 1996).

4.3.3 Type of Structure

There are two types of bridge structures, they are as follows:

Bridge with Deck Slab – here the slab is an integral part of the superstructure, such as solid or voided thick slabs, concrete box girder and T girder bridges. These are all costly to rehabilitate and more difficult and costly to replace. So these structures require additional treatment (yearly monitoring) to ensure long term durability.

Bridges without Deck Slab – some bridges such as side by side precast box beams or T-beams are constructed such that the top flange of the beam acts as the deck slab. The rehabilitation of these bridges usually includes a construction of a 150 mm concrete slab with one layer of longitudinal and transverse reinforcement followed by water proofing and paving. Before this is done the bridge is evaluated for the extra load (new concrete slab) acting on the beams.

4.3.4 Structure Service Life

A structure usually requires replacement when it does not meet current design criteria for geometry or load supporting capacity or some other deficiencies are in components of the structure to limit its service life. When this occurs, usually the most cost effective rehabilitation strategy is chosen to keep the structure in service until it is replaced (MTO, 1996).

4.3.5 Contractor Expertise

Depending on where the structure is located, consideration must also be given to the expertise of local contractors and available construction equipment. Rehabilitation methods requiring a specialized contractor from outside the area for small contracts may not be cost effective. In such cases, alternative rehabilitation methods which can be done by local contractors are considered (MTO, 1996).

4.3.6 Environmental Concerns

This factor is very important as it affects the rehabilitation method and timing. The rehabilitation method chosen should minimize inconveniences to the public, and be scheduled so as to minimize traffic congestions during peak traffic periods. When working in environmentally sensitive areas, the rehabilitation method chosen should have the least impact on the environment (MTO, 1996).

4.4 Types of Rehabilitation Methods

Generally when repairing/rehabilitating a bridge deck, three different options can be considered. These range from patching selected parts of the deck, complete replacement of the deck or using protection systems on the deck. In the following sections different options are discussed.

4.4.1 Patching

Patching or concrete repair is intended to restore the structural integrity and function of the old concrete. When selecting repair materials the following factors are usually considered:

- Structural compatibility of the material and expected performance with the residual concrete.
- Availability, cost and anticipated life.
- Ease of construction and availability of qualified contractors in the area.

It is very important when choosing repair materials, that they have close physical characteristics to the original (residual) concrete. This characteristic applies to bond requirements, the elastic properties, and the expansion characteristics of the new and old materials. Tensile and compressive strength should be as high as the original concrete. The materials used to repair the concrete should have low shrinkage, low permeability and low water to cement ratio to inhibit moisture and chloride penetration into the repaired concrete. These repair concrete materials should also not promote any chemical reaction with the embedded reinforcement.

The repair material selected must have good bonding characteristics and should adhere to the existing concrete surface. Usually an epoxy bonding coat is applied on top of the residual concrete before the repair material is placed to increase the bond strength. Another type of bonding material that can be used is a 1:1 sand-cement grout. This grout is usually scrubbed onto the residual concrete surface to wet it uniformly, to displace air films and to incorporate any loose particles still adhering to the surface (Xanthakos, 1996). These bonding agents are used because they serve as a barrier against the migrating chloride ions that can initiate reinforcement corrosion. Additionally, with the cement grout, corrosion inhibitors can also be added to increase the protection of the reinforcing steel.

Generally, concrete repairs can be shallow or deep. Repairs are called shallow when the depth of concrete deterioration is less than 19 mm, and the reinforcing steel is not exposed as shown in Figure 4.1 (Radomski, 2002). The defective concrete is saw cut into rectangles or squares and removed by pneumatic breakers. The surface is then cleaned thoroughly and the repair material is placed (Brinckerhoff, 1993). Deep repairs are repairs requiring removal of the defective concrete deeper than the top mat of steel reinforcement as shown in Figure 4.2 (Radomski, 2002). The reinforcing bars are exposed and must be thoroughly cleaned by sandblasting or waterblasting. They are then examined and if supplementary bars are required they are added if there is excessive amount of corrosion resulting in section loss (Brinckerhoff, 1993).

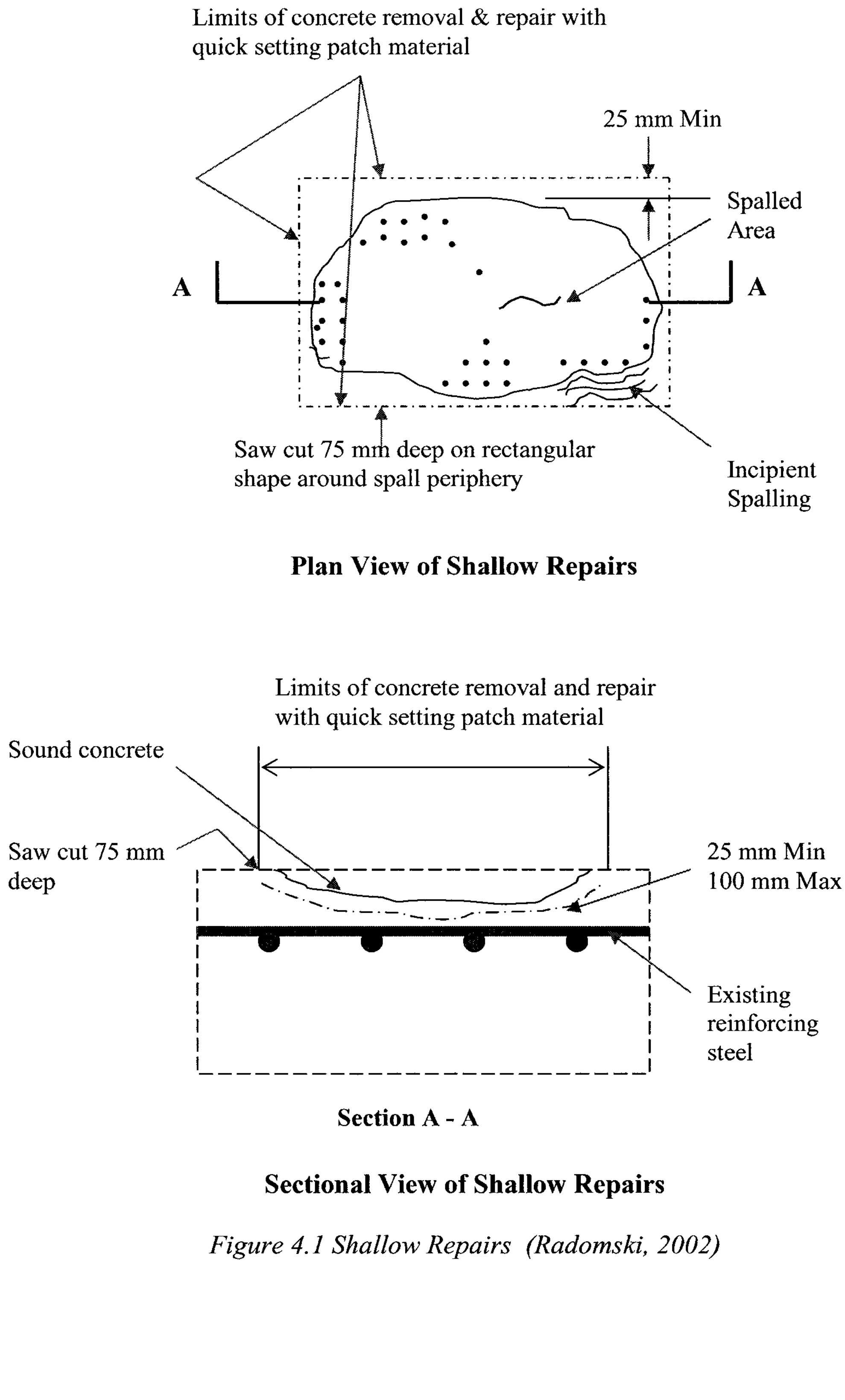
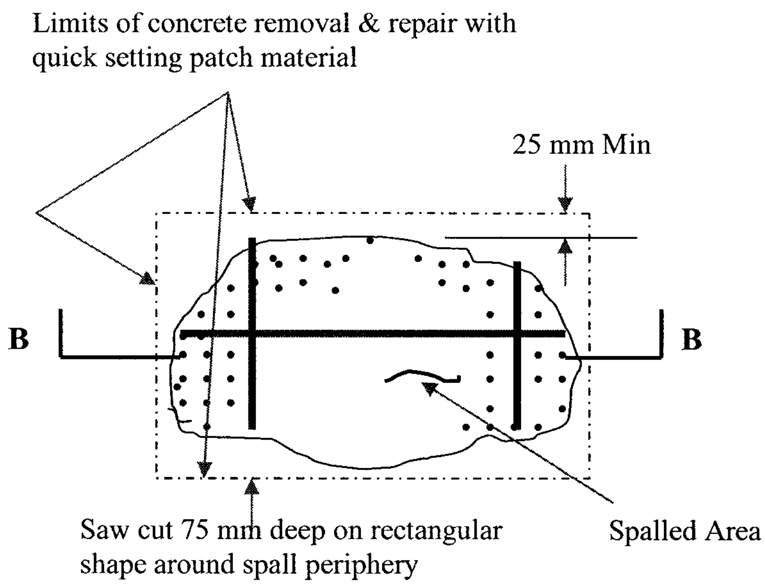
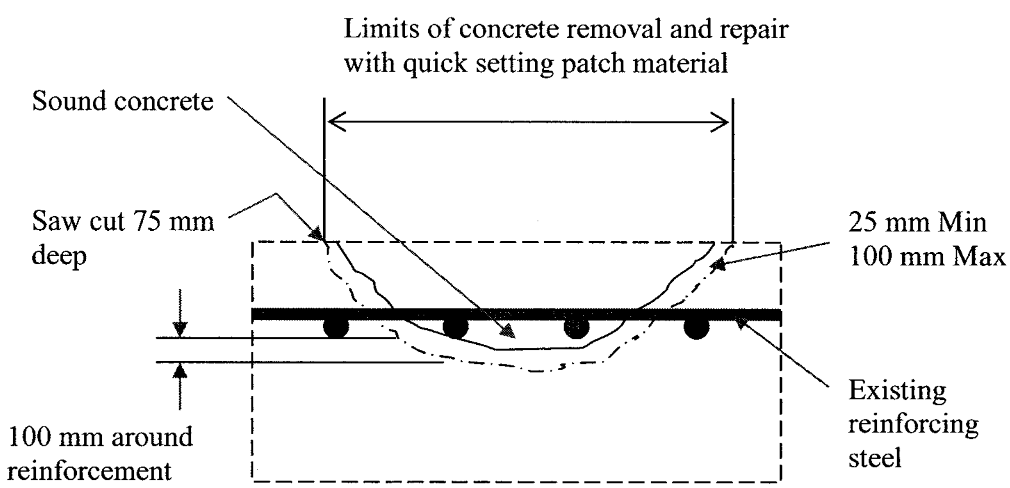


Figure 4.1 Shallow Repairs (Radomski, 2002)



Plan View of Deep Repairs



Section B - B

Sectional View of Deep Repairs

Figure 4.2 Deep Repairs (Radomski, 2002)

Patching is also a frequently used method of rapidly repairing a bridge deck; it involves removing the chloride contaminated and delaminated concrete, sandblasting the residual concrete and filling the hole with a rapid curing concrete (usually high early strength or steel fiber reinforced concrete). After this is done, cracks are usually repaired and a rapid protection system is installed (asphalt overlay on membranes or polymer overlays).

Patching, as outlined by Emmons (1992), has several advantages, they are as follows:

- Patching, crack repair and application of protection treatment can be done in stages, minimizing traffic disruption and avoid bridge closures.
- Cured materials can be opened to traffic in 2-4 hours.
- Concrete removal cost is low (little concrete is removed).

However this method has also several disadvantages as outlined by the ACI (1993), they are as follows:

- Corrosion not stopped since all critically chloride contaminated concrete not removed.
- Some poor quality concrete remains.
- Insufficient time to properly prepare surface.
- The rapid setting materials are not properly batched or consolidated.
- Patch usually cracks because of shrinkage.
- Repairs are open to traffic before significant strengths develop.

4.4.1.1 Materials Commonly used for Patching

The following five materials are commonly used for concrete repair.

4.4.1.1.1 Cement-Based Mortar

This is the most common of all the repair materials because it is widely available and has a low cost. It can be used for, both small repairs and large repairs. Usually, various types of Portland cement can be chosen depending on the intended function and condition of exposure. Admixtures can also be added to improve workability, reduce permeability, increase strength and/or accelerate the setting time of the mortar used (Radomski, 2002).

4.4.1.1.2 Quick-Setting Nonshrink Mortar

Quick-setting nonshrink mortar also called high early strength mortars, are usually combined with admixtures which increase their strength and improve their bond and workability whilst reducing curing time. Quick-setting nonshrink mortar is also known to control shrinkage cracks and provide better bonding between old and new concrete.

4.4.1.1.3 Epoxy Mortar

This type of mortar has been around since the 1960s (Fowler, 1990). Generally, a typical epoxy mortar consists of two components: the epoxy resin and a curing agent. The advantages of epoxy mortars as outlined by Lackpour (1993) are as follows:

- Low modulus of elasticity that is not sensitive to temperature variations.
- High compressive, tensile, flexural and shear strengths.
- Good bond characteristics under thermal compatibility tests.
- Non sensitive to moisture and wet environment with good resistance to chemical action.
- High resistance to impact and abrasion.

4.4.1.1.4 Resin-Based Polymer Concretes

This type of repair materials has been around since the 1970s (Fowler, 1990). Today, one common type of polymer concrete is the Sika Pronto Monoma, which is capable of developing a compressive strength of 38 MPa in one hour at 73°C, reaching 90 MPa in 24 hours. Another common polymer concrete is the Metacrylic Mortar system which is capable of having a compressive strength of 14 MPa in 8-16 hours and 42 MPa in 24 hours. The Metacrylic Mortar has a high modulus of elasticity which remains constant under temperature variations, thus ensuring compatibility with the residual concrete (Brinckerhoff, 1993). The unit cost for resin based polymer concrete is approximately five times the unit cost for normal concrete (Radomski, 2002).

4.4.1.1.5 Cement-Based Polymer Concretes

Commonly called latex-modified concrete, this repair material combines a polymer mixture with a cement-based mortar, which in the presence of water enhances its physical properties. The resulting mixture consists of very small spherical plastic particles suspended and dispersed throughout the cement paste (Lackpour, 1993). This resulting mix has lower permeability and shrinkage and improved chemical resistance, flexural strength and resistance to abrasion. The unit cost for cement based polymer concrete is approximately one and half times the cost for normal concrete (Radomski, 2002).

4.4.2 Complete Deck Overlays

To prevent deterioration from recurring after an overlay is placed, concrete deck slabs are being designed with features which protect them against adverse conditions. Some of these design criteria are as follows:

- A top reinforcement cover of 50 mm (AASHTO, 1990) or 64 mm required by most Northern states in the United States (FHWA, 1991) and 70 mm for the provinces of Canada (Ontario Highway Bridge Design Code (OHBDC), 1992).
- Epoxy coating of the top and sometimes bottom reinforcing bars. In Ontario, the Ministry of Transportation has opted to use stainless as its top layer of reinforcing as it is much more corrosion resistant than epoxy coated bars (Guidelines for Inspection and Acceptance of Stainless Steel Reinforcement on the Construction Site, 2001).
- Construction on heavily traveled roads can be done using the following: a concrete deck plus 38 mm top cover overlayed with latex-modified concrete, silica fume (high early strength concrete) or low-slump dense concrete for the northern states (AASHTO, 1990) and a waterproofing and asphalt paving for most Canadian provinces (MTO, 1996).
- Improved concrete quality, through a lower water to cement (w/c) ratio, higher strength air entrainment and richer mix.

Another important protective measure that can be used to prevent deterioration is to increase the design live load to reflect on today's heavier trucks. Many states/provinces now specify an increase of up to 25% for the live load acting on bridges and primary roads (Babsi and Hawkins, 1988).

When the concrete is deteriorated below the reinforcing steel on the top half of the deck slab, deck replacement becomes more of a practical solution as opposed to repairing/patching. A deck overlay has the following advantages:

- Protects against the impact of heavy trucks and the further intrusions of chlorides and other contaminants.
- Prevents carbonation (the effect of carbon dioxide on the deck causing a lowering of pH making the material less alkaline causing spalling to occur)
- Corrects uneven surface created by wear.
- Provides a non skid riding surface.
- Creates a uniform appearance.

Overlays must also have good strength, protect the reinforcing steel, resist abrasions, resist freezing and thawing and adhere well to the concrete substrates. In addition, it must be easy to apply and be cost effective.

4.4.2.1 Common types of Deck Overlays

Four common types of deck overlays exist in North America. They are described in the following sections.

4.4.2.1.1 Low Slump Dense Concrete (LSDC)

This was originally developed by the Iowa State Department of Transportation (Lackpour, 1993). LSDC contains aggregates with a maximum size of 13 mm, a w/c of 0.35, air entraining agent with an air content of 8 ± 1%. This mix is a rich dry concrete mix containing 475 kg/m^3 of cement. There is also a substantial amount of silica fume to ensure the hardness required to prevent excessive wear. The Slump for this mix should not exceed 25 mm. Having a concrete with a low slump causes problems in handling, placement and workability. To solve this, superplasticizers are usually added. When placing these overlays care must taken to ensure a constant thickness, because varying thickness sometimes causes shrinkage cracks along the boundary. After placing, curing involves placing a layer of saturated burlap over the concrete, followed by polyethylene sheets.

4.4.2.1.2 Latex Modified Concrete (LMC)

Here normal concrete is mixed with styrene-butadiene latex, this results in a highly efficient and durable concrete which can be used for both patching and overlaying. Sometimes shrinkage cracks appears when this concrete is used. It is usually approximately 6-13mm in length because the latex particles stops it from getting any bigger, the latex acts as stretched rubber, forming an impermeable surface which retards the intrusion of contaminants. The LMC is usually mixed at the site so the mix components can be accurately measured and calibrated. LMC should not be placed at temperatures below $16^{o}C$ or above $30^{o}C$. After placing, curing involves placing a layer of saturated burlap over the LMC surface, followed by a layer of polyethylene sheets for 24 hours. After which the layers are removed and the LMC is allowed to air dry for 72 hours. This allows the styrene-butadiene latex to be air cured.

4.4.2.1.3 Bituminous Overlays

Bituminous overlays are placed on decks to provide a smooth riding surface (Sprinkel et al., 1993). Many decks across North America are built with a protective asphalt overlay on top of the reinforced concrete decks. After the concrete overlay is placed, the bridge is then covered with a bituminous fabric membrane (called waterproofing). Usually a tack coat is applied to increase the bond strength between the concrete overlay and the waterproofing membrane. The waterproofing membrane edges are usually lapped so that no water can penetrate. After the membrane is placed a bituminous concrete layer of 50 mm thick is placed and rolled.

Bituminous overlays have been used in the United States on lightly traveled and less salted bridges. In Ontario, it is used more frequently on both lightly and heavily traveled bridges as the Ministry of Transportation has made it mandatory that waterproofing membrane be placed on top of the reinforced concrete deck followed by a layer of bituminous overlay.

4.4.2.1.4 Polymer Overlays

These overlays are placed on decks to reduce the infiltration of chloride ions and water, and to increase the skid resistance (Fontana and Bartholomew, 1981). These overlays are thin and follow the contours of the deck; they cannot be used to substantially improve the ride quality or drainage (Sprinkel, 1993). However when compared to bituminous or concrete overlays their increase in dead load is less.

The most commonly used polymer binders for bridge decks are: epoxy, unsaturated polyester styrene and methacrylate. Different polymers are suited to various applications because of their viscosity, gel time, flexural strength, modulus of elasticity, coefficient of expansion etc. For example, low viscosity binders work well for crack filling, as prime coats and for highly filled premixed and slurry mixtures. High viscosity binders/resins may be better suited to multi layer overlays (ACI Manual of Concrete Practice, 1988). Aggregates used in polymer concretes should be hard, be off high quality, such as basalt, silica, quartz or granite. Synthetic aggregates such as aluminum oxide or emery can also be used.

4.4.3 Types of Protective Systems

There are five types of common protective systems currently being used; they are discussed in the following sections.

4.4.3.1 Cathodic Protection

Cathodic Protection is the application of direct current to the reinforcing steel so that the reinforcing steel becomes cathodic (i.e. it prevents the reinforcing from discharging ions so that corrosion does not occur). This is achieved by introducing a separate metal to act as an anode. There are two types of Cathodic systems; Galvanic anode system and Impressed current system. The impressed current system shown in Figure 4.3 is normally used on bridge decks (Tonias, 1994). A current from an external source is impressed on the circuit between the anodes and the reinforcing mat (Hooker and Lutz, 1986). This method has been proven to stop corrosion in salt contaminated bridge decks, regardless of the chloride content (Dharmaratne and Gronvold, 1992). Cathodic protection is ideally suited for decks with large areas of corrosion potential less than -0.35 V with relatively small areas of spalls and delaminations (MTO, 1996).

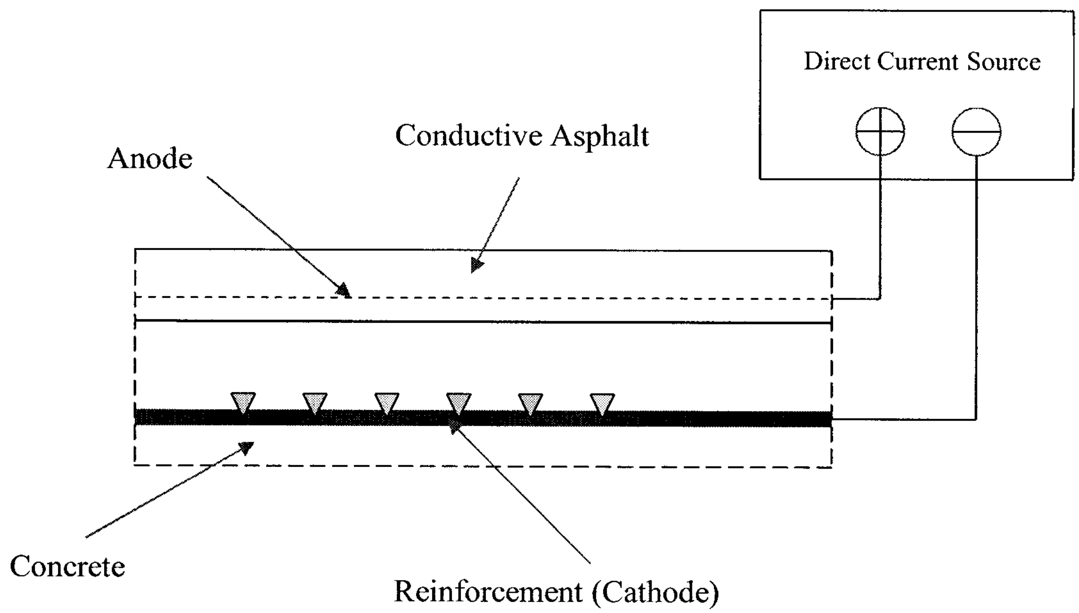


Figure 4.3 Schematic Illustration of Cathodic Protection (Pedeferri, 1995)

Disadvantages of Cathodic Protection are as follows:

- It is a very expensive method in installing and maintaining system (Pedeferri, 1995).
- Requires a uniform and constant power supply (Tonias, 1994).
- Needs to be properly monitored and maintained in order to be effective (Manning, 1990). This usually adds to the maintenance cost for the structure.
- There is an increase in dead load with the added conductive asphalt layer (Tonias, 1994).

4.4.3.2 Conductive Asphalt

This technique uses several pie shaped, high silicon iron anodes which are fixed using epoxy resins at various locations over the deck slabs and covered with 50-75 mm of conductive asphalt. Conductive asphalt is a layer of electrically conductive coke mix to distribute the current over the entire deck surface. A more durable asphalt wearing surface is placed on top. The anodes are connected to a rectifier (converts alternating current to direct current) placed near the bridge to continuously regulate the current up or down (Hooker, 1984). The anode then takes this current and spreads it through the conductive asphalt layer, to the deck surface and through the deck concrete. This current neutralizes the destructive current flow through the reinforcing bars which cause rust formation.

4.4.3.3 Anode Mesh Systems

During the rehabilitation process, after the deck slab has been repaired and scarified, an anode mat is rolled onto the deck and covered with concrete overlay. There are currently two types of meshes available; one is a grid of copper connectors with a flexible conductive polymer coating and the other is a titanium mesh. These mesh panels are joined by transverse conductor strips (copper or titanium). Holes are drilled through the deck and lead wires are inserted and connected to a junction box and a rectifier, a continuous direct current of 50-100 Watts is applied to neutralize the corrosion process (Brinckerhoff, 1993).

4.4.3.4 Concrete Sealers

According to Sprinkel et al. (1993), sealers can reduce infiltration of chloride ions for five to ten years. Sealers are used on bridge decks to reduce the infiltration of chloride ions and water (Sprinkel, 1992). The materials can usually be applied by spray, roller, brush or squeegee.

The main function of a sealer is to penetrate the surface pores and capillaries of the concrete leaving a thin hydrophobic film 0-10 mm thick (for sealers with low solid content, i.e. < 40%). For high solid content sealers, they can leave a 10-30 mm thick film (Sprinkel, 1992). In order to provide adequate skid resistance, sealers must be placed on heavily textured surfaces. These textured surfaces are made by thinning the fresh concrete, by shotblasting the harden surface (residual concrete) or sawcutting grooves (3.2 mm wide by 3.2 mm deep). The deck must be patched prior to placement of the sealer and the patching materials must be compatible with the sealer to prevent any infiltration of chloride ions and water.

4.4.3.5 Corrosion Inhibitors

Corrosion inhibitors are added to concrete and are meant to supplement the concrete's natural ability to protect the embedded reinforcing bars by forming a passive oxide layer on the steel. This normally occurs when the alkalinity in the concrete is maintained at a pH of 12 (Paul, 1998). The most commonly used product on the market today is calcium nitrate. The amount of calcium nitrate added to the concrete depends on the anticipated chloride content the concrete will be exposed to over a given period of time. Calcium nitrate has been used for over 20 years successfully in full slab replacement as well as in concrete overlays for bridge decks.

Today, another new product with a completely different chemistry has been introduced in the market; it is referred to as a migrating corrosion inhibitor (MCI). This product is surfaced applied to existing decks and is designed to migrate to the embedded reinforcing steel to protect it against future corrosion. It is a water based blend of surfactants and amine salts. This water based blend migrates as a vapor through the concrete to form a thin, protection film on the reinforcing steel (as much as 40 mm in 24 days) (McGovern, 1994). Extensive testing of both products has indicated they are effective at reducing corrosion rates in chloride contaminated concrete (Strategic Highway Research Program, 1994).

4.4.4 Innovative New Materials/Techniques

There are currently three new technologies being developed and being used all over North America, they are as follows:

4.4.4.1 High Performance Concrete (HPC)

HPC is defined as "Concrete meeting special combinations of performance and uniformity requirements that cannot always be achieved routinely using conventional constituents and normal mixing, placing and curing practices (Delatte et al., 2001).

Over the last 10 years there have been major improvements in the field of concrete materials. HPCs have compressive strength ranging from 60 to 90 MPa, and are being produced using conventional materials and concrete production methods (Myers and Yang, 2001).

The advantages of HPC are as follows:

- Enhanced design flexibility.
- Improved durability performances resulting in reduce maintenance costs.
- Increase in service life of the structure.
- Higher strength associated with HPC translates into larger spans with reduced number of supports.
- Higher strengths can also be used to achieve wider beam spacing and consequently reduced number of supporting members.
- It also offers reduced complexity and construction time for experienced contractors allowing for cost savings.
- Allows the construction of smaller, lighter sections, thus reducing the dead load of the structure.

Despite the obvious benefits, the implementation of HPC has been very slow. This is primarily due to several factors including the uncertainty related to current design codes and a lack off familiarity of designers and contractors with practices and requirements for proper design and construction of HPC structures.

The typical design life of today's conventional concrete bridges is estimated at 40 years, whilst for HPC bridges it is estimated at 75 years (Myers and Carrasquillo, 1999). This is where the real long term cost savings and advantage lies. Unfortunately, many designers, specifiers or owners are only interested only in front end costs and do not consider the long term advantage using a more expensive but durable material. The owner is often unwilling to pay the additional project costs associated with the performance related requirements to ensure long term durability performances.

4.4.4.2 Steel Fiber Reinforced Concrete

This type of high performance concrete has been increasing in use as an overlay primarily due to the better crack arrest properties of this material. The main problem with high performance concrete is shrinkage and cracking. When fiber reinforced concrete is used, if there are cracks, they are usually very small in size, because of the ability of the fibers to hold the fresh concrete in place. Studies carried out by Ong and Paramasivam (1997) have proven that these small crack openings don't harm the concrete but it can improve the durability of the pavement structure by reducing the ingress of deterioatating substances.

Fwa and Paramasivam (1990) perform experiments to determine the feasibility of using steel fiber concrete overlays. They concluded that using 1% steel fiber volume fraction in concrete overlays improved the resistance to surface abrasion and that load carrying capacity was equal to or higher than the original design.

Langlois et al. (1994) also performed field experiments to obtain information on the influence of characteristics of concrete overlays, the type of surface preparation to receive the overlay materials, and on the durability of the bond between overlay and the old concrete. For this study, the repair materials investigated were ordinary Portland cement, silica fume concrete (high early strength concrete), steel fiber reinforced ordinary

Portland cement and steel fiber reinforced silica fume concrete. Langlois et al. concluded that fiber concrete overlays showed a significant improvement in the durability of repairs. They also believe that durable repairs are possible if proper construction practices are followed at the project site. They also mentioned that a low water to cement ratio bonding agent provides an extremely good and durable bond.

4.4.4.3 Thin Bonded Overlays

Thin bonded concrete overlays offer an economical method for rehabilitating concrete pavements and deck surfaces. The primary use of thin bonded overlays (usually less than 100 mm thick) is for repairs and toppings of slabs or concrete pavements (Granju, 1996). This new concrete is required to bond monolithically to the old concrete (Menn, 1992). The durability of thin bonded overlays depends primarily on the durability of their bonds when they become attached to existing concrete/slabs (Betterton et al., 978). To improve the bond strength, the weak layers at the interface are removed and roughen as shown in Figure 3.4.

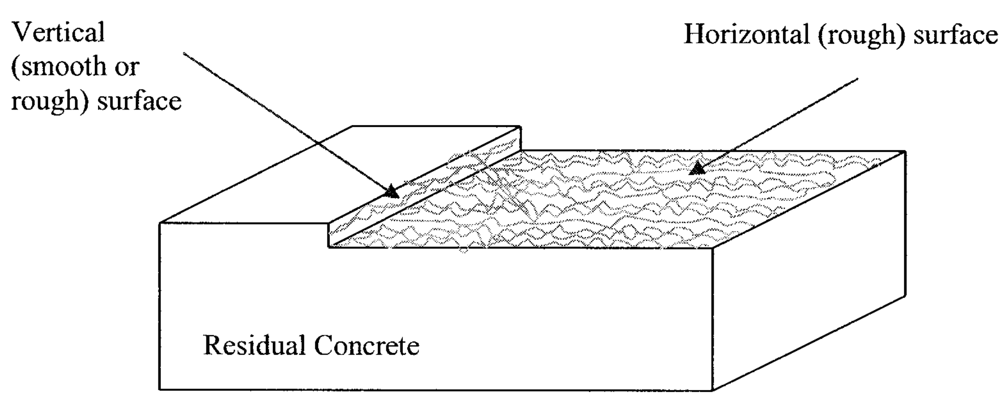


Figure 4.4 Illustration of a Typical Roughen Section (Minoru et al., 2001)

Another means of enhancing the bond strength between the thin overlays and existing concrete is with the use of steel fibers. Fibers are crack restraining and hence they improve the durability of the bond between the thin overlay and the residual concrete (Chanvillard et al., 1990). It is therefore essential to ensure that a strong bond is achieved before traffic is allowed to run over the newly constructed thin bonded overlay. With today's innovative technologies such as high early strength concrete, traffic can be allowed onto the overlay within 6-12 hours after placement (Delatte et al., 1996).

Thin Bonded overlays have been used throughout North America. There are two case studies that will be carefully examined, one is in Montreal in Canada and the other is in Alabama in the United States.

Chanvillard et al. (1989) performed an experimental repair of Highway 40 in Montreal. This was a 6 lane highway supporting 3000 vehicles a day and it was repaired in 1986 with thin bonded overlay. In his study, the use of plain thin bonded overlays was compared with steel fiber thin bonded overlays. Chanvillard et al. (1989) concluded that in plain overlays, there was a sharp rate of crack growth and rapid deterioration of the pavement in less than 2 years after the overlays had been placed. The steel fiber overlay has much less crack growth and it stabilized after the 1st year of it being laid. After 8 years of use, the steel fiber overlays were still sound, with limited cracking as shown in Figure 4.5.

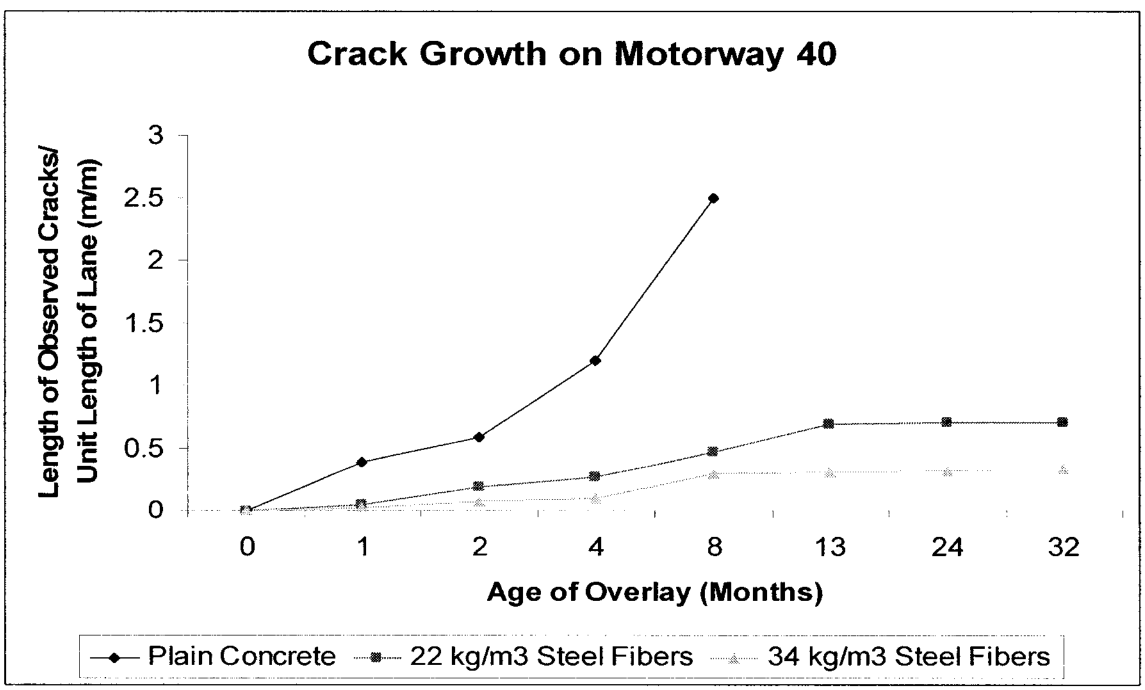


Figure 4.5 Crack Growth on Montreal's Highway 40 (Chanvillard et al., 1989)

In Alabama, the weather conditions are more favorable as they are not faced with cold weather nor do they use deicing salts on their bridges. They did some tests with thin bonded overlay as well. They used thin bonded overlays on 19 of their bridges using different overlay mixtures. Ramey and Derickson (2003) concluded the following from their investigation:

- 4 overlays 6.4 mm thick using urethane polymer concrete. They concluded that it provided a service life of only 3 years.
- 12 overlays 9.5 mm thick using polyester polymer concrete. They concluded it provided a high variable performance. Four of the overlays had a service live of only 1 year. The remaining 8 overlays are 10 years old and they are near the end of their service life.
- 2 overlays 9.5 mm thick using co-polymer concrete also called flexogrid. They concluded that it provided excellent performance. Both overlays are 8 years old and in excellent condition. Lastly, they made 1 overlay ranging in thickness from 12.7 mm to 19.1 mm using an asphaltic based NOVACHIP. They concluded that is has been in service since 2000 and is still in excellent condition.

4.5 Problems during Construction

So far only different rehabilitation techniques have been discussed. This section of this thesis will focus on problems which are common during a bridge rehabilitation project. Although many organizations/agencies have made vast progress in reducing and expediting a wide range of rehabilitation techniques, more work is still required in this area in order to minimize problems arising during the construction stage of a project.

Organizations/agencies need to pay more attention to matters such as corrosion protection, rehabilitation conditions, project accessibility and continued structure usage during repair to help minimize common construction problems experienced in many bridge rehabilitation projects. There are three common types of problems during the construction phase of a project. They are discussed in detailed in the following sections:

4.5.1 Project Constraints

In many bridge rehabilitation projects carried out in urban cities there is always some sort of project restriction. In most cases, it's effectively managing and minimizing the total time required to complete the entire project in these heavily traveled areas.

These project time restrictions are outlined as follows:

- Pouring concrete, usually done in the evenings or early mornings where the weather is cooler, this helps minimize the amount of cracking found in the concrete after hardening.
- Ensuring the deck is open to the public at all times during the day to allow traffic to flow freely. Whenever possible the organization/agency should also manage the project to ensure most if not all the lanes be opened during peak traffic hours. When these are accomplished, the organization/agency will not face as much criticism and complaints from the public and from the commercial sector along the route (Xanthakos, 1996).

Generally traffic management plans are carefully developed and planned using data gathered during the planning and monitoring phase (couple of months before construction begins), outlining the peak traffic periods as well as the average daily traffic flowing over the bridge. These traffic management plans as outlined by Xanthakos (1996) are as follows:

- Minimum inconvenience and maximum safety for the motorists, this preserves good public relations.
- Safety for the construction crew undertaking the project, the workers will only carry out major construction during off peak hours that is when traffic volumes are at a minimum (during the nights, weekends).
- Very economical, if done properly can literally save thousands of dollars per day as well reduced total construction time required for the completion of the project.

4.5.2 Impact of Rehabilitation on a Structure

This issue arises in structures that are in very poor conditions due to increase salt application rates or excessive loading that is close its design capacity. Many decks in service today were designed in the late 1950s and throughout the 1960s with no protection against salt attack, temperature extremes and moisture exposure. This primarily occurred due to the lack knowledge and experience of not knowing the potential disaster these three can cause. Today many organizations/agencies designed bridge decks following very stringent design guidelines such as:

- Increasing top concrete cover to protect the steel reinforcing.
- Using stainless steel and epoxy coated reinforcing steel as the top reinforcing mat and in some situation they are also used in the bottom mats as well because of there ability to resist corrosion better than normal reinforcing steel.
- Two layer construction – in many US states they use a regular concrete deck followed with a 38 mm top cover of latex modified concrete, silica fume, or low slump dense concrete (Brinckerhoff, 1993). In Canada, many provinces use a regular concrete deck together with a layer of waterproofing followed by a thin layer of asphalt (Structure and Rehabilitation Manual, 1996).

These guidelines work well with new bridge decks but when it is carried out on an existing bridge it can be problematic. If these guidelines are used in rehabilitating a bridge deck, there will be a significant increase in dead load to the structure. Since many existing structures are facing loads close to their design limits, adding additional loading may cause serious problems (possible collapse of the structure). If other elements such as the girders and piers have to be strengthen, this will cause a significant cost increase in rehabilitating the structure. It will also add to the project duration. In some situations after a careful detailed cost analysis is carried out outlining all of the rehabilitation methods available, it might be cheaper to replace the old structure with a new one.

4.5.3 Environmental Impacts

For this thesis only deck rehabilitation is considered. In Chapter 2 Deck Evaluation and Assessment was discussed. There are really no environmental concerns with all of the different types of surveys and test carried out because many of them are non destructive. When destructive testing (concrete coring) are done, cores are relatively small when compared to the overall deck surface so it can be discarded.

In Chapter 3, common Concrete Removal techniques such as HHPB, Milling, Hydrodemolition have similar environmental impacts. They all produce dust, noise and concrete debris. To minimize these environmental concerns the following steps are usually carried out.

- Concrete barriers are set up adjacent to incoming traffic and or around work area.
- Usually on site, there are flag men on duty to control and direct traffic flow if a problem arises.
- To control noise, motorists and/or worker safety, work is sometimes done at night.

Of the three removal techniques mentioned above, hydrodemolition posses a serious environmental threat if the slurry and run off water produced is not properly disposed off. The slurry and run off water is high in alkalinity and must be neutralized before it is removed. Neutralization is done by mixing the slurry with fresh water. After neutralization it is either vacuumed up or allowed to discharge in the storm sewers, not the catch basin. The storm sewers are interconnected and run to an effluent plant where the water is treated and waste are removed before allowed back into the water stream (ponds, rivers, lakes etc.). Catch basin or drainage holes on bridge decks are interconnected and run directly into our water stream. However it is generally considered better practice to vacuum up the slurry and take it directly to the waste management plant (Vorster et. al, 1992).

In Chapter 4, Rehabilitation techniques, there are two ways of applying all of the mentioned techniques, either through Patching or Overlaying. For patching, the environmental concerns are similar to destructive tests mentioned above for Chapter 2, because the area of the patch is relatively small when compared to the overall size of the deck, so it can be easily controlled. Overlays can also be easily controlled. Their are always concrete curbs on either side of the bridge deck preventing any materials from falling over, either on traffic if the bridge is over existing roads below or in rivers or streams if the bridge runs over a water stream. Overlays can posse an environmental concern if the overlay material is allowed to get into the drainage holes during placement. To prevent this, the drainage holes are plugged up until the overlay has been placed.

5.0 Survey Development

5.1 Introduction

The questionnaire contained a total of 13 questions. Some of the questions were combined together under one heading as they discussed subjects that were interrelated. The questionnaire was developed to achieve a better understanding of how North American bridge owners/managers were doing in terms of bridge deck repair and rehabilitation. The questionnaire was sent to over 100 individuals from 50 different organizations and agencies such as Ministries of Transportation and Departments of Transportation throughout North America, Engineering Consultants and Bridge Authorities. 30% of these various organizations and agencies responded to the questionnaire and their list is presented in Appendix B. The questionnaire was made up of six parts. Each part is discussed in the following sections.

5.2 Condition Surveys

This part of the questionnaire focused specifically on Condition Surveys and is made up of two questions. The emphasis here was on two common techniques which have been used for over the past decade (Corrosion Potential Surveys and Radar Surveys) and one relatively new technique called Linear Polarization Measurement (3LP test). In addition to these three, all the participants were asked to identify other surveys used on their bridge decks. The questionnaire also focused on how often they performed these condition surveys prior to the actual rehabilitation taking place. After the actual rehabilitation is completed, the questionnaire also focused on any measures these organizations and agencies might have in place to monitor the structure. The aim was to determine how advanced these various organizations and agencies were in terms of optimizing their budgets to repair critically deteriorated structures. In addition, the authors wanted to know if these organizations/agencies had any measures in place to know if their rehabilitation strategies and techniques were working.

1. For your organization/agency's bridge rehabilitation program, (a) Do you use the following in condition surveys (Y/N); and
(b) How effective is each of the techniques described below:

Technique	Y/N	Effectiveness (%)
Corrosion rate assessment (e.g. 3LP technique)		
Corrosion potential survey		
Deck assessment using Radar technology		

If your organization/agency uses other techniques, please specify below

Please provide additional comments if necessary

2. (a) How often does your organization/agency carry out these surveys on bridge decks to determine if concrete deterioration has began?

(b) How long after the survey is carried out does the actual rehabilitation begin?

(c) Does your organization/agency also perform updates to monitor the structure?

5.3 Removal of Deteriorated Concrete

This part of the questionnaire focused on concrete removal and is made up of four questions. There are three common removal techniques used on bridge decks. They are jackhammering, milling and hydrodemolition. In addition, to these three removal techniques, the questionnaire also focused on removal techniques such machine mounted demolition attachments, splitting and thermal demolition. Participants were also asked to list other removal techniques they have or are currently using. There has been research over the years focusing on whether or not using heavier machinery for concrete removal can be hazardous to the residual concrete. There are suggestions that these heavy machines may cause the residual concrete to crack and debonding may occur at the reinforcing steel to concrete interface. A question was developed to see whether the participants were taking any measures to prevent these problems from occurring. In addition, a question was developed focusing on three removal criteria. These criteria were listed to determine which or a combination of the three was the deciding factor for a particular organization/agency in determining what type of repair or rehabilitation to use. This was aimed to determine if these organizations/agencies relied on one or more removal criteria before a final decision is made as to which intervention to use.

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3. For the removal of deteriorated concrete,
(a) Does your organization/agency use the methods listed below (Y/N)?

(b) From your experience with these techniques, would you please give an indication on how effective (%) these methods have been for your rehabilitation program?

(c) Would you also give an indication of the percentage usage for each technique?

Methods	Y/N	Effectiveness (%)	% Usage
Machine-mounted demolition attachments			
Hydro demolition			
Splitting			
Jackhammers			
Thermal demolition			
Others, please specify below			

____

____

(d) Please provide comments on each used removal method's impact on the environment and the traffic.

____

____

4. Which removal criteria does your organization/agency use to determine the required type of intervention:

Removal zone	Criteria Used
Delamination only	
Delamination & High Corrosion Potential	
Removal of a uniform thickness over the entire deck	

Additionally, please provide comments with regards to the impact of condition and volume of traffic on types of intervention (e.g. expeditious versus long).

What is your organization/agency's philosophy in terms of selecting short & frequent interventions versus long lasting & less frequent interventions with longer service life?

5. What measures/techniques do your organization/agency use or have adapted to limit cracking propagation when removing deteriorated concrete?

6. What types of surface preparation techniques does your organization/agency use before placing fresh concrete?

5.4 Concrete Repair/Rehabilitation

This part of the questionnaire focused on repairing or rehabilitating the bridge deck. It comprises of two questions. After completing a condition survey, a good understanding is achieved on how badly the deck has deteriorated. Two choice of restoration are generally employed after determining the extent of deterioration. Firstly the structure can be repaired by patching which means that concrete deterioration is not severe and can be repaired relatively quickly and cheaply. The second choice would be to replace the entire deck with a new one capable of out performing the old one. The aim of this part of the questionnaire was to determine the techniques used by organizations and agencies for repair and for complete replacement. The questionnaire listed some common replacement materials used throughout North America, and asked for any other replacement material used by an organization/agency.

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7. Does your organization/agency use the following types of concrete replacement for either patch repair or deck overlays:

Concrete Replacement	P=Patches O=Overlays	Compressive strength (MPa or psi)	Effectiveness (%)	% Usage
Normal concrete				
High performance concrete*				
High Early Strength concrete				
Latex Modified concrete				
Asphalt concrete				
Other, ...				

* High performance concrete can be with silica fume or any other supplementary cementing material

8. Would you please indicate which of the following options {i) Rapid protection, ii) Rapid repair, and iii) Total rehabilitation of bridge decks} is used when dealing with the listed interventions:
   (a) Please indicate the average thickness of the new layer.
   (b) Please indicate the average life span obtained when using these methods.

Interventions	Options	Thickness (mm)	Expected Life Span (yrs)
Asphalt overlays with membranes			
Asphalt overlays without membranes			
High-early strength concrete overlays			
High Performance concrete overlays			
Patching with high-early strength concrete			
Patching with asphalt concrete			
Polymer overlays with waterproofing			
Polymer overlays without waterproofing			
Fiber reinforced concrete			
Other, please specify			

Please provide comments if needed

____

____

5.5 Concrete Compatibility

This part of the questionnaire focused on concrete compatibility. It is made up of one question. This issue is very important when repairing a bridge deck using patches or when the top mat of the deck is being replaced with a new overlay i.e. concrete removal does not go below the top layer of reinforcing steel. In order to obtain proper bond strength between the new and the old concrete, usually the following measures are taken to achieve monolithic action: (a) use of repair materials of similar properties as the old concrete, (b) roughen the residual surface and use of a bonding agent and (c) minimizing the vibration fresh concrete in construction stage. The aim of this part of the questionnaire was to see what each North American organization/agency were doing in this regard.

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9. When performing a rehabilitation on an existing bridge,

(a) Does your organization/agency adapt measures to ensure that there is compatibility between the new and existing concrete. If yes, could you please list some of the measures?

____

____

(b) Do you take any measures to minimize vibrations, which may occur from adjacent lanes due to traffic loads?

____

____

5.6 Proper Finishing and Curing of Concrete

This part of the questionnaire focused on two very important techniques to ensure long term durability of a reinforced concrete deck. It is comprised of two questions. Today, many organizations and agencies throughout North America are moving away from normal strength concrete (NSC). They are turning their attention to high performance concrete (HPC) as their desired choice for new overlays because of HPCs long term durability and low permeability. But in order to achieve these two characteristics, HPC must be properly placed and finished to minimize air void formation. HPC must also be properly cured to obtain low permeability and to minimize the amount of cracking and shrinkage. If these steps are taken then long term durability of this concrete is almost certain. The aim of this part of the questionnaire was to determine what various organizations/agencies were doing to ensure long term durability and low permeability. The lower the permeability of concrete, the better it resists deterioration mechanisms such as chloride ions, freezing and thawing and carbonation.

---

10. Depending on the time of year (weather conditions) do you perform any extra work to ensure proper finishing and low permeability? Please explain:

____

____

11. Depending on the type of intervention/materials used on bridge deck rehabilitation; (a) Could you please list the curing techniques your organization/agency carries out, as well as the length of curing associated with each technique.
(b) How effective (%) is each of these techniques in minimizing cracking and controlling shrinkage?

Intervention/Materials used	Curing Techniques	Length of Curing (days)	Effectiveness (%)

5.7 Bridge Decks Service Life

The final part of the questionnaire was developed mainly to determine what these various organizations/agencies use for the term 'service life'. They were asked to provide an appropriate definition for the term 'service life'. They were also provided with a table containing factors that might be used for estimating the service life of a bridge deck. Each organization/agency was asked to give an approximate figure for their yearly budget spent on their maintenance program. They were also asked to give an estimate of the number of bridges they currently look after. Lastly, this part of the questionnaire also asked each organization/agency to give a cost/ft^2 for various scenarios. The aim of this part of the questionnaire was to determine how big these organizations/agencies were and to get an idea of how much they are paying to repair, rehabilitate and keep their structures in service.

12. Would you please explain how your organization/agency defines the term **SERVICE LIFE**?

13. For estimating the service life of a structure do you consider the following?

Properties	Yes	No
Measurements of permeability of Chloride Ions		
Corrosion of reinforcing bars (> 0.35 mV, %HCP, etc.)		
Skid resistance		
Wear		
Bond strength		
Percentage of delamination		
Other, please specify		

Please provide other comments.....

Please provide an estimate budget for the bridge rehabilitation program within your Organization/Agency.....

Please provide the number of bridges your Organization/Agency is dealing with.....

Please provide an indication about the cost ($/sqf) related to each type of intervention (patching, heavy duty deck rehabilitation, total replacement, etc.)

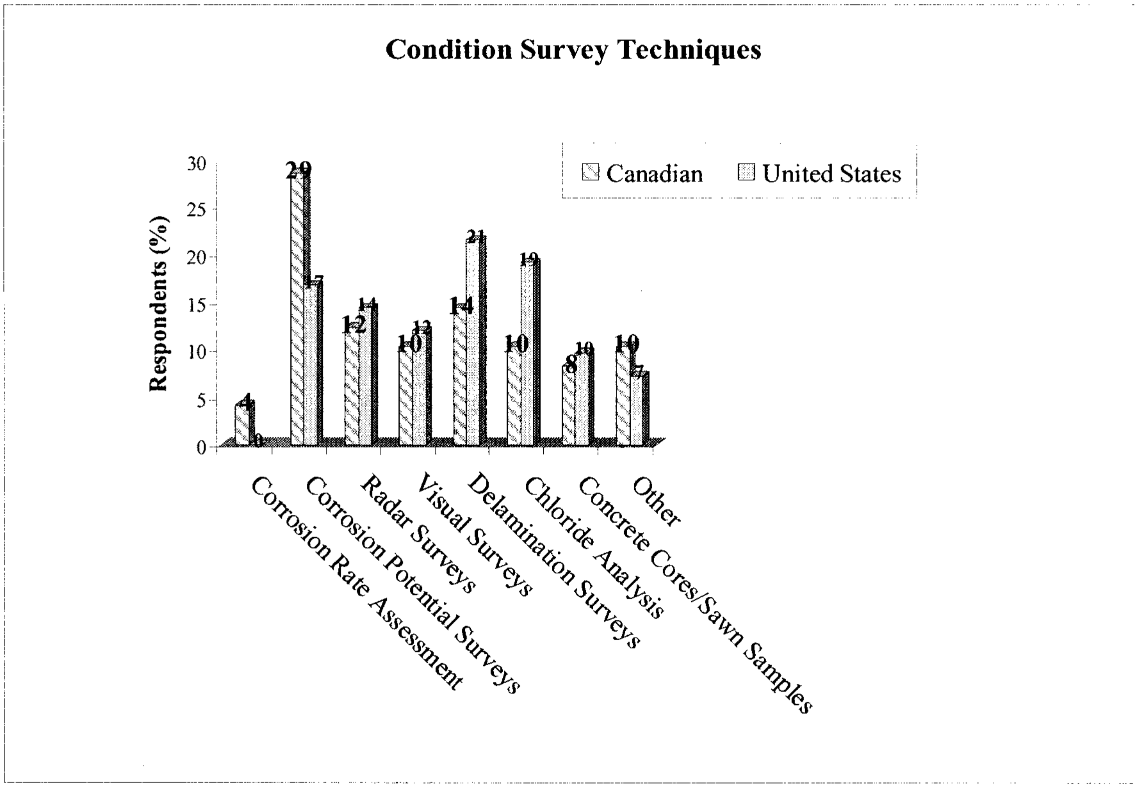
Please provide an indication about the cost versus the service life

6.0 Results and Discussion of Questionnaire

6.1 Condition Surveys (Questions 1 &2)

This part of the questionnaire focused mainly on three techniques: corrosion rate assessment (3LP), corrosion potential surveys and radar surveys. However, many organizations/agencies mentioned other surveys which they carried out when doing a complete analysis of a suspected deteriorated bridge deck. From Figure 6.1, 29% of respondents from Canada perform corrosion potential surveys compared to 17% in the US. This test is used to determine if concrete deterioration and rebar corrosion is occuring. US organizations rely more on delamination surveys (21%) and chloride analysis testing (19%) as compared to Canada's 14% and 10%, respectively. Canadian organizations also do permeability testing (2%) and asphalt removal (determines whether deteriorating is really occurring or not) (4%) in their condition surveys. Interestingly, US respondents at 0% have not used the corrosion rate assessment either experimentally or in the field. Other than these differences, both US and Canadian organizations perform very similar surveys with similar percentages.

Canadian respondents confirm that corrosion rate assessment (3LP) was 70% effective in determining rebar corrosion, whilst there was no response from the United States (Table 6.1). For the corrosion potential surveys, Canadian respondents said that it was 69% effective in determining rebar corrosion compared to 68% of the US. For the radar surveys, it was found to be 50% effective based on Canadian responses compared to 28% of US.



* Other Canadian is made up of 2% Thermography, 2% Permeability, 4% Asphalt Removal and 2% Concrete Cover Surveys.
** Other American is made up of 5% Thermography and 2% Concrete Cover Survey.

Figure 6.1 Condition Survey Techniques

Table 6.1 Response on the Effectiveness of Each Technique used in Condition Surveys

Techniques Used in Condition Surveys	% Range of Effectiveness		% Mean Effectiveness	
	Canada	USA	Canada	USA
Corrosion Rate Assessment	70	N/A	70	N/A
Corrosion Potential Surveys	50-80	50-80	69	68
Radar Surveys	50	10-50	50	28

Figures 6.2 and 6.3 show the response of organizations/agencies in carrying condition surveys for new and old structures to monitor concrete damage or deterioration in order to determine the requirement of some form of repair/rehabilitation works. As there was no big difference in responses between Canada and the US, all the data was combined. From Figure 6.2, there is clear distinction that many organizations/agencies have no fixed timing (57%) to perform a condition survey on a particular structure. After analyzing

their reasons, many of them relied on the first signs of deterioration before they start performing any kind of surveying on the structures. For the older structures in Figure 6.3, many of the organizations/agencies relied on a varying time frame as well (47%), because of lack of funding which prevents them from carrying out any routine surveys. However, 27% of the respondents carried out a condition survey at least once every two years on old structures that shows signs of deterioration.

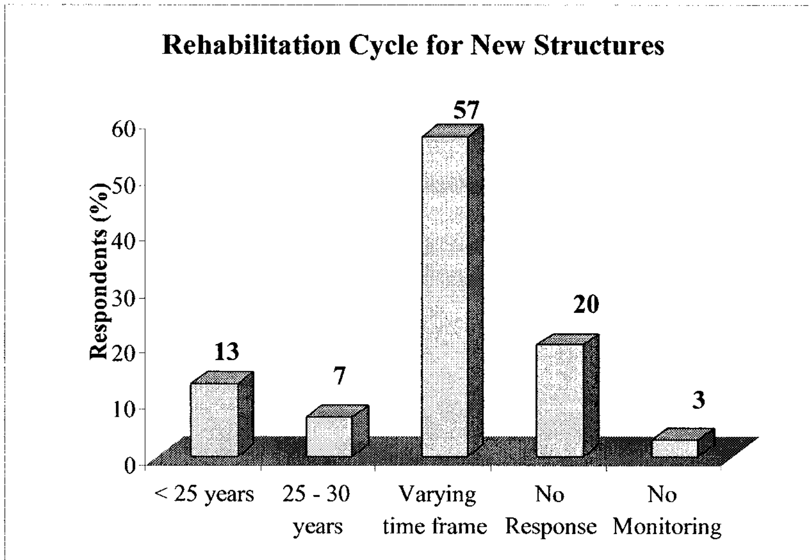


Figure 6.2 Rehabilitation Cycle (New)

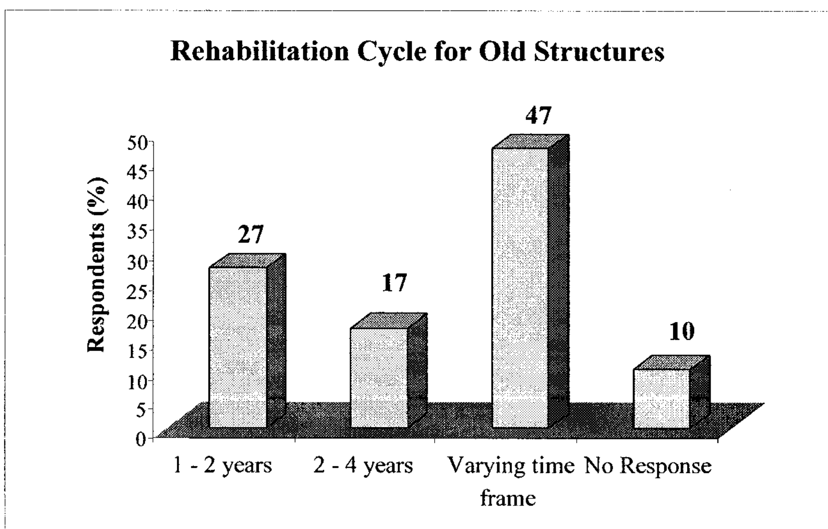


Figure 6.3 Rehabilitation Cycle (Old)

Figure 6.4 (based on combined Canada and USA data) shows that many of the organizations/agencies normally carried out actual rehabilitation within the first four years after a condition survey is carried out, 37% for both 1-2 years and 2-4 years. However, many of the respondents indicated that the ideal time frame would be between 1-2 years. If it is greater than 2 years, another condition survey may be required before the rehabilitation process can begin.

For monitoring deck slab after a rehabilitation project is complete all the data obtained from both Canada and the United States were also combined as there was no big difference in responses. From Figure 6.5, 40% of the respondents had a varying time frame for performing any type of monitoring after the rehabilitation process was completed; this was largely due to funding. Many organizations/agencies complained of lack of funding to accurately monitor the structure. However, 23% of the organizations/agencies did carry out some monitoring within the first 2 years after the rehabilitation process. 23% of the respondents indicated that they do not perform any type of monitoring.

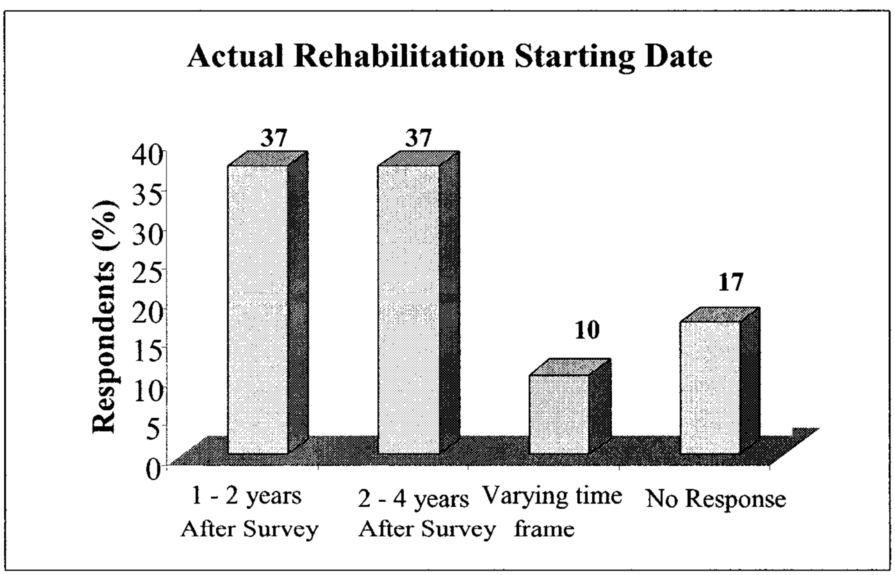


Figure 6.4 Actual Rehabilitation Starting Date

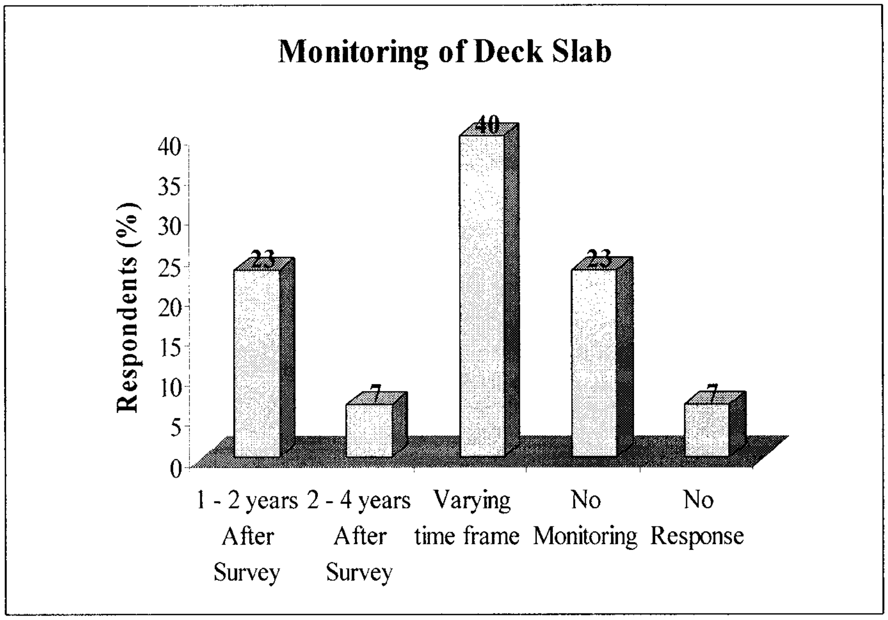
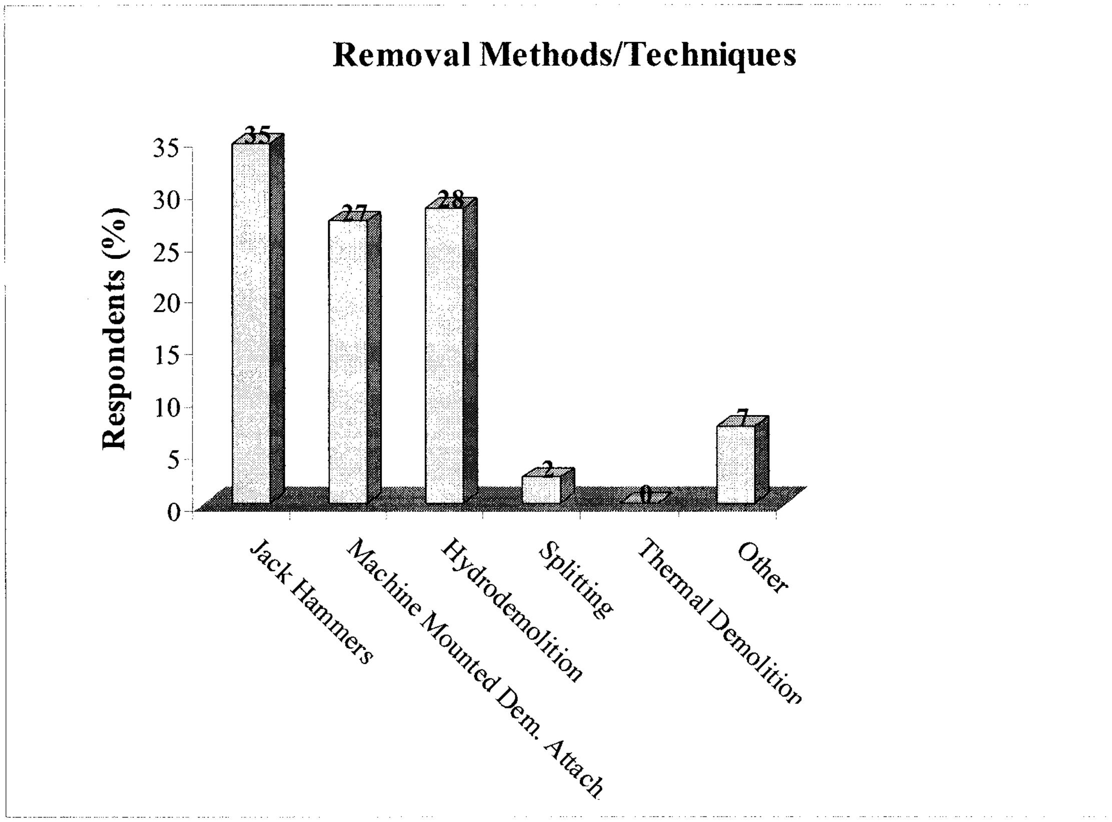


Figure 6.5 Monitoring of Deck Slab

6.2 Removal of Deteriorated Concrete (Questions 3-6)

From Figure 6.6, there is a clear indication that jackhammers with 35% is the most common concrete removal equipment, whilst hydrodemolition is second with 28%. Machine mounted demolition attachments was third at 27%. Interestingly, all respondents have never used or tested thermal demolition as a possible removal technique.

Canadian respondents said that jackhammers were 96% effective in removing deteriorated concrete compared to 87% in the US (Table 6.2). Machine mounted demolition attachments was 96% effective based on Canadian respondents compared to 83% based on American respondents. For hydrodemolition, it was 93% effective for Canadians, whilst it was 86% effective for the Americans.



* Other is made up of 2% Chipping Hammers, 1% Saw Cutting and 4% Grinding/Milling.

Figure 6.6 Concrete Removal Methods/Techniques

Table 6.2 Responses on Effectiveness of Concrete Removal Techniques

Removal Methods/Techniques	% Range of Effectiveness		% Mean Effectiveness	
	Canada	USA	Canada	USA
Jack Hammers	85-100	70-100	96	87
Machine Mounted Dem. Attach.	90-100	70-100	96	83
Hydrodemolition	75-100	50-100	93	86

Canadians use jackhammers 88% of the time when removing deteriorated concrete compared to 63% for Americans (Table 6.3). The US had a wide range of usage ranging from 5-100% compared to 65-100% in Canada. For machine mounted demolition attachments, Canadians use them 28% of the time, whilst Americans use them 50% of the time. Clearly, American respondents believe that this technique doesn't really cause any damages (namely cracking) to the residual concrete left after removal of all the deteriorated concrete. Both Canadian and US respondents had a wide range of usage ranging from as low as 4% to as high as 100%. For hydrodemolition, Canadians use it 5% of the time, whilst the American use it 25% of the time. Many of the respondents believe that hydrodemolition is very effective and quick in removing deteriorated concrete. However, this technique can cause serious consequences to the environment and motorists if it is not handled properly. The slurry (made up of concrete debris and water) produced from hydrodemolition must be handled with care to avoid contaminating the run off water. Canada had a very small range of usage for this technique (1-10%) namely because of these environmental concerns.

Table 6.3 Responses on Usages of each Concrete Removal Techniques

Removal Methods/Techniques	% Range of Usage		% Mean Usage	
	Canada	USA	Canada	USA
Jack Hammers	65-100	5-100	88	63
Machine Mounted Dem. Attach.	4-100	7-95	28	50
Hydrodemolition	1-10	1-80	5	25

All three common removal techniques have the disadvantage of producing a lot of noise, and waste and if not monitored properly can cause damages to the existing concrete. To minimize traffic interruptions, the techniques are used at night or weekend.

To determine the removal criteria in choosing the required type of intervention, Canadian and US data are combined as they show no significant differences. 35% of the respondents indicated that they relied on concrete delamination alone to determine the type of intervention (rehabilitation choice) to be carried out (Figure 6.7) while 32% favored both concrete delamination and high corrosion potential to determine the type of intervention. 29% of the respondents relied on removing a uniform thickness of concrete over the entire deck to confirm the type of intervention.

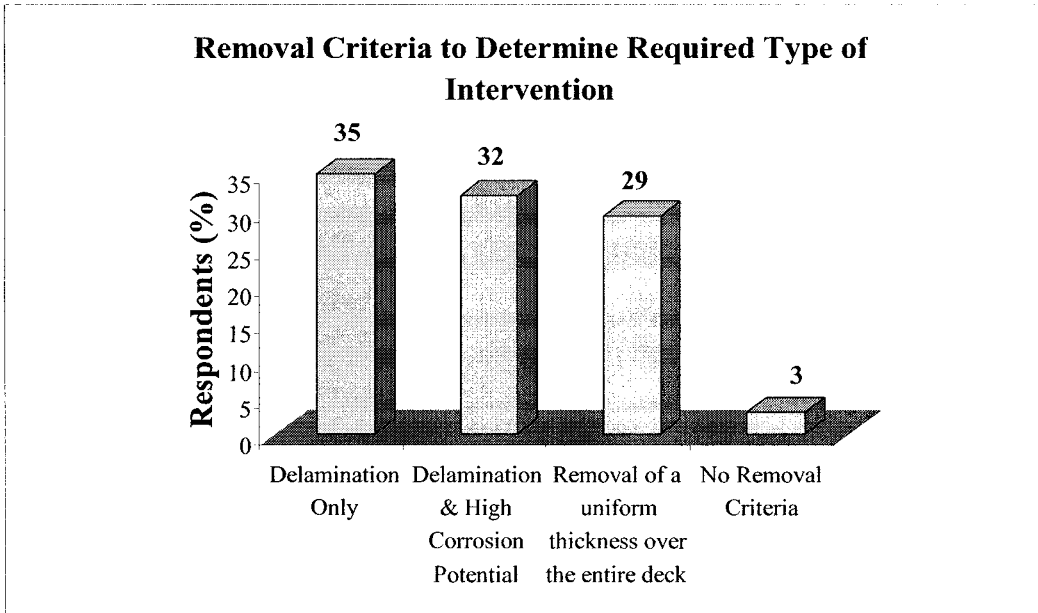


Figure 6.7 Removal Criteria

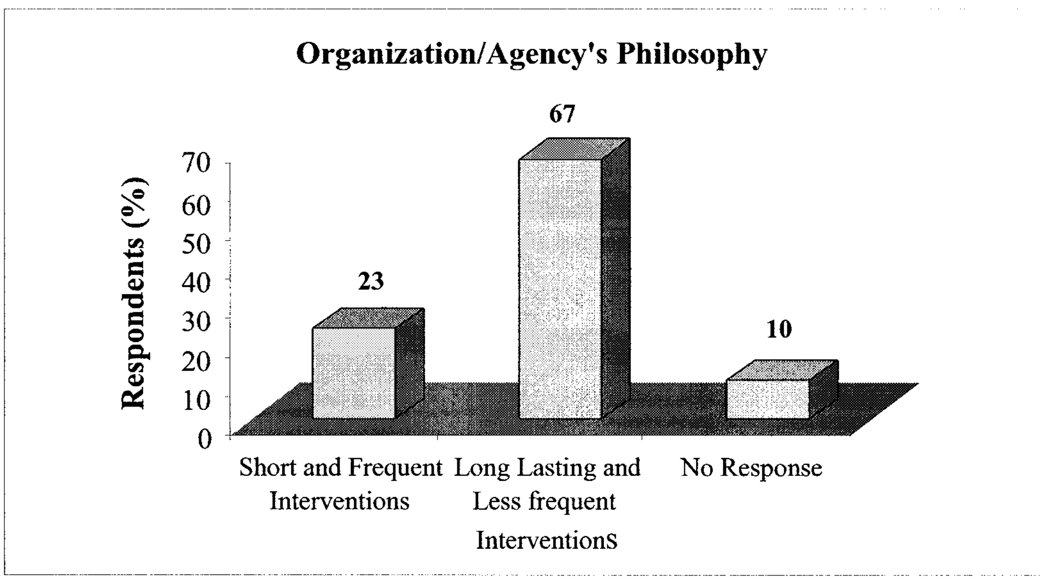
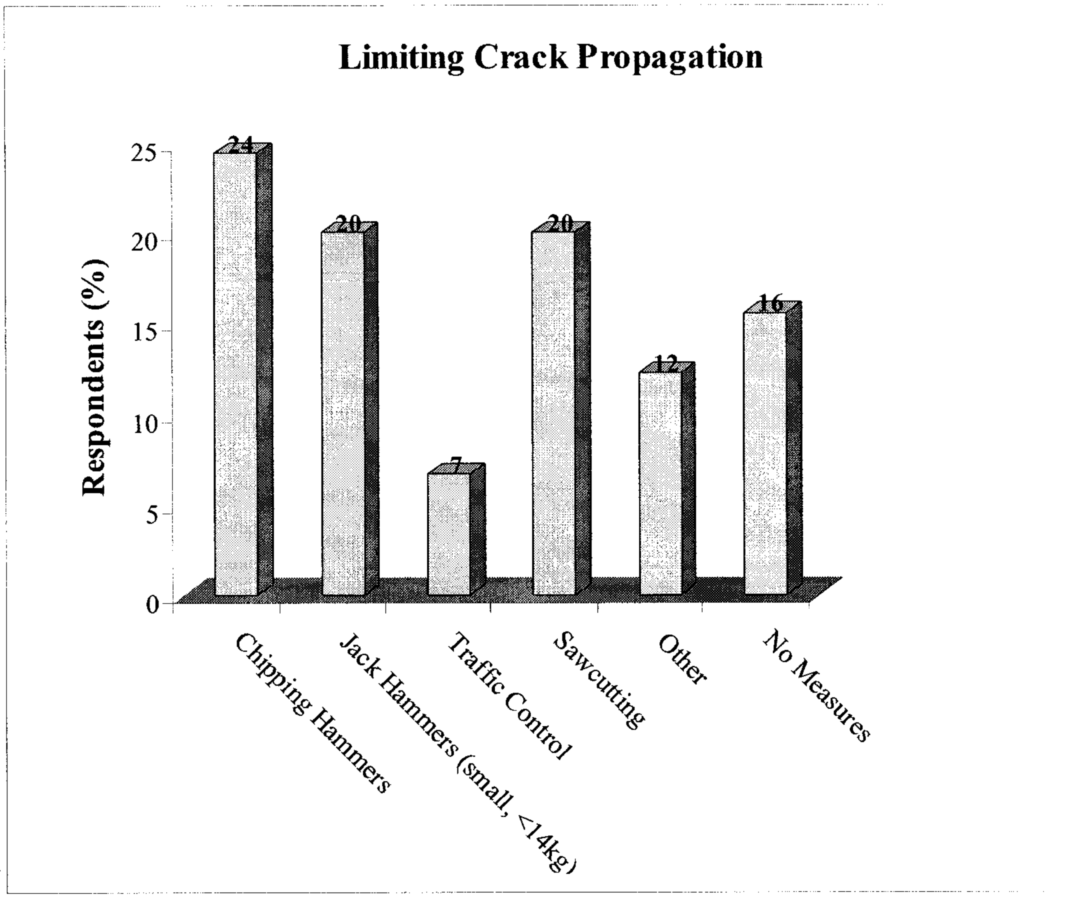


Figure 6.8 Organization/Agency's Philosophy

Figure 6.8 shows that 67% of the respondents (based on combined Canada and USA data) believed in long lasting and less frequent interventions. This means that on heavily traveled bridges, a complete deck replacement is preferred instead of an overlay. While performing these replacements or protections to the deck, they tried to minimize the impact on the traffic by working on one lane at a time, or closing the bridge at the night or weekends. 23% of the respondents preferred short and frequent interventions. Many of the respondents adopt this philosophy on decks which are in good condition requiring only minor repairs. They also use this philosophy on decks that are awaiting complete deck replacement but have to remain in service until funding is received to carry out the required work.

Figure 6.9 is prepared based on combined Canada and USA data. 24% of the respondents believed in using chipping hammers (similar to but lighter than jackhammers) compared to 20% favoring lighter jackhammers (<14 kg) to help limit crack propagation during removal of deteriorated concrete. 20% of the respondents also believed that sawcutting reduce crack propagation. Finally, 16% of the respondents indicated that they have no measure in place to help limit crack propagation. Some of the respondents explained that they have not had any problems with crack propagation and they see no reason to adapt any measures.

Since there was no big difference in surface preparation techniques between Canada and the United States, all the data are combined and plotted in Figure 6.10. 40% of the respondents used Sandblasting/Airblasting as an effective surface preparation technique before placing fresh concrete. It is important to note that the surface is first sandblasted and then airblasted. However, some of the respondents only use sandblasting to prepare their surfaces. Sandblasting is used to roughen the surface of the residual concrete as well as to clean the reinforcing bars by removing rusts. 18% of the respondents used a Slurry/Latex material to prepare the surface of the residual concrete before the fresh concrete is placed. This material is sticky and helps improve the bond strength between the new and old concrete. 16% of the respondents use prewetting combined with a sand/cement paste as their means of surface preparation. 14% of the respondents used air blasting as their means of surface preparation. However, many of the respondents used a combination of the technique mentioned in Figure 6.10. For example some of them used sandblasting/airblasting and a slurry/latex material to provide better bonds for the fresh and old concrete.



* Other is made up of 2% Reduce Traffic Speed, 2% Remove to Sound Concrete, 2% Curing of Concrete, 2% using similar Repair Materials as old deck, 2% Avoid Vibrating the rebars, 2% Monitor Operator.

Figure 6.9 Limiting Crack Propagation

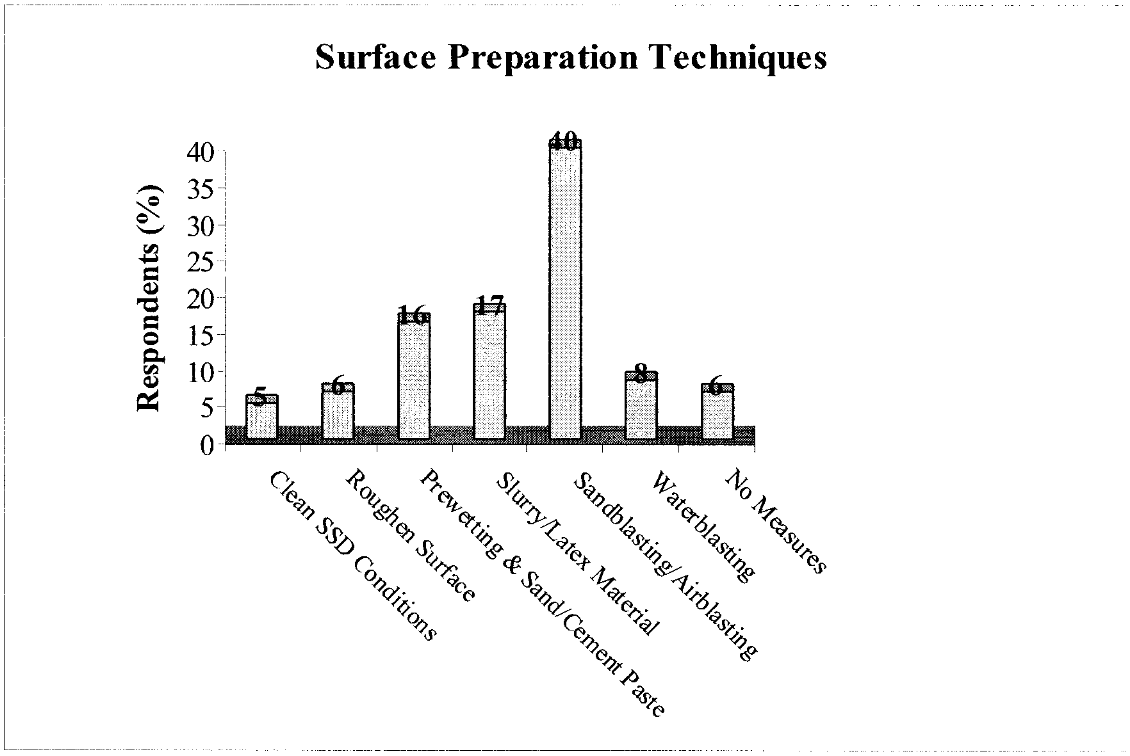
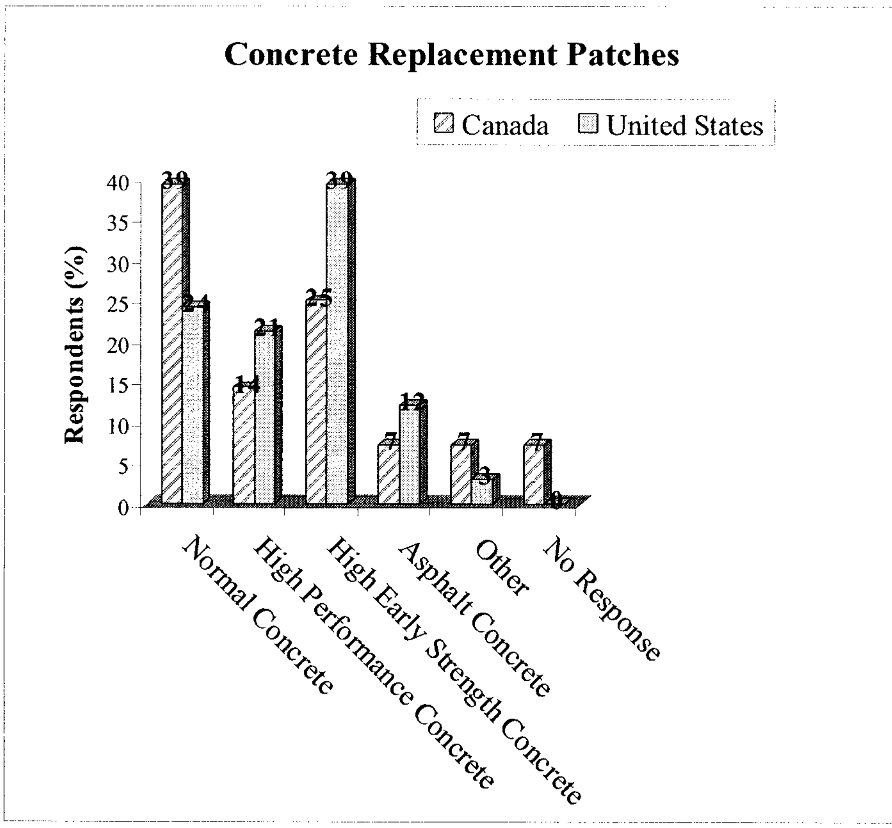


Figure 6.10 Surface Preparation Technique

6.3 Concrete Repair and Rehabilitation (Questions 7 & 8)

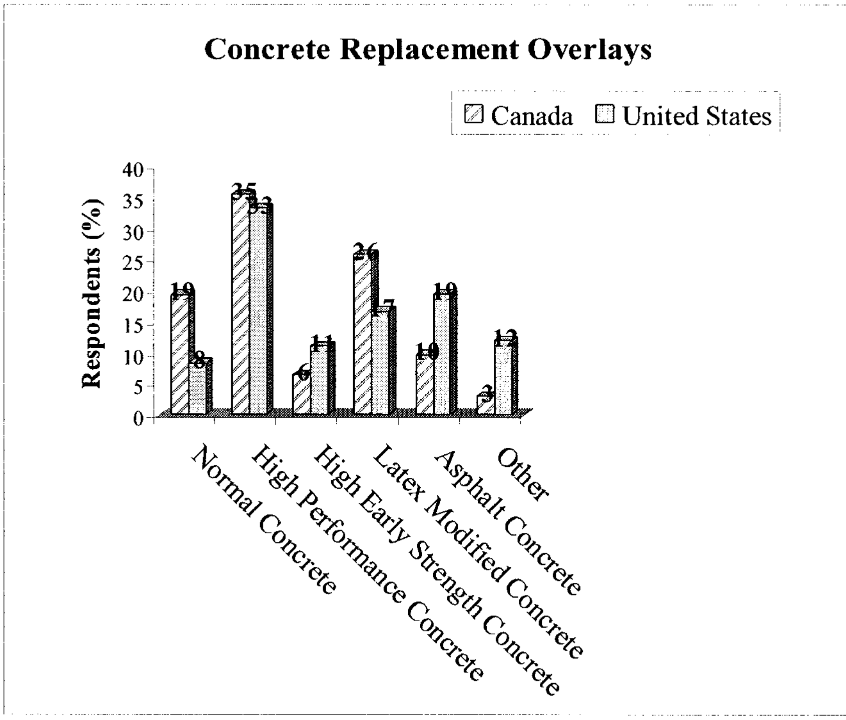
Concrete replacement was divided in two parts, one for the patches and the other for the overlays. From Figure 6.11, 39% of Canadian respondents used normal concrete for patch repair compared to 24% for the USA. The Americans rely more on high early strength concrete (39%) as a patch repair material than Canadians (25%). 14% of Canadians use high performance concrete as a patch material compared to 21% of Americans. Generally for a patch material to be effective, it must have similar properties to the existing residual concrete.



* Other Canadian is made up of 7% Proprietary Patching Products
** Other American is made up of 3% Epoxies

Figure 6.11 Concrete Replacement Patches

Many of the concrete overlays today, are made using high performance concrete. In Canada, 35% of there overlays are made up of high performance concrete compared to 33% in the USA (Figure 6.12). 26% of the Canadian respondents use latex modified concrete as overlays compared to 17% for Americans respondents. Interestingly, 19% of the Americans respondents use asphalt concrete as an overlay material compared to only 10% in Canada. It suggests that American organizations/agencies are starting to realize the benefits of using an asphalt overlay as a protective system for the concrete overlays.



* Other Canadian is made up of 3% No Response
** Other American is made up of 6% Low Slump Concrete, 3% Epoxies and 3% Polymers

Figure 6.12 - Concrete Replacement Overlays

There was no significant difference in maximum compressive strength of patch material in Canada and the USA (Table 6.4). Maximum compressive strength ranged between 30 and 40 MPa for normal concrete patches, 50 and 60 MPa for high performance concrete and 35 and 50 MPa for high early strength concrete.

Table 6.4 Responses on Maximum Compressive Strength for different types of Patches

Concrete Replacement Patching	Range of Compressive Strength (MPa)	
	Canada	USA
Normal Concrete	40	31
High Performance Concrete	50	62
High Early Strength Concrete	40	35

No significant difference in maximum compressive strength of overlay materials was also observed between Canada and USA (Table 6.5). Maximum compressive strength was 35 MPa for normal concrete, ranged between 50 and 62 MPa for high performance concrete, between 30 and 45 MPa for high early strength concrete and between 28 and 45 MPa for latex modified concrete overlay.

Table 6.5 Responses on Maximum Compressive Strength for different types of Overlays

Concrete Replacement Overlays	Range of Compressive Strength (MPa)	
	Canada	USA
Normal Concrete	35	35
High Performance Concrete	50	62
High Early Strength Concrete	45	30
Latex Modified Concrete	45	28

Table 6.6 shows a difference in effectiveness between Canada and the United States, when using high early strength concrete patches. Canada shows a range of effectiveness of 50-70% compared to 70-100% in the USA. Canada and the USA both had the same effectiveness of 80% for asphalt concrete patches. Both Canada and the USA show an effective range of 70-100% for normal concrete. The USA have an 88% effectiveness rate for High Performance concrete patches, whilst there was no response from Canadian respondents.

Table 6.6 Effectiveness of different types of Patches

Concrete Replacement Patches	Range of Effectiveness (%)		Mean Effectiveness (%)	
	Canada	USA	Canada	USA
Normal Concrete	70-100	70-100	79	89
High Performance Concrete	N/A	75-100	N/A	88
High Early Strength Concrete	50-70	70-100	63	90
Asphalt Concrete	80	70-90	80	80

Table 6.7 shows that the responses obtained from both Canada and the USA for effectiveness of overlays were similar. Both Canadians and US respondents have an effectiveness range greater than 70% for high performance concrete. Canada has an effectiveness range of 70-100% for normal concrete overlay while the USA had a range of 90-95% indicating that the USA achieved better results using this technique. The effectiveness range for high early strength concrete was 75% for Canada (only one

response) while 70-90% for the USA. Both latex modified concrete (LMC) and asphalt concrete overlays had effectiveness greater than 80% for both Canada and the USA.

Table 6.7 Effectiveness of different types of Overlays

Concrete Replacement Overlays	Range of Effectiveness (%)		Mean Effectiveness (%)	
	Canada	USA	Canada	USA
Normal Concrete	70-100	90-95	88	93
High Performance Concrete	70-90	75-100	81	92
High Early Strength Concrete	75	70-90	75	80
Latex Modified Concrete	80-90	90-100	84	93
Asphalt Concrete	80	70-100	80	88

Table 6.8 shows quite a big difference in responses between Canada and the USA on the use of different types of patches. 57% of Canadians used normal concrete compared to 19% for Americans. However, both Canada and the USA had similar range of usages ranging from as low as 5% to as high as 90%. 28% of Canadian respondents use high performance concrete patches compared to 65% of Americans although they had similar range of usages from as low as 5% to as high as 95%. 19% of Canadians used high early strength concrete patches compared to 34% of Americans although they had similar range of usages from as low as 1% to as high as 90%.

Table 6.8 Usages of different types of Patches

Concrete Replacement Patches	Range of Usage (%)		Mean Usage (%)	
	Canada	USA	Canada	USA
Normal Concrete	15-90	5-70	57	19
High Performance Concrete	5-75	5-95	28	65
High Early Strength Concrete	1-60	5-90	19	34
Asphalt Concrete	-	10-50	-	28

Table 6.9 compares the responses on the use of different types of overlays in Canada and the USA. 66% of Canadian respondents used normal concrete compared to 48% for Americans, 34% of Canadian respondents used high performance concrete compared to 51% for Americans, 53% of Canadian respondents used high early strength concrete

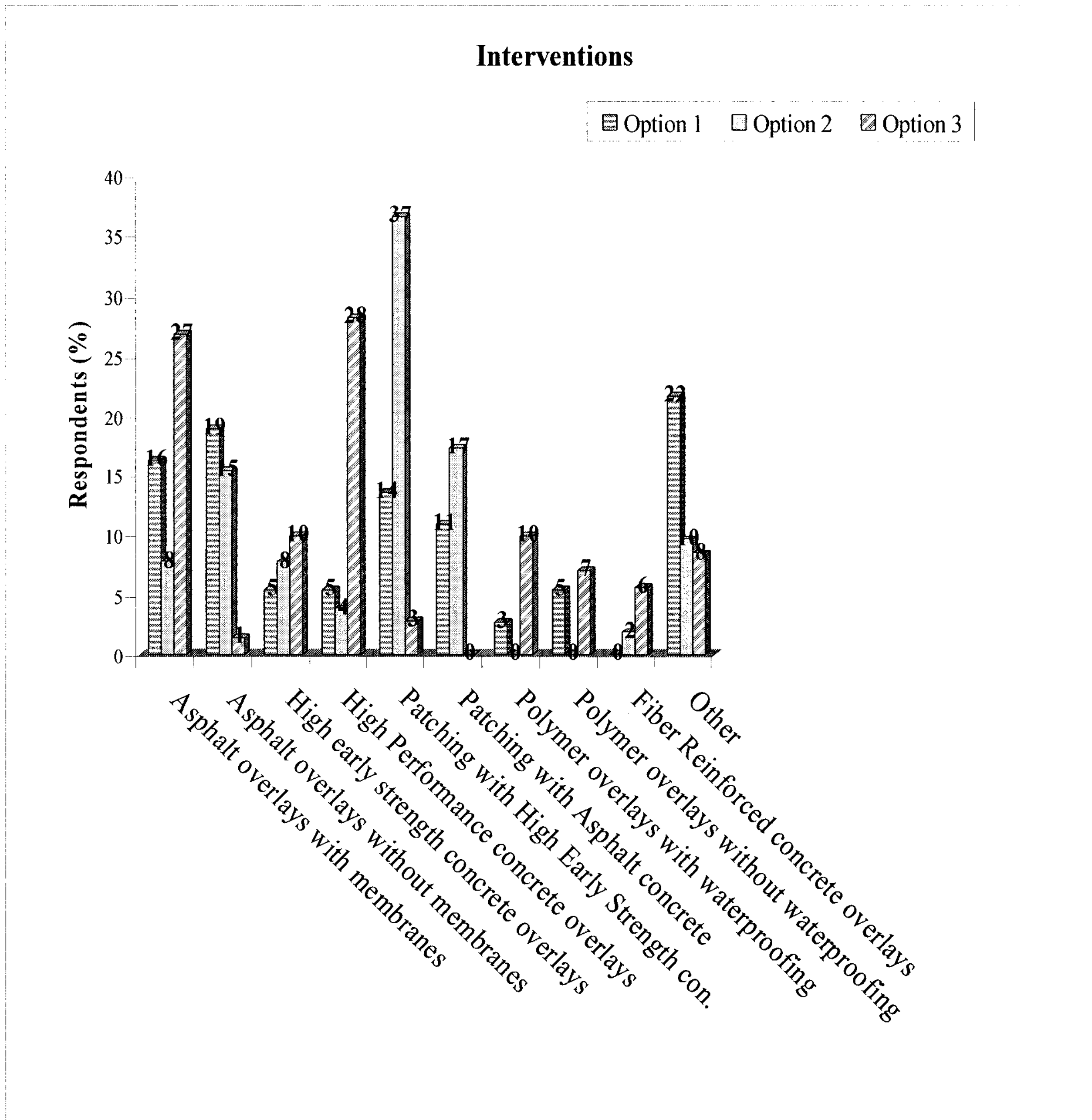
compared to only 20% for Americans, 19% of Canadian respondents used latex modified concrete compared to 35% for Americans and 65% of Canadian respondents used asphalt concrete overlays compared to 34% for Americans. However, both Canada and the United States had similar range of usages for different types of concrete overlays.

Table 6.9 Usages of different types of Overlays

Concrete Replacement Overlays	Range of Usage (%)		Mean Usage (%)	
	Canada	USA	Canada	USA
Normal Concrete	15-90	5-90	66	48
High Performance Concrete	1-75	1-100	34	51
High Early Strength Concrete	10-95	10-30	53	20
Latex Modified Concrete	2-85	5-80	19	35
Asphalt Concrete	5-95	1-96	65	34

Figure 6.13 compares different types of intervention used based on combined data obtained from Canada and the USA. Option 2 (rapid repair) at 37% using high early strength concrete for patching was mostly used. Option 2 (patching with asphalt concrete) is at 17%, high performance concrete overlays (primarily used for Option 3 in complete deck rehabilitation) is at 28% and asphalt overlays with membranes (option 3) is at 27%. The most common interventions for Option 1 are asphalt overlays with membranes (16%) and asphalt overlays without any membranes (19%).

Table 6.10 shows the average thickness and life span of various interventions based on the combined Canadian and USA data. Option 1 had the lowest thickness at 24 mm for Polymer overlays without waterproofing (only American data was obtained for this intervention). Option 1 also had the highest thickness at 88 mm for Asphalt concrete patches. For Option 2, high early strength concrete patches at 70 mm were the thickest intervention; whilst Asphalt concrete patches at 53 mm were the least thick intervention (only American data was obtained for this intervention). For Option 3, Fiber Reinforced concrete overlays at 71 mm were the thickest intervention; whilst high early strength concrete overlays at 47 mm were the least thick intervention.



Option 1 - Rapid Protection, Option 2 - Rapid Repair, Option 3 - Complete Deck Rehabilitation

* Other Option 1 is made up of 3% Low Slump Concrete Overlay and 19% No Response.
** Other Option 2 is made up of 2% Low Slump Concrete Overlay, 2% Epoxy Patches and 6% No Responses.
*** Other Option 3 is made up of 1% Low Slump Concrete Overlay, 1% Normal Concrete, 1% Gemcrete and 4% No Response.

Figure 6.13 Various Types of Interventions used

Table 6.10 Average Thickness for various types of Interventions

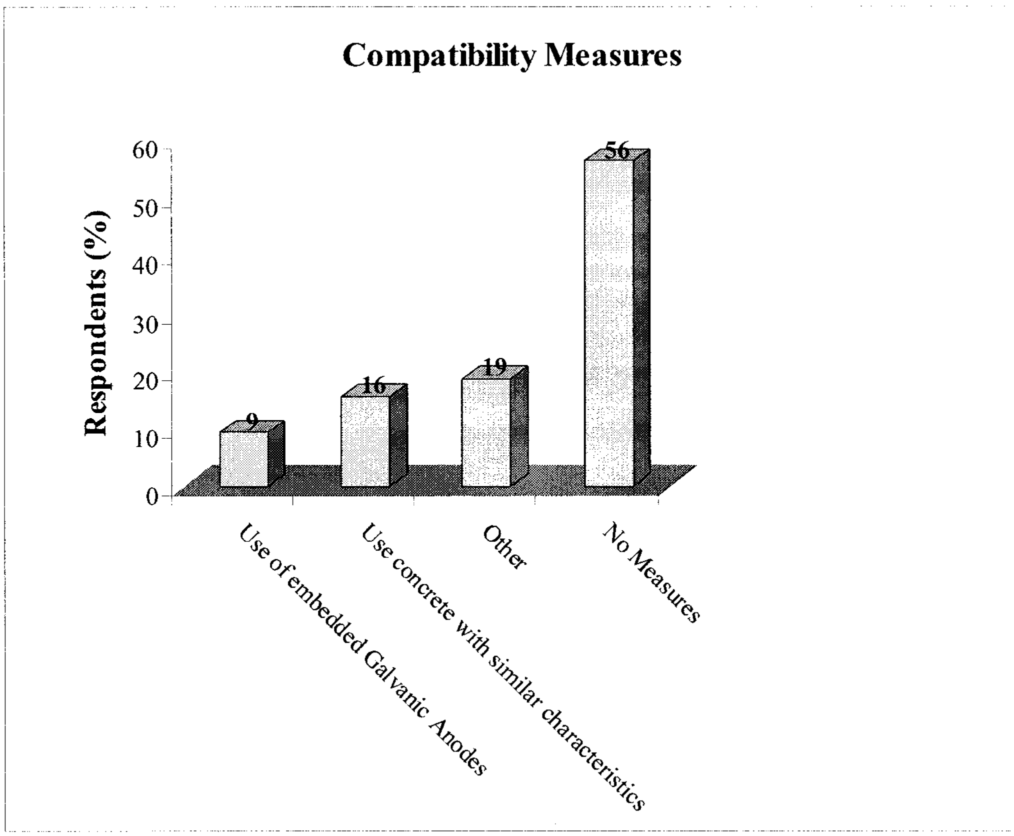
Interventions	Average Thickness (mm)			Average Life Span (yrs)		
	Option			Option		
	1	2	3	1	2	3
Asphalt overlays with membranes	69	64	65	14	15	22
Asphalt overlays without membranes	87	54	50	7	10	25
High Early Strength concrete overlays	50	62	47	13	19	22
High Performance concrete overlays	63	63	60	12	25	26
Patching with High Early Strength concrete	57	70	N/A	15	12	N/A
Patching with Asphalt concrete	88	53	N/A	2	6	N/A
Polymer overlays with waterproofing	50	N/A	51	13	N/A	21
Polymer overlays without waterproofing	24	N/A	57	14	N/A	22
Fiber Reinforced concrete overlays	N/A	60	71	N/A	28	30

Option 1 - Rapid Protection, Option 2 - Rapid Repair, Option 3 - Complete Deck Rehabilitation

Rapid protection interventions gave the lowest life span (2 years for patching with asphalt concrete), whilst complete deck rehabilitation yields the highest life span (30 years for fiber reinforced concrete overlays). For option 1, the highest life span was 15 years with high early strength concrete patches while the lowest life span was with Asphalt concrete at 2 years. For option 2, the highest life span was 28 years with fiber reinforced concrete overlays while the lowest life span was with the use of asphalt concrete patches at 6 years. For option 3, the highest life span is 30 years when fiber reinforced concrete overlays while the lowest life span was with polymer overlays with waterproofing at 21 years. Interestingly, for options 1 and 3 it can be seen that polymer overlays with or without waterproofing had very similar life expectancy. The same can be said for Option 3 when comparing asphalt overlays with and without waterproofing membranes.

6.4 Concrete Compatibility (Question 9)

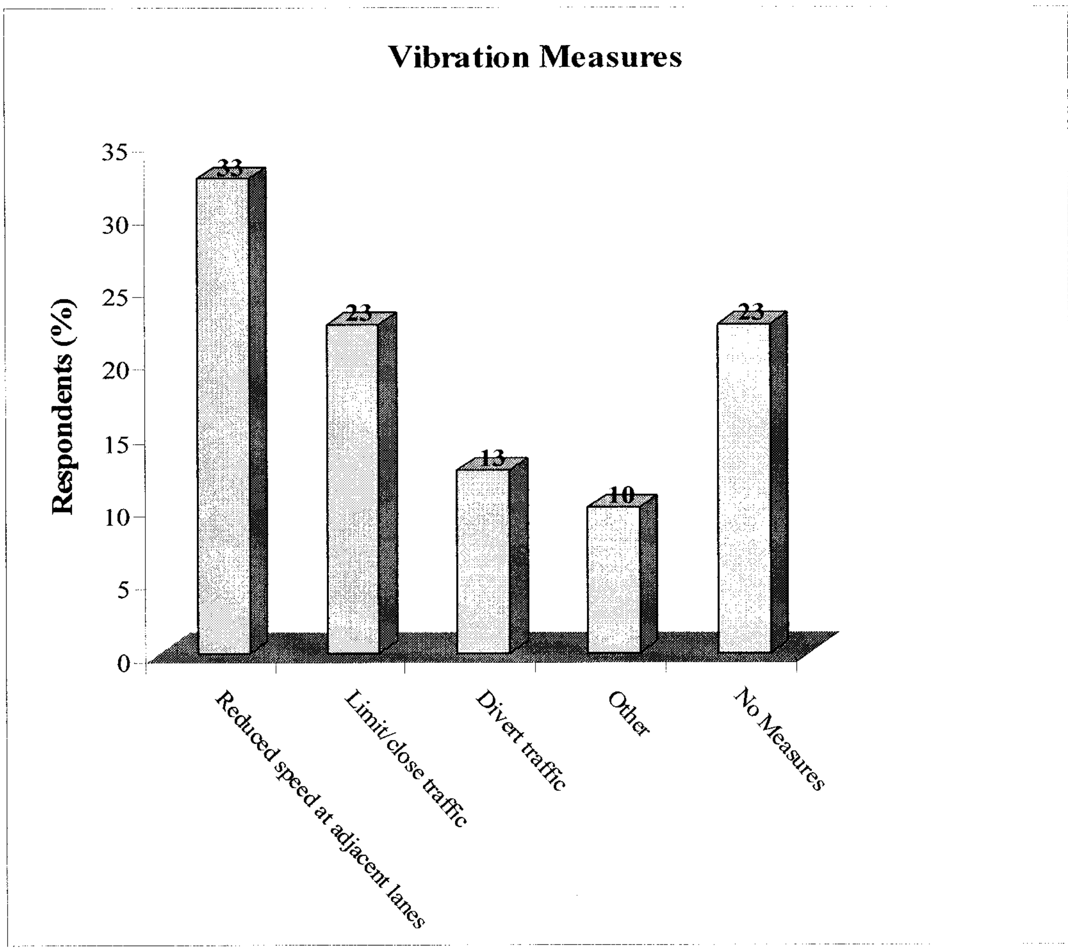
For this question all of the data was combined as there were no significant differences in responses between Canada and the United States. Figure 6.14 shows that 56% of the respondents do not perform any specific measure to ensure that there is compatibility between the new and old concrete. 16% of the respondents use concrete with similar characteristics that is if the residual concrete was normal concrete, the repair material used will also be normal concrete. 9% of the respondents use galvanic anodes to protect the reinforcing steel from corrosion. 9% of the respondents also said that they normally roughen the residual concrete surface as well as spray a bonding agent (slurry/latex material) before the new concrete is placed.



* Other is made up of 3% Corrosion Inhibitors, 3% Use of Lithium to mitigate ASR, 3% Check level of chloride and 10% Rough Surface and use Bonding agent.

Figure 6.14 Compatibility Measures

Canadian and USA responses on vibration measures were similar and hence combined together in Figure 6.15. 33% of the respondents normally reduce speed of the adjacent lanes in order to minimize vibrations. 23% limit/close traffic to help reduce vibrations, whilst 23% have no measure in place to minimize vibrations. Several of the respondents indicated that they never had any problems with the vibrations produced by motorists during the bond strength development between the new and old concrete. 13% of the respondents indicated that they normally divert traffic during the bond development phase of new and old concrete.

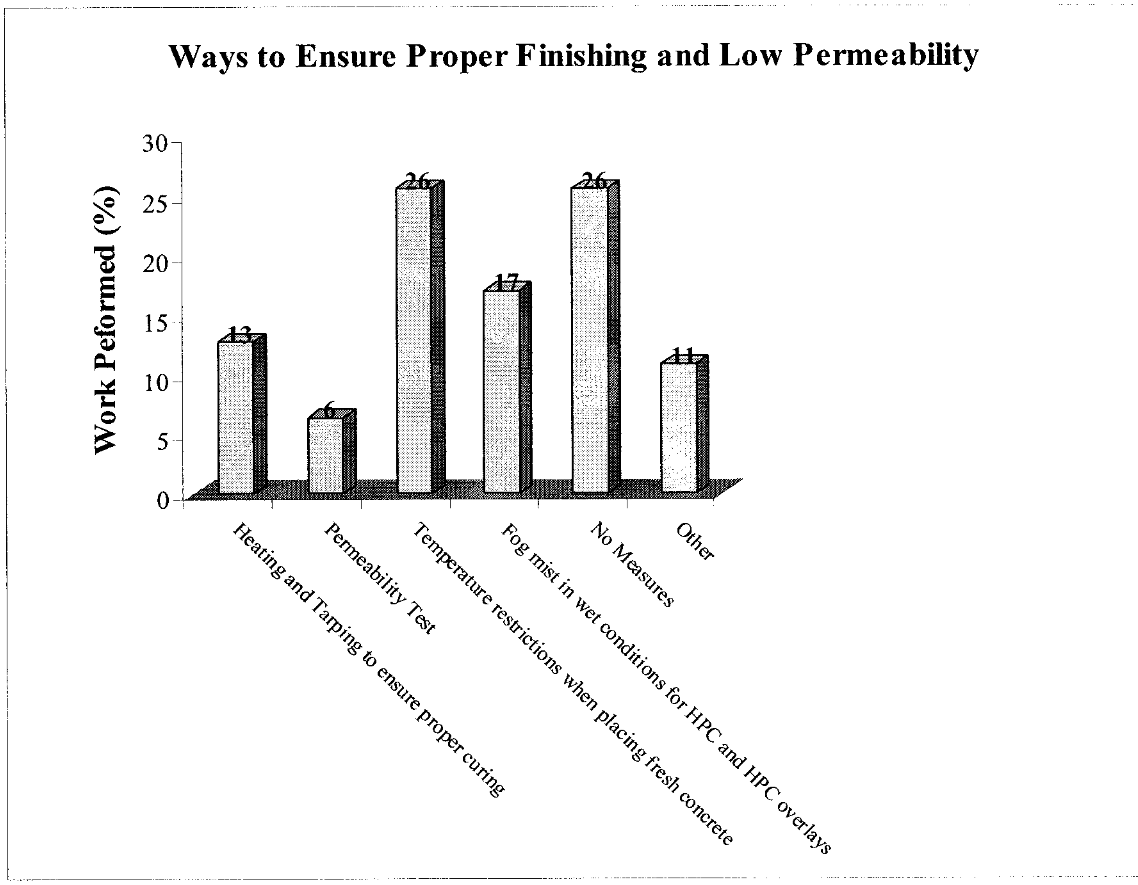


* Other is made up of 3% No Overlays on Lively Decks (Steel Girders), 3% Pour during off peak hours (nights/weekends) and 4% Disconnect diaphragms between girders.

Figure 6.15 Vibration Measures

6.5 Proper Finishing and Curing of Concrete Questions 10 & 11)

No significant differences in responses between Canada and the United States were observed and hence data are combined. Figure 6.16 shows that 26% of the respondents normally have temperature restrictions when placing fresh concrete. For example, the concrete is cooled down during the summer to avoid too much evaporation and heated during the winter to ensure proper hydration of concrete. 26% of the respondents have no measures in place to ensure proper finishing and low permeability. For high performance concrete, 17% of the respondents perform fog misting to ensure proper finishing and to avoid any cracking from occurring. 13% of the respondents provide heating and tarping to ensure proper curing mainly to avoid cracking and to ensure low permeability.

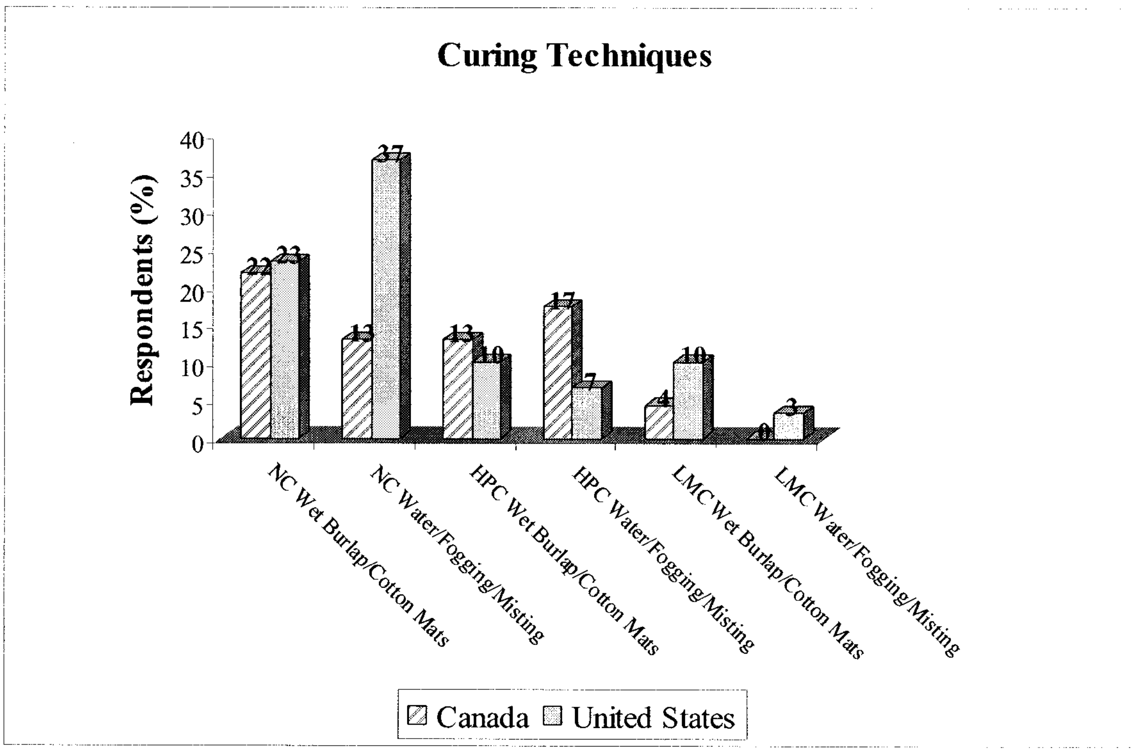


* Other is made up of 2% Spec a Rapid Chloride Permeability Value, 2% Retarders in concrete for workability, 2% Trial Placements and 5% Machine Finishing.

Figure 6.16 Ways to Ensure Proper Finishing and Low Permeability

For this question there were generally two major curing groups, one containing wet burlap with black plastic sheets or moist cotton mats. These two materials were grouped together because they act similarly and the curing procedure for each is also similar. The other group contained curing with water, fogging or misting, since all three used water.

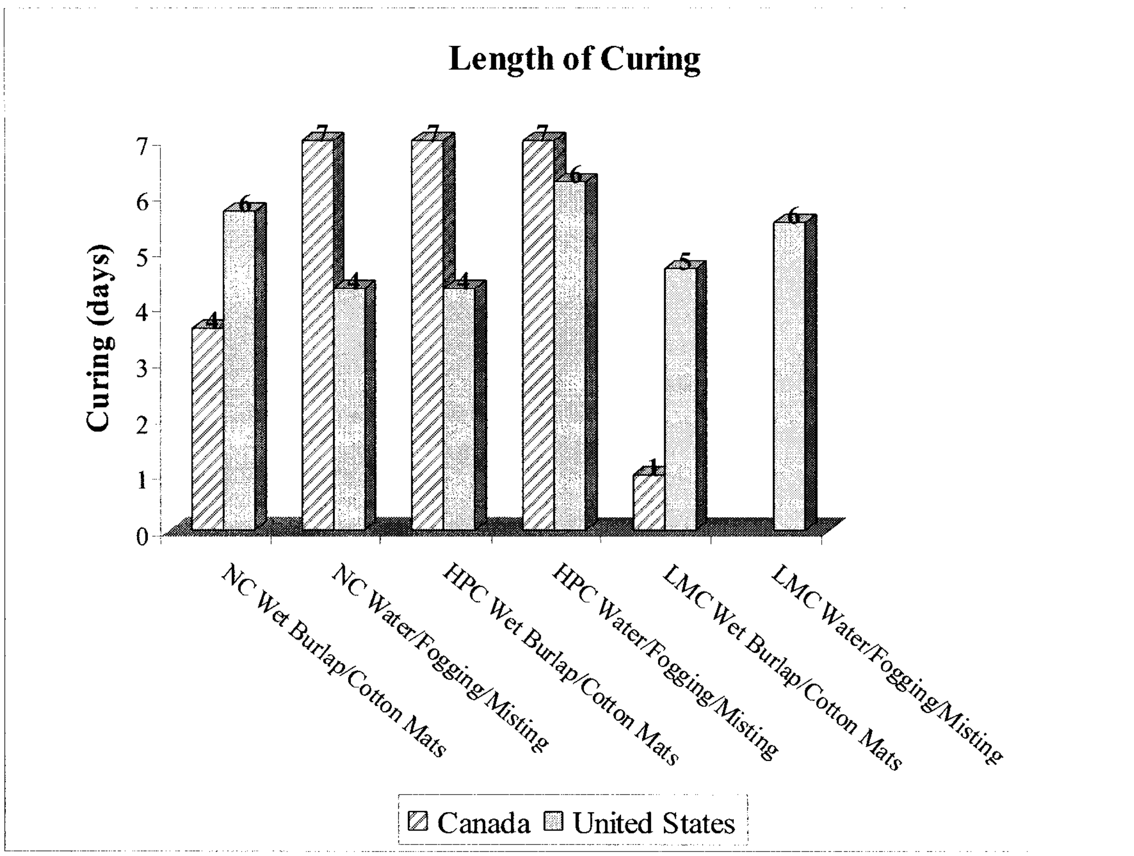
Figure 6.17 shows that 37% of the Americans cure normal concrete using water in some form compared to only 13% of the Canadians. 23% of the Americans use wet burlap or moist cotton mat compared to 22% of Canadians. 13% of Canadian respondents cure HPC using wet burlap or moist cotton mats compared to 10% of Americans. When water, fogging or misting is used to cure HPC, 17% of the Canadian use this technique compared to 7% of the Americans. 4% of the Canadian use LMC curing using wet burlap or moist cotton mats technique compared to 10% of the Americans.



* NC - Normal Concrete, HPC - High Performance Concrete, LMC - Latex Modified Concrete

Figure 6.17 Curing Techniques

Figure 6.18 shows that the duration of curing for each technique and no significant difference is observed between Canadian and American respondents. The only big difference was curing with LMC using wet burlap or moist cotton mats. One of the Canadian respondent indicated that 1 day is needed to cure LMC whilst the American respondents indicated that they needed on average 5 days.



* NC - Normal Concrete, HPC - High Performance Concrete, LMC - Latex Modified Concrete

Figure 6.18 Curing Duration

The curing effectiveness data for both the Canadian and American respondents are combined and analyzed together in Table 6.11. 88% of the respondents were able to effectively cure latex modified concrete (LMC) using either wet Burlaps sheets or moist cotton mats. 87% of the respondents were able to effectively cure high performance concrete (HPC) using some form of curing technique involving the use of water, fogging or misting. 85% of the respondents were able to effectively cure normal concrete (NC) using either wet burlap or moist cotton mats. The least effective intervention to cure was the LMC using either water, fogging or misting at 70%.

Table 6.11 Curing Effectiveness of using various Interventions and Curing Techniques

Intervention/Curing Techniques	Range of Effectiveness (%)	Mean Effectiveness (%)
NC Wet Burlap/Cotton Mats	50-100	85
NC Water/Fogging/Misting	20-100	73
HPC Wet Burlap/Cotton Mats	60-100	83
HPC Water/Fogging/Misting	70-95	87
LMC Wet Burlap/Cotton Mats	75-100	88
LMC Water/Fogging/Misting	70	70

NC - Normal Concrete, HPC - High Performance Concrete, LMC - Latex Modified Concrete

6.6 Bridge Decks Service Life (Questions 12 & 13)

Canadian and American respondents provided various definitions for the term "Service Life" which are provided in Tables 6.12 and 6.13.

Table 6.12 Definition of Service Life according to Canadian Respondents

Definition of Term "Service Life"	Source
Service life is the remaining portion of the design life	St. John, Dept. of Works
Service life is the time period that the structure could remain in service without any major rehabilitation/maintenance work.	Ontario Ministry of Transportation
Service life is the period of time (real time) the structure is maintained in a safe condition and is kept in service.	Nova Scotia, Department of Transportation
Service life is the number of years that a structure is not practical/economical to carry out the rehabilitation	Morrison Hershfield Ltd.
Duration that a component serves its intended purpose in a safe and functional manner	McCormick Rankin Corp.
OHBDC Definition	URS Canada Inc.
Service Life equal Useful Life of the Structure. When it needs Replacement/Rehabilitation	Seaway International Bridge Ltd.
Service Life is defined as the number of years the bridge deck can be kept in operation before requiring complete replacement	City of Ottawa
Service Life is defined as the time until the structure will be scheduled for a condition survey prior to programming for repair	City of Toronto
Service Life is defined as the useful life of the bridge for the purpose intended	City of Winnipeg

Table 6.13 Definition of Service Life according to American Respondents

Definition of Term "Service Life"	Source
Service Life varies depending on such attributes as environment, existing conditions and expected element performance	Kansas DOT
Service Life is the time until deck replacement	Rhode Island DOT
Functionally open to traffic	Pennsylvania DOT
Time that a particular element remains in use. Some repair/rehabilitation maybe required during an elements service life	Illinois DOT
Apparent time between one treatment and the next, where deterioration forces the next deterioration	Montana DOT
Time until next major rehabilitation	Wisconsin DOT
The number of years that a structure (or element) remains in service with reasonable maintenance cost	Maine DOT
Duration of satisfactory performance of an element under routine operating and maintenance conditions	Minnesota DOT
Time until major rehabilitation or replacement is required	New Hampshire DOT
AASHTO Definition	New Jersey DOT

There were no significant differences in responses between Canada and the United States regarding service life estimation based on various properties. Figure 6.19 shows that 23% prefer service life prediction based on delamination compared to 21% for measurement of permeability of chloride ions and 18% for corrosion of reinforcing bars.

Figure 6.20 shows that the United States clearly spend more money in bridge deck rehabilitation as compared to Canada. 50% of the Canadian respondents had an annual budget of 1-10 million dollars. 36% of the American respondents had an annual budget of 10-30 million and 21% had 30-100 million dollars. 25% of Canadians and 21% of American respondents were not able to provide an indication of their annual budget. Some explained by saying that they had no exact records, whilst others said that they were not at liberty to disclose that kind of information.

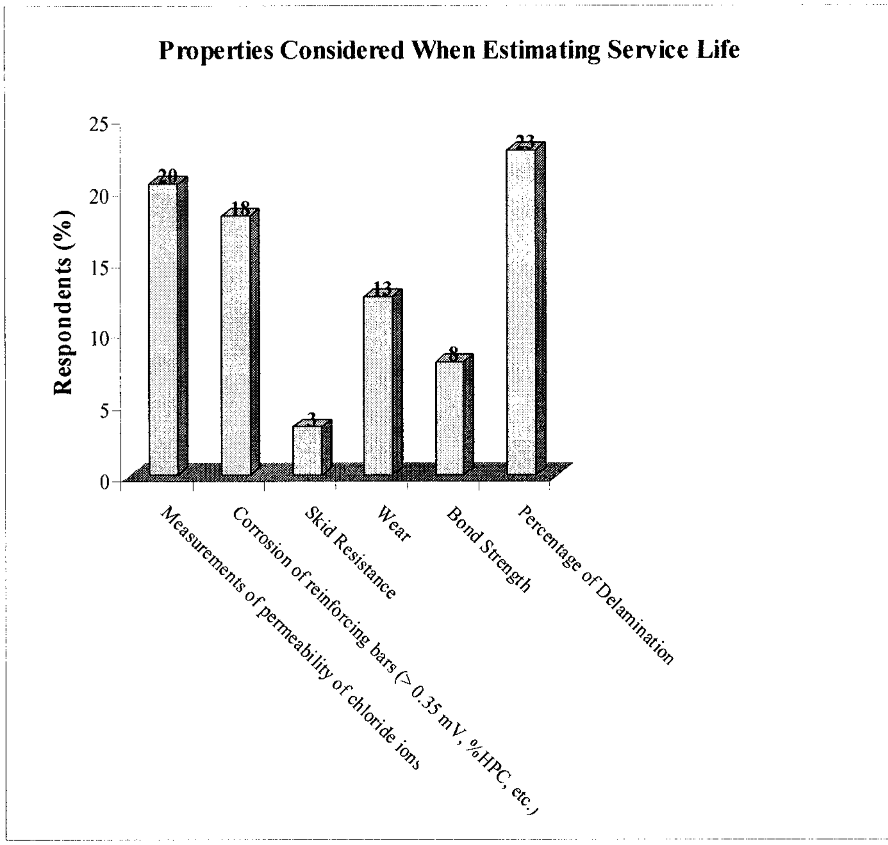


Figure 6.19 Properties Considered When Estimating Service Life

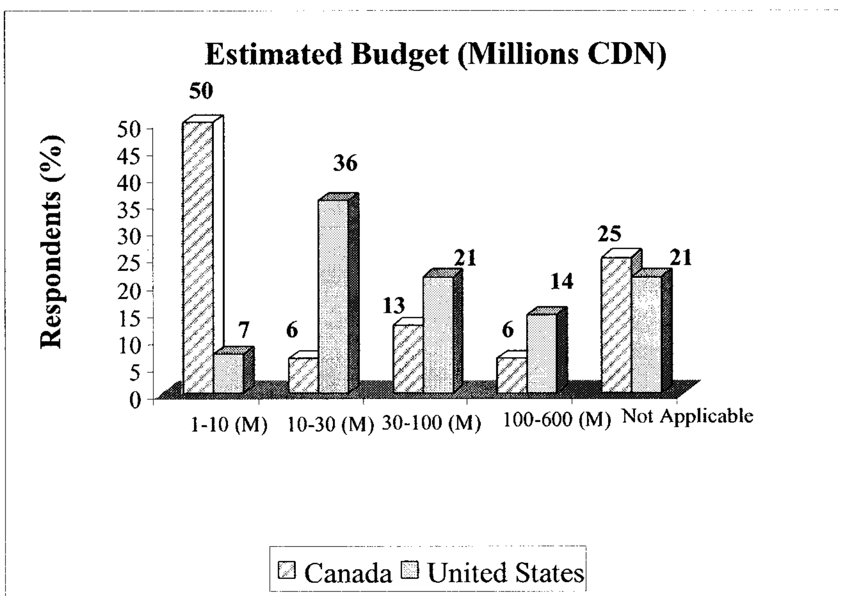


Figure 6.20 Estimated Budgets

Figure 6.21 shows that a fair number of organization/agencies (64%) in the United States looked after over 3000 bridges whilst only 12% of Canadian organizations/agencies looked after more than 3000. This explains why the United States estimated budget is larger. 25% of Canadian respondents looked after 1-100 bridges, 25% looked after 100-1000 bridges and another 25% looks after 1000-3000 bridges.

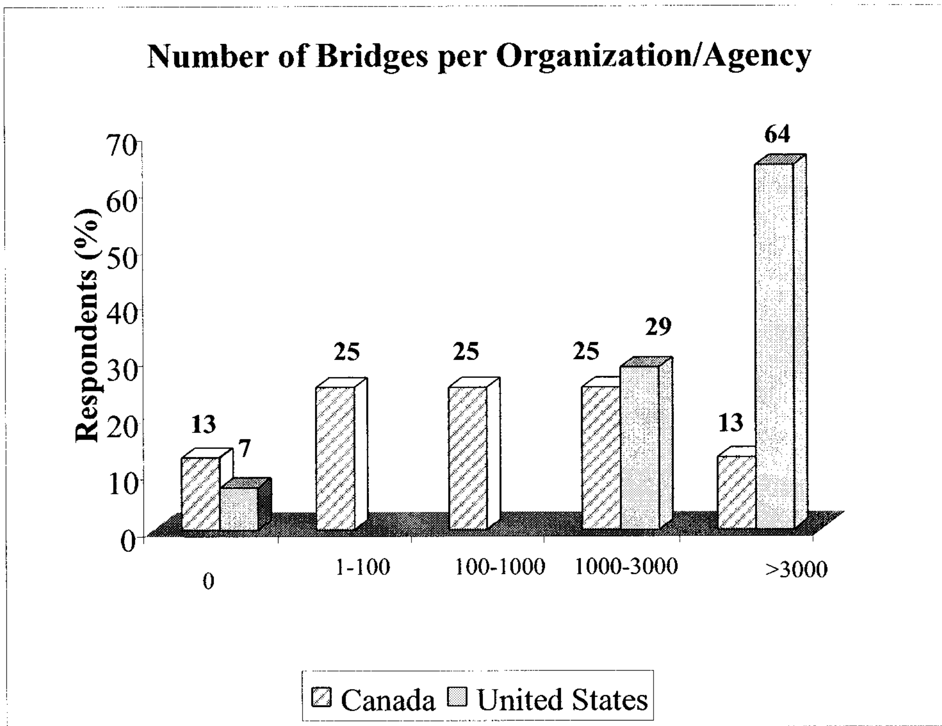


Figure 6.21 The number of Bridges per Organization/Agency

Table 6.14 shows that there is quite a similarity in prices per square feet between Canada and the United States. However for each type of Intervention, for both Canada and the United States there are a large number of respondents who were unable to give an estimate of their costs. After analyzing the responses obtained, many of them said that they were not in a position to disclose this confidential information. In general, Canada spend more for complete deck replacement when compared to the United States (four respondent's greater than $500/ft^2 for Canada as compared to 2 respondents for the

United States). Canada also spend more for complete bridge replacement (seven respondents greater than \$500/ft^2 as compared to 3 respondents for the United States).

Table 6.14 Costs for Various types of Interventions

Intervention Cost (\$CDN/ft^2)	**Number of Respondents**							
	Patching		**Deck Overlay**		**Deck Replacement**		**Bridge Replacement**	
	Canada	**USA**	**Canada**	**USA**	**Canada**	**USA**	**Canada**	**USA**
25-50	2	5	1	2	1	1		
100-500	5	1	2	2	3	5	1	2
500-1000					3	2	2	1
1000-3000					1		5	2
Not Available	9	8	13	10	8	6	8	8

7.0 Conclusion and Recommendation

This thesis was primarily made up of four components, Condition Surveys (Chapter 2), Concrete Removal Techniques (Chapter 3), Bridge Rehabilitation Techniques/Methods (Chapter 4) and the results from the questionnaire (Chapter 6). Conclusions and Recommendations for each chapter will be discussed separately in the following sections.

7.1 Conclusions for Chapter 2

- Visual Observation Surveys are relatively cheap, and can be carried out relatively quickly with minimal traffic disruptions. It is also the most common form of non destructive evaluation used during bridge inspections (Phares et al., 2000). However it has one disadvantage as it does not reveal hidden defects or deteriorations occurring inside of the concrete deck.

- For Delamination Surveys, generally three types of non destructive tests can be carried out, they are the chain drag, radar survey and/or impact echo method. The chain drag test is relatively inexpensive and can be done relatively quickly. However it has a couple of disadvantages as described as follows:

    - For accurate readings, it relies primarily on the experience of the operator being able to accurately detect concrete delamination using his/her ears.
    - It is recommended that this test be done on concrete surface only. When done on asphalt surfaces concrete delamination can be difficult to detect and observations can be unreliable and inaccurate.

Ground Penetrating Radar surveys can be used to detect concrete delamination, rebar locations and asphalt thickness. A radar survey can gather a high volume of data in a short period of time with minimum traffic flow disruptions. Most radar surveys are carried out on mobile vehicles, allowing data to be gathered at traffic speeds. However processing the data is very time consuming if done manually.

For this survey to be efficient it requires a specialized data processing software to analyze the data gathered (Maser, 1996). The main benefit of this survey is it allows organizations/agencies and option to regularly monitor there bridge decks. However the accuracy of this method has been debated by many researchers, as it is only moderately successful in estimating the percentage of deteriorated concrete on bridge decks (Halabe et al., 1997).

The Impact Echo method is a very accurate method for detecting concrete delamination, however it is very time consuming and expensive (Scott et. al, 2002). It often requires lane closures or complete closures of the bridge deck for several hours to perform this method.

- There are two types surveys used in detecting rebar corrosion, they are Half Cell Potential surveys and Linear Polarization Resistance surveys. The main difference between these two techniques is that the Half Cell Potential survey can only determine if the structure is corroding whilst Linear Polarization Resistance surveys can directly measure the corrosion rate of the steel reinforcement in the concrete structure. However both methods give accurate results if they are done properly. Half Cell Potential surveys has the disadvantage of being very time consuming, it requires the bridge to be closed for 7-12 hours and it can only be done on dry decks. Linear Polarization Resistance surveys on the other hand, are a very quick method, very simple to carry out and if done properly give a good indication of the corrosion rate occurring in the rebars.

7.2 Recommendations for Chapter 2

- Visual Observation surveys are the cheapest surveys used in bridge deck evaluations, hence the reason they are the most widely used. This survey should be done at regular intervals to monitor the structure for any signs of concrete deteriorations. This type of survey requires experienced and well trained personnel in order to effectively monitor the structure. Therefore proper training of inspectors is required in order to detect early signs of concrete deterioration.

- To aid visual observation surveys, other surveys such as Ground Penetrating Radar and Linear Polarization Resistance should also be carried out. Both can be done relatively quickly with minimum traffic disruptions. However before Ground Penetrating Radar surveys can be used more regularly by organizations/agencies a lot more research is required to improve its accuracy. Linear Polarization Resistance has been around since the late 1980s; however it is only in the last few years it has been developed for use on bridge decks. Although accurate, a lot of organizations/agencies are still not familiar with this type of survey.
- Whenever it is possible to close the entire bridge to perform condition surveys, then additional surveys such as the Impact Echo method and Half Cell Potential surveys should be carried out, because both give very accurate results, but are time consuming.

7.3 Conclusions for Chapter 3

There are three common types of concrete removal techniques they are, Hand Held Pneumatic Breakers (HHPB), Milling and Hydrodemolition.

- HHPBs are relatively cheap removal equipments but they are highly labour intensive. They are the most common type of concrete removal equipment. HHPB can be made in a variety of sizes for different types of applications. They are also used to remove concrete from overhead or vertical surfaces. HHPB are commonly used on projects involving patching, in small areas from between, around and below steel reinforcement and anywhere larger concrete removal equipments are not accessible.
- Milling and Hydrodemolition are both capital intensive methods (i.e. very little manpower is involved). Milling is a simple process which relies on weight and power of a large machine to remove concrete by repeated impacts of multiple cutting teeth mounted on a rotating cutting mandrel. Milling machines are very expensive and they require regular maintenance to work properly. However they are high production removal equipments which are capable of removing both

asphalt and concrete in one pass. Milling machine can only be used to remove concrete above the reinforcing steel. If the machine comes in contact with the steel reinforcement its cutting teeth can easily be damaged.

- Hydrodemolition is highly sophisticated, which utilizes a computer calibrated high pressure waterjet to destroy the cement matrix to remove the concrete. Hydrodemolition equipment is also very expensive and requires intensive maintenance to work properly (Vorster et al., 1992). Hydrodemolition can remove concrete between, around and below the steel reinforcement. However this removal technique has a big disadvantage of producing waste (slurry and run off water) that can harm the environment if not properly contained.
- A major disadvantage of both Milling and Hydrodemolition is they require a good portion of the bridge be closed for an extended amount of time to allow contractors to properly setup these equipments. HHPB in contrast requires minimal set time and space.

7.4 Recommendations for Chapter 3

Generally HHPB are primarily for patch repair or together with either Milling or Hydrodemolition. Milling and Hydrodemolition on the other hand are used when complete overlays are required. A lot more research is still needed for Hydrodemolition because it is still a relatively new technique. However once its equipment is properly set up it produces high production rates if no maintenance is required. This technique is growing in popularity from the response obtained from the questionnaire (Chapter 6 of this thesis).

There are four factors that determine the selection of the appropriate removal equipment as outlined by Vorster et al. (1992). They are as follows:

- Quality – the chosen concrete removal technique must satisfy the quality constraints of selectivity, residual damage and bond quality. If the chosen method cannot satisfy these requirements it will be not be feasible and should not be chosen.

- Availability – the equipment, materials and labour needed to apply the chosen removal equipment must be readily available. It should also be possible to use the chosen removal equipment on site without any major delays.
- Flexibility – the chosen concrete removal technique should be able to accommodate changes in scheduling, quantity and speed or work brought about by unforeseen changes which might occur during the duration of the project.
- Total Cost – it is important to consider not only the unit cost of performing the work, but other costs as well in determining which removal technique to choose. Other costs which are considered in the financial analysis are traffic control, repeated mobilization, limited access times and the cost of user delays.

7.5 Conclusions for Chapter 4

- There are nine common defects found on bridge decks, they are: effect of salt, overloading, scaling, cracking, delamination, spalling, chloride contamination, honeycombing/air pockets and frost damage.
- There are six factors which affect the selection of an appropriate rehabilitation technique, they are: cost analysis, importance of structure, type of structure, structure service life, contractor expertise and environmental concerns.
- There are three types of rehabilitation methods, they are: patching, overlaying and protection systems. Patching restores structural integrity and function of the residual concrete. It is important to note, when carrying out patching that the repair material be of similar physical characteristics to the residual concrete. These include similar compressive and tensile strength, low shrinkage and permeability, and low water to cement ratio to inhibit moisture and chloride penetration into the repair material.

  When concrete deterioration reaches below the top mat of the reinforcing steel deck replacement becomes more practical as opposed to repairing/patching. Repairing/Patching can only provide temporary corrosion control. It cannot stop corrosion from reoccurring with time, whilst a complete overlay can. Overlays should provide good strength, protect the reinforcing steel, resist abrasion, resist freezing and thawing and should also bond strongly to the concrete deck.

Protection systems were primarily designed for undamaged concrete decks. It is cheaper for an organization/agency to install a protection system than it is to repair the deck when it has been damaged (Rostam, 1991). Protection systems can also be used on high chloride contaminated decks that show no signs of concrete deterioration (MTO, 1996). Essentially protection systems are effective in removing or inhibiting chloride ions from getting to the steel reinforcement. However they are very expensive to install.

7.6 Recommendations for Chapter 4

- Careful life cycle cost analysis is required to determine which rehabilitation technique/method should be chosen based on the six criteria mentioned earlier.
- Patching is a fairly rapid repair method. Most of the repair material can be open to traffic within 2-4 hours after placement. However, this method only provides a temporary solution to the overall problem of concrete deterioration and steel reinforcement corrosion. The repair material chosen should have low shrinkage and permeability otherwise cracking appears soon after.
- Overlays are being designed today to prevent deterioration caused by deicing salts and freezing and thawing. These designs also take into consideration increased truck size, which have increased dramatically over the last ten years. Organizations/agencies such as AASHTO, FHWA, OHBDC, etc. have made alterations in the way they now design bridge decks because of new technologies such as modeling software which provide a good indication of the service life of an overlay.
- Cathodic Protection is a very good protection system. It is being used on bridge decks and the results are impressive. It has proven to stop corrosion in stop corrosion in salt contaminated bridge decks, regardless of the chloride content (Dharmaratne and Gronvold, 1992).

- New rehabilitation materials such as High Performance concrete and Steel Fiber Reinforced concrete are becoming increasingly popular for use by organizations/agencies because of there main advantage of being able to provide a long and durable service life which requires low maintenance. These new materials also allow for the construction of smaller and lighter weight sections, thus reducing the dead load of the structure.
- Thin Bonded Overlays are also another new repair/rehabilitation material used by organization/agencies. Although not as popular as high performance concrete and steel fiber reinforced concrete, they offer an economical method for the rehabilitation of concrete pavements and deck surfaces. They are usually less than 100 mm in thickness. In order for thin bonded overlays to be effective it depends largely on the durability of its bond strength when it becomes attached to the residual concrete.

7.7 Conclusions and Recommendations for Chapter 6

An extensive literature for the bridge deck rehabilitation practices in North America is provided. Feedback from various government and private organisations across Canada and the USA in response to the questionnaire on various rehabilitation practices and issues provided valuable practical informations which can be very useful for the selection of appropriate techniques for future applications.

The favoured bridge deck rehabilitation techniques and practices adopted by various organizations across North America are summarized in three parts such as Part 1, Part 2 and Part 3 (Figure 7.1).

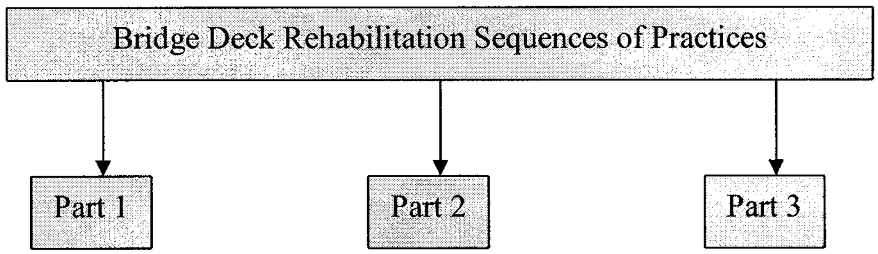


Figure 7.1 Bridge Deck Rehabilitation Sequences of Practices

7.7.1 Part 1

This part covers condition surveys, monitoring practices and rehabilitation starting dates adopted by North American organizations and are illustrated in Figure 7.2.

Condition Surveys

Corrosion potential survey (CPS) is the most common of the three condition surveys mentioned. It is used for detecting corrosion of rebars in the bridge deck. This survey is time consuming and requires a lane or complete deck be closed for 7-12 hours. 50% of the Canadian respondents use CPS as part of the deck assessment evaluation as opposed to 17% of the Americans. A Delamination survey is also preferred as it is cheap and can be done relatively quickly. It uses chains or impact hammers and is not very accurate as the accuracy depends on the experience of the operator being able to detect concrete delamination using his/her ears. Radar survey can be used to determine concrete delamination, rebar locations and asphalt thickness very quickly and with minimal disruption of traffic. This method requires the use of data processing software and the accuracy of this method has been debated by many researchers. However, researchers believe more work is still required to improve on its accuracy.

Structure Monitoring

Monitoring of structures is a good practice if organizations/agencies can afford to on a regular basis. Monitoring new structures is beneficial for the organization/agency. It can save money and provide more repair or protection options if problems are detected early as opposed to a severely damaged bridge deck without monitoring options. Monitoring old structures can determine how long the structure can remain in service before it will require major repairs.

Rehabilitation Starting Date

For a condition survey to be effective, actual rehabilitation of the deteriorated structure should begin no later than 1-2 years after a condition survey is carried out. After 1-2 years if the rehabilitation has not began, it is advised that another condition survey be carried out to further assess any additional damages caused to the structure.

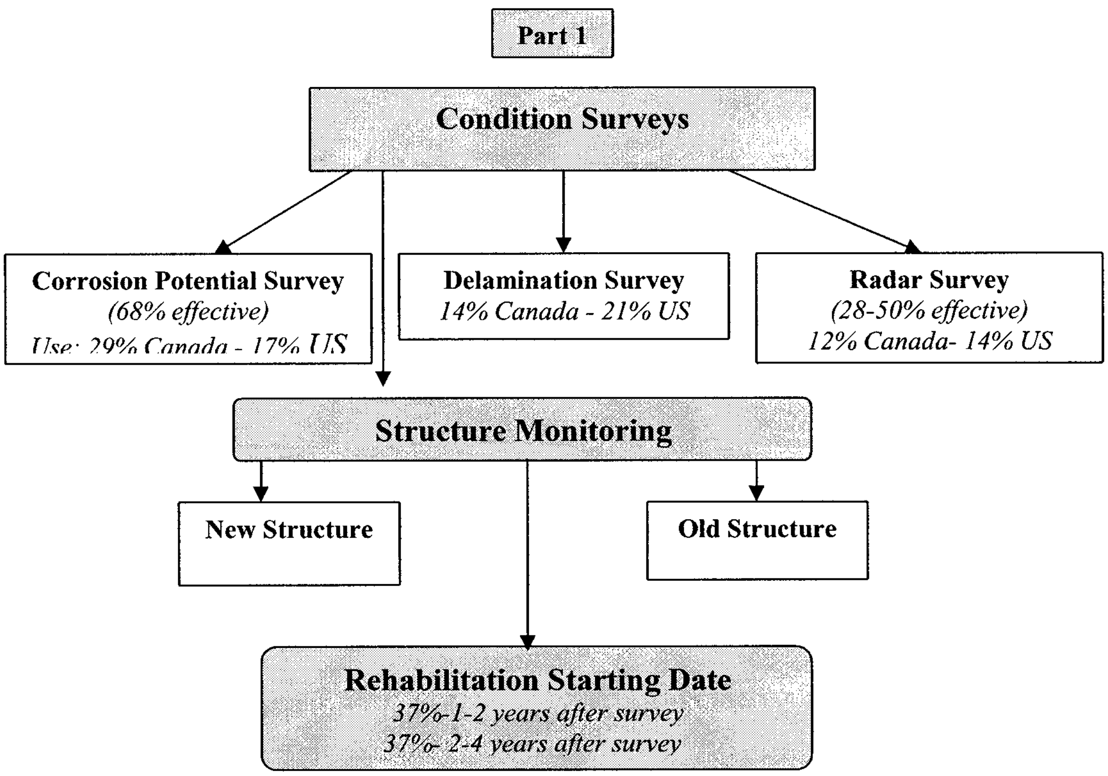


Figure 7.2 Condition Survey, Monitoring of Structures and Rehabilitation Starting Date

7.7.2 Part 2

Part 2 provides a summary of the preferred concrete removal techniques and associated procedures adopted by various organizations in North America and is illustrated in Figure 7.3. It covers (a) concrete removal methods/ techniques, (b) removal criteria, (c) organization/agency philosophy for intervention, (d) limiting crack propagation and (e) surface preparation techniques.

Concrete Removal Methods/Techniques

Three most common types of concrete removal methods/techniques are identified such as jackhammers, hydrodemolition and machine mounted demolition attachments. Jackhammers at 35%, are by far the most popular concrete removal equipment in North America, due to (a) its applicability on different types of bridge projects, (b) its cheapness and (c) its applicability to remove concrete from overhead or vertical surfaces. Hydrodemolition at 28%, is the second most common removal method and is highly productive despite some environmental concerns. All three removal methods/techniques are highly effective (>85%). However, Jackhammers are used 75% of the time compared to hydrodemolition at 15%.

Removal Criteria

Three choices are given to every organization/agency in the survey. 35% of the respondents indicated that they preferred visual and measured concrete delaminations alone to determine the type of intervention (rehabilitation choice) to carry out. 32% of the respondents preferred concrete delaminations and high corrosion potential values to determine their choice of intervention while 29% preferred the removal of a uniform thickness over the entire bridge deck to make a final decision about the adaptation of a type of rehabilitation technique. Sometimes after removal of this uniform thickness, sound concrete is encountered which show no sign of deterioration or damage. This can save organizations/agencies money by not having to repair the entire bridge deck.

Organization/Agency Philosophy for Intervention

Organizations/Agencies in North America preferred long lasting and less frequent interventions (67% of the respondents). Since many of these respondents were from urban cities, they were ready to spend extra money and time in using the proper repair/rehabilitation methods and practices so that they would not need to worry about repairing/rehabilitating for at least 20-25 years.

Limiting Crack Propagation

There are three common ways in which organizations/agencies limit crack propagation during concrete removal during bridge deck rehabilitation. 24% of the respondents indicated that the use of chipping hammers instead of heavier jackhammers helps to reduce microcracks in the residual concrete. 20% of the respondents indicated that the use of smaller jackhammers (<14kg) instead of larger ones also helps in reducing the formation of microcracks in the residual concrete. 20% of the respondents used sawcutting in the removal of the deteriorated concrete in order to minimize crack propagation while 16% of the respondents indicated that they had no measures in place to minimize/eliminate crack propagation.

Surface Preparation Technique

There are four common surface preparation techniques used by organizations/agencies across North America. 40% of the respondents used sandblasting/airblasting while 8% used waterblasting to prepare surfaces before overlays placement. These techniques are very good for removing rust on steel reinforcement and provide the benefit of eliminating rebar corrosion when new overlays are placed. 17% of the respondents use a slurry latex material (tack coat) while 16% use prewetting with sand/cement paste over the residual surface before the new overlay is placed. This helped to increase the bond strength between new and old concrete surfaces and also ensured monolithic action.

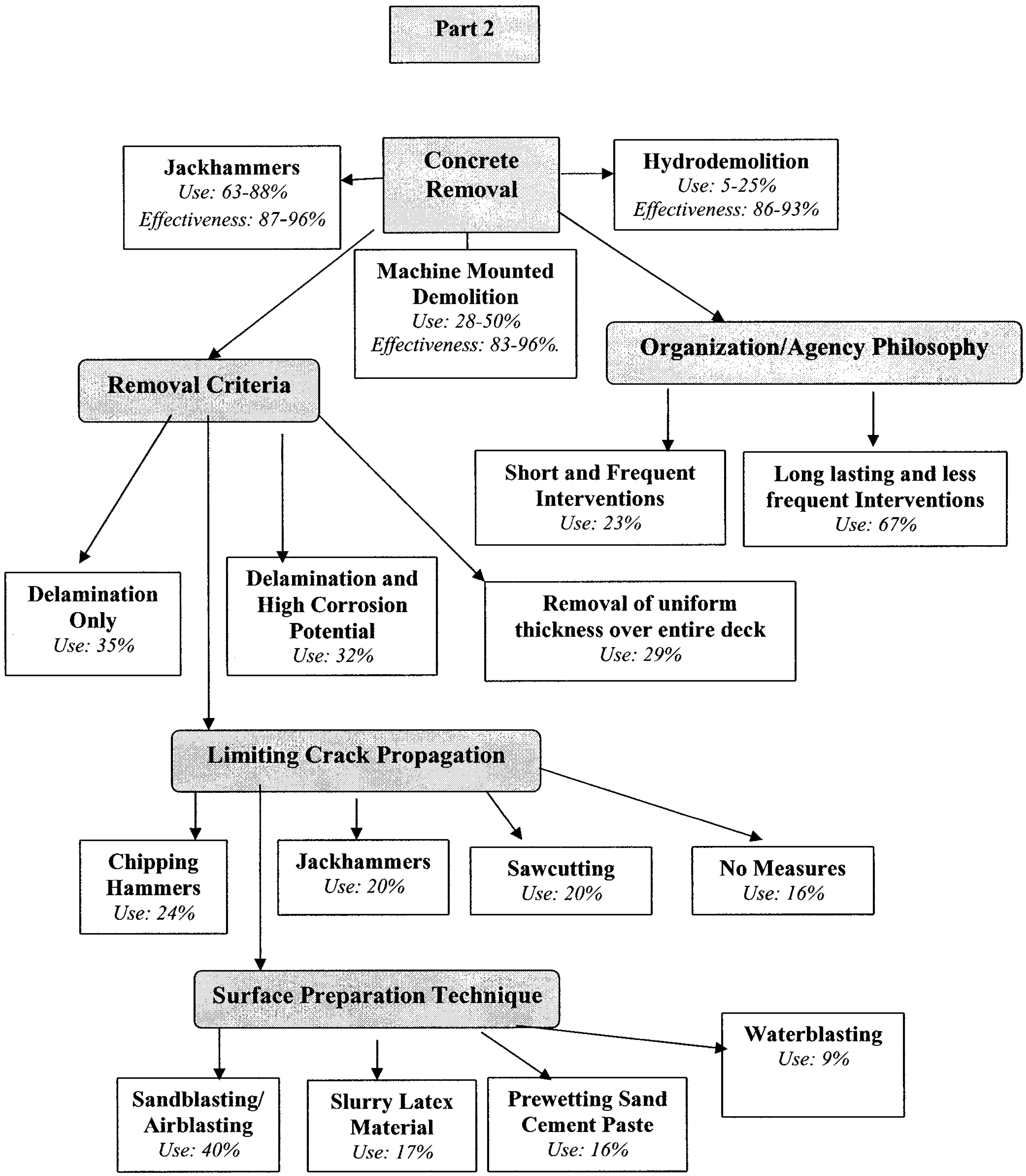


Figure 7.3 Concrete Removal Techniques and Associated Procedures

7.7.3 Part 3

Part 3 addresses briefly the types/modes of repair, materials/material compatibility, finishing techniques, vibration measures and curing techniques mostly adopted by various organizations in North America. Figure 5.4 is being used to guide readers to understand these procedures.

Rehabilitation Practices

There are two types of rehabilitation practices commonly carried out, they are patching and overlaying. Common patching materials are Normal Concrete (NC) (32% of the respondents used NC), High Performance Concrete (HPC) (18% of the respondents used HPC) and High Early Strength Concrete (HESC) (32% of the respondents used HESC). The average compressive strengths for these patching materials are as follows: 30 MPa for NC, 37 MPa for HPC and 30 MPa for HESC.

Common overlay materials are HPC (34% of the respondents used HPC), Latex Modified Concrete (LMC) (22% of the respondents used LMC), NC (14% of the respondents used NC) and Asphalt Concrete (AC) (15% of the respondents used AC). The average compressive strengths for these overlaying materials are as follows: 37 MPa for HPC, 32 MPa for LMC, and 31 MPa for NC.

Rapid Repair/Protection and Complete Deck Replacement:

There are three ways in which patches and overlays can be used. They can be used for rapid protection, rapid repair and complete rehabilitation of a bridge deck. For rapid protection the common material used is AC with membranes. AC is a temporary solution for protecting exposed and deteriorated concrete until major repairs are required. Once AC is properly compacted and rolled during placement, it should be low in permeability and it should be able to resist the penetration of chloride ions until major repairs are carried out. The waterproofing membrane also assists in resisting the penetration of chloride ions from attacking the already weaken deteriorated concrete. Rapid protection uses both Patching and Overlaying materials.

For rapid repair the common materials used are HESC and AC. Rapid repair only uses patching materials. These materials are primarily used to fill potholes caused by spalling and ponding of water.

For complete deck replacement the common materials used are HPC and AC with membranes. HPC is becoming increasingly popular for use as an overlay material because of it characteristics such long term durability, high compressive and tensile strength and low permeability. This results in HPC having a long service life (>25 years from respondents). AC with membranes provides a good protective coating for the concrete deck. In this case the concrete deck can be made of NC, LMC, or HPC. AC with membranes protects the concrete deck from moisture penetration as well as chloride penetration which can cause rebar corrosion.

Concrete Compatibility

When repairing/rehabilitating a bridge deck it is important to ensure that the chosen repair material is compatible with the residual concrete. If this is not the case, there will be no monolithic action between the new and old concrete, which will cause cracking and debonding to occur. Sixteen percent of the respondents suggested using concretes with similar characteristics to ensure monolithic action is achieved. Ten percent of the respondents suggested roughening of the old surface and applying a bonding agent. This will increase the bond strength between the two surfaces. Fifty six percent of the respondents said they had no measure in place to ensure concrete compatibility.

Vibration Measures

Another important issue to consider when placing fresh repair material is vibration produced from adjacent lanes due to traffic loads. These vibrations if not properly controlled has the potential of damaging the bond strength between the old and new concrete. If there is insufficient bond strength, cracking and debonding will almost definitely occur.

There are three common ways to reduce these vibrations; 33% of the respondents said reducing traffic speeds helps, 23% of the respondents said limiting or closing traffic during placement reduces any vibrations and 13% of the respondents said diverting traffic will also assist in reducing these vibrations. Twenty three percent of the respondents however admitted that they have no measure in place to reduce these vibrations.

Proper Finishing

After the repair material is placed, the next step is to ensure proper finishing and curing of the repair/rehabilitation material. There are three common ways to properly cure the repair material, they are discussed as follows:

- In the winter season, 13% of the respondents said it's a good practice to heat and tarp the fresh concrete. The will ensure that concrete hydration occurs instead of the fresh concrete freezing.
- Twenty six percent of the respondents believed it is a good practice to have temperature restrictions when placing fresh concrete. Some respondents suggested placing early in the morning or in the evening/nights when the weather is cooler. The will reduce evaporation and shrinkage from occurring. It will also help reduce small hairline cracks from developing due to rapid evaportation.
- Seventeen percent of respondents said that it is also good practice to fog mist when placing HPC overlays during hot days. Since HPC has a low water to cement ratio, and the fresh concrete is usually a lot drier when compared to NC, cracking can easily occur due to evaporation. Fog misting keeps the fresh concrete cool and moist. This lowers the heat of hydration, thus reducing the probability of hairline cracking from occurring.

Interestingly, 26% of respondents said that they had no measures in place in order to ensure proper finishing of the material.

Curing Techniques

There are two major curing groups; (1) wet burlap with plastic sheets or cotton mats and (2) curing with water either through fogging or misting. Twenty five percent of the respondents preferred curing NC overlays with water as opposed to 23% curing NC overlays with wet burlap. For HPC overlays the results were identical, 12% for both curing groups.

Length of Curing

The length of curing varied between 5-7 days for both NC and HPC overlays using both of the curing groups.

7.8 Final Remarks

Today's available financial resources are not able to keep pace with the growing demand for maintenance and rehabilitation of deteriorating bridges. It is very important to make the best possible use of these limited financial resources. Bridge Rehabilitation Engineers are very important decision makers because they carefully evaluate expected repair cost and recommend the best possible benefit/cost rehabilitation techniques. When all of these different aspects are combined and considered for a single project it is referred to as a Bridge Management System.

The database produced in this thesis is essential in developing an effective and economical Bridge Management System in terms of maintenance cost as well as service life. In order to effectively carry out expeditious and effective rehabilitation techniques described in this thesis, a lot of project and time management as well as cost analysis are involved to ensure that projects are done on schedule otherwise the impacts on traffic, the environment and on the project's budget would be very high.

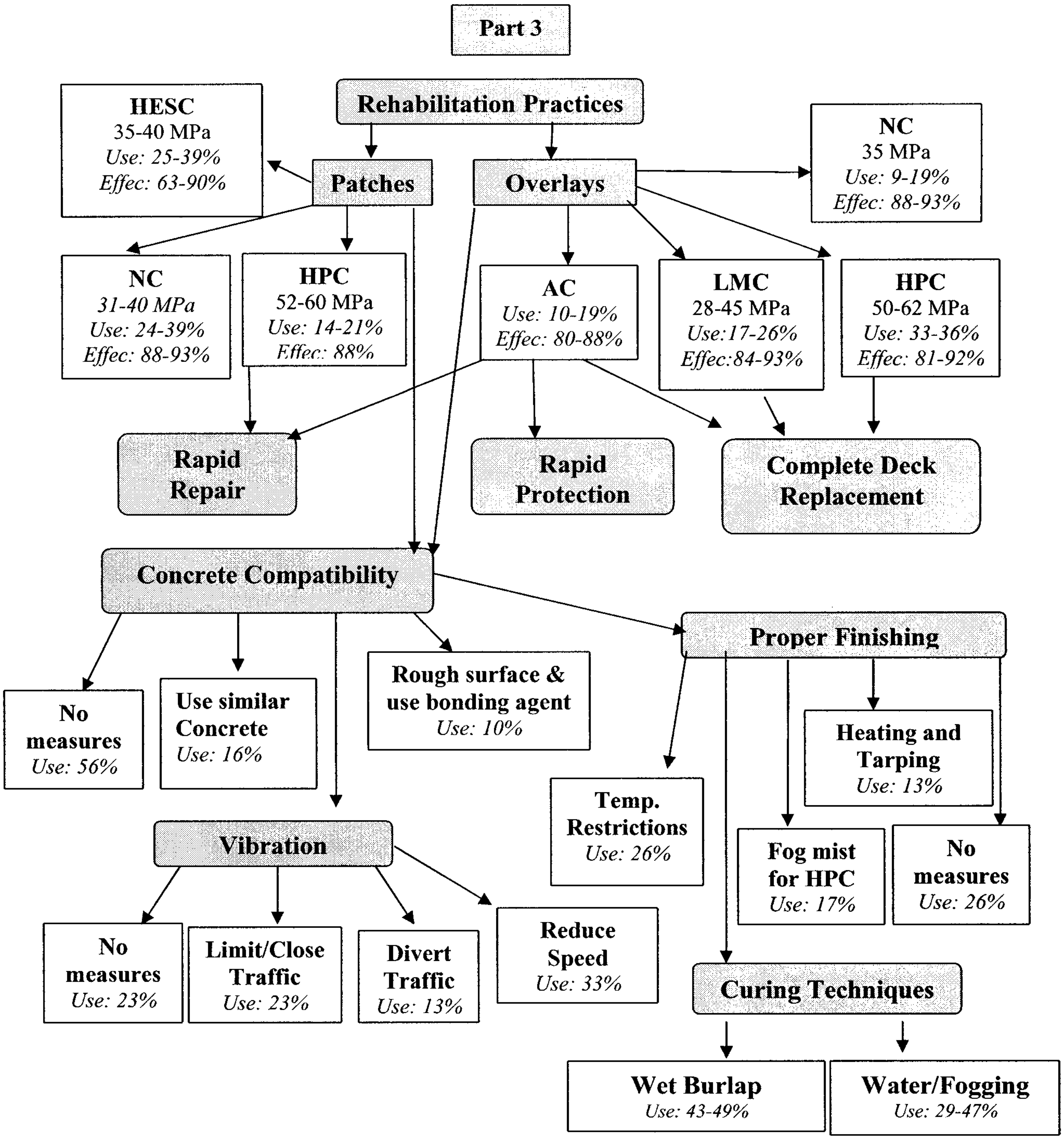


Figure 7.4 Rehabilitation Practices and Methods

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APPENDIX A

SURVEY RESULTS

Question 1

Canada

Techniques Used in Condition Surveys	Yes	No	% Range of Effectiveness	% Mean Effectiveness
Corrosion Rate Assessment	2	14	70	70
Corrosion Potential Surveys	14	2	50-80	68.91
Radar Surveys (DART and GPR)	6	10	50	50
Visual Surveys	5			
Delamination Surveys	7			
Chloride Analysis	5			
Concrete Cores/Sawn Samples	4			
Other**	5			
Thermography (infrared and imaging)*	1			
Permeability*	1			
Asphalt Removal*	2			
Concrete Cover Surveys*	1			

** Other is the sum of the all of the *

United States

Techniques Used in Condition Surveys	Yes	No	% Range of Effectiveness	% Mean Effectiveness
Corrosion Rate Assessment	0	14		
Corrosion Potential Surveys	7	7	50-80	67.5
Radar Surveys (DART and GPR)	6	8	10-50	28.3
Visual Surveys	5			
Delamination Surveys	9			
Chloride Analysis	8			
Concrete Cores/Sawn Samples	4			
Other**	3			
Thermography (infrared and imaging)*	2			
Permeability*	0			
Asphalt Removal*	0			
Concrete Cover Surveys*	1			

** Other is the sum of the all of the *

Question 2

(a) Combined List (Canada and United States)

Rehabilitation Cycle for New Structures	Respondents
< 25 years	4
25 - 30 years	2
Varying time frame	17
No Response	6
No Monitoring	1

Rehabilitation Cycle for Old Structures	Respondents
1 - 2 years	8
2 - 4 years	5
Varying time frame	14
No Response	3

(b) Combined List (Canada and United States)

Actual Rehabilitation Process	Respondents
1 - 2 years After Survey	11
2 - 4 years After Survey	11
Varying time frame	3
No Response	5

(c) Combined List (Canada and United States)

Structure Monitoring	Respondents
1 - 2 years After Survey	7
2 - 4 years After Survey	2
Varying time frame	12
No Monitoring	7
No Response	2

Question 3

Canada

Removal Methods/Techniques	Yes	No	% Range of Effectiveness	% Mean Effectiveness	% Range of Usage	% Mean Usage
Jack Hammers	14	2	85-100	95.63	60-100	87.83
Machine Mounted Dem. Attach.	12	4	90-100	95.86	4-100	27.71
Hydrodemolition	11	5	75-100	92.50	1-10	5.44
Splitting	1	15				
Thermal Demolition		16				
Chipping Hammers	2					
Saw Cutting	1				15	15.00
Grinding/Milling	2					

United States

Removal Methods/Techniques	Yes	No	% Range of Effectiveness	% Mean Effectiveness	% Range of Usage	% Mean Usage
Jack Hammers	14		75-100	87.18	25-100	63.25
Machine Mounted Dem. Attach.	10	4	75-100	82.86	7-95	49.63
Hydrodemolition	12	2	50-100	86.11	1-80	25.40
Splitting	1	13				
Thermal Demolition		14				
Grinding/Milling	1					

Combined List (Canada and United States)

Removal Methods/Techniques	Yes	No
Jack Hammers	28	2
Machine Mounted Dem. Attach.	22	4
Hydrodemolition	23	5
Splitting	2	15
Thermal Demolition		16
Other**	6	
Chipping Hammers*	2	
Saw Cutting*	1	
Grinding/Milling*	3	

** Other is the sum of the all of the *

Question 4

(a) Combined List (Canada and United States)

Removal Zone	Respondents
Delamination Only	23
Delamination & High Corrosion Potential	21
Removal of a uniform thickness over the entire deck	19
No Removal Criteria	2

(b)

Selecting Interventions	Respondents
Short and Frequent Interventions	7
Long Lasting and Less frequent Interventions	20
No Response	3

Question 5

Combined List (Canada and United States)

Measures/Techniques to Limit Crack Propagation	Concrete Removal
Chipping Hammers	11
Jack Hammers (small, <14kg)	9
Traffic Control	3
Sawcutting	9
Other**	6
No Measures	7
Reduce Speed of Traffic*	1
Remove to Sound Concrete*	1
Curing of Concrete*	1
Using repair materials similar to deck material*	1
Avoid Vibrating the Rebars*	1
Operator Ability Monitored*	1

** Other is the sum of the all of the *

Question 6

Combined List (Canada and United States)

Surface Preparation Techniques	Total
Clean SSD Conditions	3
Roughen Surface	4
Prewetting & Sand/Cement Paste	10
Slurry/Latex Material	11
Sandblasting/Airblasting	25
Waterblasting	5
No Measures	4

Question 7

Canada

Concrete Replacement	Patches	Range of Comp. Strength (MPa)	Mean Comp. Strength (MPa)	Range of Effect. (%)	Mean Effect. (%)	Range of Usage (%)	Mean Usage (%)
Normal Concrete	11	25-40	32.05	70-100	79.17	15-90	57
High Performance Concrete	4	25-50	37.50			5-75	28
High Early Strength Conc.	7	20-40	31.25	50-70	63.33	1-60	19
Asphalt Concrete	2			80	80.00	95	95
Other**	2						
No Response*	2						
Proprietary Patching Products*	2	30	30.00	50	50.00	10	10
Epoxies*	0						

** Other is the sum of the all of the *

Concrete Replacement	Overlays	Range of Comp. Strength (MPa)	Mean Comp. Strength (MPa)	Range of Effect. (%)	Mean Effect. (%)	Range of Usage (%)	Mean Usage (%)
Normal Concrete	6	25-40	31.67	70-100	87.50	15-90	66
High Performance Concrete	11	25-50	40.83	70-90	81.25	1-75	34
High Early Strength Conc.	2	25-45	35.00	75	75.00	10-95	53
Latex Modified Concrete	8	30-45	36.67	80-90	84.00	2-85	19
Asphalt Concrete	3			80	80.00	5-95	65
No Response	1						

United States

Concrete Replacement	Patches	Range of Comp. Strength (MPa)	Mean Comp. Strength (MPa)	Range of Effect. (%)	Mean Effect. (%)	Range of Usage (%)	Mean Usage (%)
Normal Concrete	8	21-31	27.50	70-100	89.00	5-70	19
High Performance Concrete	7	28-62	36.20	75-100	88.33	5-95	65
High Early Strength Conc.	13	28-35	29.33	70-100	90.00	5-90	34
Asphalt Concrete	4			70-90	80.00	10-50	28
Other**	1						
Proprietary Patching Products*	0						
Epoxies*	1			90	90.00	1	1.00

** Other is the sum of the all of the *

Concrete Replacement	Overlays	Range of Comp. Strength (MPa)	Mean Comp. Strength (MPa)	Range of Effect. (%)	Mean Effect. (%)	Range of Usage (%)	Mean Usage (%)
Normal Concrete	3	21-31	29.00	90-95	92.50	5-90	48
High Performance Concrete	12	21-62	33.17	75-100	91.88	1-100	51
High Early Strength Concrete	4	28-30	29.00	70-90	80.00	10-30	20
Latex Modified Concrete	6	24-28	27.00	90-100	93.33	5-80	35
Asphalt Concrete	7			70-100	87.50	1-96	34
Other**	4						
Low Slump Concrete*	2	31-35	33.00	90	90.00	90-5	93
Epoxies*	1			90	90.00	1	1
Polymers*	1	35	35.00	80	80.00	4	4
No Response*	0						

** Other is the sum of the all of the *

Question 8

Canada

Interventions	Option 1	Thickness (mm)	Average Thickness (mm)	Expected Life Span (years)	Average Life Span (years)
Asphalt overlays with membranes	3	90,80	90.00	12,25	18.50
Asphalt overlays without membranes	2	50,80	65.00	<5	5.00
High early strength concrete overlays	1			12	12.00
High performance concrete overlays	1			12	12.00
Patching with High early strength con.	1	30-60	45.00		
Patching with asphalt concrete	1	150-175	0.00		
Polymer overlays with waterproofing					
Polymer overlays without waterproofing					
Fiber reinforced concrete					
Other**	5				
Low Slump Concrete Overlay*	0				
No Response*	5				

** Other is the sum of the all of the *

Interventions	Option 2	Thickness (mm)	Average Thickness (mm)	Expected Life Span (years)	Average Life Span (years)
Asphalt overlays with membranes	2	80	80.00	25	25.00
Asphalt overlays without membranes	1	80	80.00		
High early strength concrete overlays	2	60	60.00	25,15-20	21.25
High performance concrete overlays	1			25	25.00
Patching with High early strength con.	5	30-80	50.00	15,15,20-25	17.50
Patching with asphalt concrete	1	70	70.00	<10	10.00
Polymer overlays with waterproofing					
Polymer overlays without waterproofing					
Fiber reinforced concrete	1	60	60.00	25-30	27.50
Other**	3				
No Response*	3				

** Other is the sum of the all of the *

Interventions	Option 3	Thickness (mm)	Average Thickness (mm)	Expected Life Span (years)	Average Life Span (years)
Asphalt overlays with membranes	7	50-90	78.86	15-40	25.00
Asphalt overlays without membranes					
High early strength concrete overlays	3	40-150	82.50	20-40	30.00
High performance concrete overlays	7	40-150	65.83	20-40	28.33
Patching with High early strength con.	2				
Patching with asphalt concrete					
Polymer overlays with waterproofing	4	50-100	63.75	25-30	28.75
Polymer overlays without waterproofing	3	50-100	68.33	20-25	23.75
Fiber reinforced concrete	2	100	100.00		
Other**	1				
Gemcrete*	1	12-18	15.00	25	25.00
No Response*	3				

** Other is the sum of the all of the *

United States

Interventions	Option 1	Thickness (mm)	Average Thickness (mm)	Expected Life Span (years)	Average Life Span (years)
Asphalt overlays with membranes	3	50-75	57.50	5-15	10.00
Asphalt overlays without membranes	5	25-75	51.00	2-15	7.80
High early strength concrete overlays	1	50	50.00	12-15	13.50
High performance concrete overlays	1	50-75	62.50		
Patching with High early strength con.	4	50-75	62.50	5-20	15.00
Patching with asphalt concrete	3	40-90	63.33	1-5	2.00
Polymer overlays with waterproofing	1	50	50.00	10-15	12.50
Polymer overlays without waterproofing	2	12-35	23.50	5-20	13.75
Fiber reinforced concrete					
Other**	3				
Low Slump Concrete Overlay*	1	50	50.00	20-25	22.50
No Response*	2				

** Other is the sum of the all of the *

Interventions	Option 2	Thickness (mm)	Average Thickness (mm)	Expected Life Span (years)	Average Life Span (years)
Asphalt overlays with membranes	2	50-75	56.25	5	5.00
Asphalt overlays without membranes	7	25-80	50.36	2-30	10.00
High early strength concrete overlays	2	75,50	62.50	12-20	16.75
High performance concrete overlays	1	50-75	62.50		
Patching with High early strength con.	14	50-110	80.00	1-25	11.12
Patching with asphalt concrete	8	25-75	46.67	1-10	5.07
Polymer overlays with waterproofing					
Polymer overlays without waterproofing					
Fiber reinforced concrete					
Other**	2				
Low Slump Concrete Overlay*	1	50	50.00	20-25	22.50
Epoxy Patches*	1	60-90	75.00	2-3	2.50
No Response*	0				

** Other is the sum of the all of the *

Interventions	Option 3	Thickness (mm)	Average Thickness (mm)	Expected Life Span (years)	Average Life Span (years)
Asphalt overlays with membranes	12	25-80	72.00	8-40	20.36
Asphalt overlays without membranes	1	50	50.00	25	25.00
High early strength concrete overlays	4	18-50	29.00	12-25	17.75
High performance concrete overlays	13	18-140	57.00	15-40	25.42
Patching with High early strength con.					
Patching with asphalt concrete					
Polymer overlays with waterproofing	3	12-50	34.00	10-25	15.83
Polymer overlays without waterproofing	2	5-75	40.00	15-25	20.00
Fiber reinforced concrete	2	50-75	56.25	25-35	30.00
Other**	2				
Gemcrete*	0				
Low Slump Concrete Overlay*	1	50	50.00	20-25	22.50
Normal Concrete*	1				
No Response*	0				

** Other is the sum of the all of the *

Question 9

Combined List (Canada and United States)

Compatibility Measures	Respondents
Use of embedded Galvanic Anodes	3
Use concrete with similar characteristics	5
No Measures	18
Other**	6
Check level of chloride*	1
Rough Surface and use Bonding agent*	3
Use of Lithium to mitigate ASR*	1
Corrosion Inhibitors*	1

** Other is the sum of the all of the *

Vibration Measures	Respondents
Reduced speed at adjacent lanes	13
Limit/close traffic	9
Divert traffic	5
No Measures	4
Other**	9
No Overlays on Lively Decks (Steel Girders)*	1
Pour during off peak hours (nights/weekends)*	1
Disconnect diaphragms between girders*	2

** Other is the sum of the all of the *

Question 10

Combined List (Canada and United States)

Ensuring Proper Finishing/Low Permeability	Respondents
Heating and Tarping to ensure proper curing	6
Permeability Test	3
Temperature restrictions when placing fresh concrete	12
Fog mist in wet conditions for HPC and HPC overlays	8
No Measures	12
Other**	5
Spec a Rapid Chloride Permeability Value *	1
Retarders in concrete for workability*	1
Trial Placements*	1
Machine Finishing*	2

** Other is the sum of the all of the *

Question 11

Canada

Intervention/Materials Used	Curing Techniques	Respondents	Length of Curing	Avg. Length of Curing (d)	Range of Effect. (%)	Mean Effect. (%)
NC Overlays	Wet Burlap/Cotton Mats	5	2-4	3.6	75-100	85.50
NC Overlays	Water/Fogging/Misting	3	7	7.0	100	100.00
HPC Overlays	Wet Burlap/Cotton Mats	3	7	7.0	80-90	85.00
HPC Overlays	Water/Fogging/Misting	4	7	7.0	90	90.00
LMC Overlays	Wet Burlap/Cotton Mats	1	1	1.0	85	85.00
LMC Overlays	Water/Fogging/Misting	0				
Patching with NC	Wet Burlap/Cotton Mats	1	4	4.0	80	80.00
Patching with NC	Forms in Place	1	4	4.0		
Patching with HPC	Forms in Place	1	7	7.0		
Waterproofing System	Air Curing	1	3	3.0	90-95	92.50
OPSS 930		1				
No Response		2				

United States

Intervention/Materials Used	Curing Techniques	Respondents	Length of Curing	Avg. Length of Curing (d)	Range of Effect. (%)	Mean Effect. (%)
NC Overlays	Wet Burlap/Cotton Mats	7	2-7	5.7	50-95	82.00
NC Overlays	Water/Fogging/Misting	11	0.25-7	4.3	20-100	69.44
HPC Overlays	Wet Burlap/Cotton Mats	3	3-7	4.3	60-100	80.00
HPC Overlays	Water/Fogging/Misting	2	4-7	6.3	70-95	82.50
LMC Overlays	Wet Burlap/Cotton Mats	3	3-7	4.7	75-100	88.33
LMC Overlays	Water/Fogging/Misting	1	4-7	5.5	70	70.00
LSC Overlays	Wet Burlap/Plastic Sheets	1	4	4.0	90	90.00
TPC Overlays		1	0.33	0.3	95	95.00
No Response		1				

LSC – low slump concrete, TPC – thin polymer concrete

Question 12

This question is included under Chapter 6 of this thesis.

Question 13

Canada

Properties	Yes	No	Uncertain
Measurements of permeability of chloride ions	11	2	
Corrosion of reinforcing bars (> 0.35 mV, %HPC, etc.)	10	3	
Skid Resistance	1	10	2
Wear	7	5	1
Bond Strength	4	8	1
Percentage of Delamination	10	1	2
Corrosion rate measurements	1		
Air Content from Cores	2		
Alkali Aggregate Reactivity from Cores	1		
Functional Deficiency	2		
Cost required to maintain service on the bridge			
No Properties	3		

(a)

Budget Estimate (CDN Millions)	Respondents
1-10 (M)	8
10-30 (M)	1
30-100 (M)	2
>100 (M)	1
Not Applicable	4

(b)

Number of Bridges	Respondents
0	2
1-100	4
100-1000	4
1000-3000	4
>3000	2

(c)

Intervention Cost ($CDN/sqft)	Respondents			
	Patching	Deck Overlay	Deck Replacement	Bridge Replacement
25-50	2	1	1	
100-500	5	2	3	1
500-1000			3	2
1000-3000			1	5
Not Available	9	13	8	8

United States

Properties	Yes	No	Uncertain
	7	4	
Measurements of permeability of chloride ions	6	4	1
Corrosion of reinforcing bars (> 0.35 mV, %HPC, etc.)	2	8	1
Skid Resistance	4	6	1
Wear	3	7	1
Bond Strength	10	1	
Percentage of Delamination	1		
Cost required to maintain service on the bridge	3		
No Properties	7	4	

(a)

Budget Estimate (CDN Millions)	Respondents
1-10 (M)	1
10-30 (M)	5
30-100 (M)	3
100-600 (M)	2
Not Applicable	3

(b)

Number of Bridges	Respondents
0	1
1000-3000	4
>3000	9

(c)

Intervention Cost ($CDN/sqft)	Respondents			
	Patching	Deck Overlay	Deck Replacement	Bridge Replacement
0-25	5	2	1	
25-50	1	2	5	2
50-100			2	1
100-500				2
Not Available	8	10	6	8

APPENDIX B

LIST OF RESPONDENTS

List of Respondents from Canada

Organization/Agency	Contact Personnel		
	Name	Title	Telephone
Morrison Hershfield	Edward Li	Bridge Engineer	
URS Canada Inc.	Ranjit Reel	Chief Bridge Engineer	(905) 882-4401
Seaway International Bridge Corp.	Thye Lee	Bridge Engineer	
McCormick Rankin Corp.	Reno Radolli	Manager	
City of Toronto	John Bryson	Manager	(416) 392-9183
City of Ottawa	John Eccles	Program Manager-Structural Rehabilitation	
City of Winnipeg	Brad Neirinck	Bridge Planning & Operations Engineer	(204) 986-7950
City of Saskatoon	Mark Patola	Bridge Engineer	(306) 975 2871
City of Vancouver	Douglas Smith	Bridge Engineer	(604) 873-7320
New Brunswick Department of Transportation	Bruce Connolly	Assistant Director	(506) 453-2673
Nova Scotia Department of Transportation	Gary Pyke	Bridge Engineer	(902) 424-4268
Newfoundland Department of Transportation	Raymond Matthews	Bridge Engineer	(709) 729-3636
PEI Department of Transportation	Darrell Evans	Manager, Design & Bridge Maintenance	(902) 569-0578
British Columbia Ministry of Transportation	Gary Farnden	Senior Rehabilitation Engineer	(250) 387-7728
Ontario Ministry of Transportation	David Lai	Head Rehabilitation Engineer	(905) 704-2347
Ministry of Transportation Quebec	Daniel Vezina	Bridge Engineer	

List of Respondents from USA

Organization/Agency	Contact Personnel		
	Name	Title	Telephone
Colorado Department of Transportation	Mark Leonard	Bridge Engineer	(303) 757-9309
Illinois Department of Transportation	Vic Modeer	Director	(217) 782-2151
Kansas Department of Transportation	Dan Schlerschlight	Bridge Engineer	
Maine Department of Transportation	Allan Haggan	Bridge Maintenance Engineer	
Minnesota Department of Transporation	Gary Peterson	Bridge Engineer	
Montana Department of Transportation	Nigel Mends	Bridge Engineer	
New Hampshire Department of Transportation	Mark Whittemore	Bridge Design Engineer	
New Jersey Department of Transportation	Harry Capers	Bridge Design Engineer	(609) 530-2557
New York State Department of Transportation	Don Streeter	Bridge Engineer	(518) 457-5956
South Dakota Department of Transportation	Tom Gilsrud	Bridge Maintenance Engineer	(605) 773-3285
Pennsylvannia Department of Transportation	Bryan Spangler	Bridge Engineer	
Rhode Island Department of Transportation	Michael Savella	Bridge Engineer	
Virginia Department of Transportation	Fred Dotson	Bridge Engineer	
Wisconsin Department of Transportation	Bruce Karow	Bridge Engineer	