RESEARCH REPORT

Cost-Effective Concrete Repair: Research, Investigation, Analysis, and Implementation

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COST-EFFECTIVE CONCRETE REPAIR

Research,
Investigation,
Analysis, and
Implementation

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Canada Mortgage and Housing Corporation, the Federal Government's housing agency, is responsible for administering the National Housing Act. This legislation is designed to aid in the improvement of housing and living conditions in Canada. As a result, the Corporation has interests in all aspects of housing and urban growth and development.

Under Part IX of this Act, the Government of Canada provides funds to CMHC to conduct research into the social, economic and technical aspects of housing and related fields, and to undertake the publishing and distribution of the results of this research. CMHC therefore has a statutory responsibility to make widely available, information which may be useful in the improvement of housing and living conditions.

This publication is one of the many items of information published by CMHC with the assistance of federal funds. The analysis, interpretations and recommendations are those of the Consultant and do not necessarily reflect the views of Canada Mortgage and Housing Corporation that assisted in the study and its publication.

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Their commentary and editorial assistance shaped this document so that it could truly and confidently be used by owners and managers of buildings.

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FOREWORD

This report is intended for use by building owners and managers in their efforts to establish a rational course of action for repair of deteriorated concrete - action that will develop a cost-effective repair strategy for the life of the building, for them, within their particular sets of income constraints and funding opportunities. The report, *Cost-Effective Concrete Repair - Research, Investigation, Analysis and Implementation*, reviews the characteristics needed to produce durable concrete, the factors that cause deterioration, common and newer test methods to evaluate the state of the concrete, and common repair techniques and cost issues associated with concrete repair practices. It briefly summarizes the history of concrete research, related to its deterioration and repair, and the development of design and construction standards as these have changed to produce more durable concrete.

One of the goals of this report is to reduce the vast amount of information already available on concrete durability and repair, to a readily usable summary that owners can reference when confronted with poorly performing concrete. There are a variety of ways that concrete can deteriorate and a variety of contributing causes. There are also a variety of ways of dealing with any one type of deterioration, the correct selection of which is as much a financial/operational issue as a technical performance issue. In this regard, the technical reasons and means for repairing concrete have been the topic of considerable research over the past 30 years or more. Technical advances in materials, demolition, repair and protection techniques have far outpaced the question of life-cycle costing and cost-benefit assessment as related to the same repair. This report examines the issue of concrete repair from the perspective of the owner. It examines the possibility of different repair strategies to suit the needs of different owners and provides examples using repair cost data and analytical costing tools that owners can use to make economic assessments. The report also provides a road map that owners can follow as they begin to tackle repair of their concrete.

Canada Mortgage and Housing Corporation commissioned this report to foster the integration of the technical and economic aspects of concrete repair to assist building owners, managers and their professional consultants in understanding the each other's concerns. The report is organized into four chapters and nine appendices. The four chapters provide information about concrete durability and the action needed to assess the extent, nature and appropriate response by property owners, managers and their consultants. The nine appendices provide more comprehensive information about concrete materials and construction, the primary mechanisms of concrete deterioration and new protection methods, life-cycle costing and costs of repair. The appendices should be useful background for property owners and managers when they discuss repair options with their consultants. The report is organized in such a way that the reader can gain a general appreciation of the issues by reading the four chapters and, where more information is desired, the reader can refer to the appendices and to the list of references and other reading noted at the end of the report.

It is hoped that this review of the various issues and the synthesis of the research given in this report will help owners and managers of buildings better understand the nature and extent of the causes of distress and the options open to them for repair, enabling them to make better decisions regarding their repair work. It is also hoped that researchers, designers and the consulting community that have come to understand why concrete fails under certain conditions will become more sensitized to the non-technical aspects of the repair and, therefore, be better prepared to help property owners and managers to assess the physical conditions with due consideration of the financial constraints involved in making the deteriorated concrete serviceable again.

AVANT-PROPOS

Le présent rapport s'adresse aux propriétaires et administrateurs d'immeubles qui sont appelés à établir un plan d'action pour la réparation de béton détérioré, plan qui aboutira à une stratégie de réparation économique pour le cycle de vie du bâtiment, compte tenu de leurs contraintes financières et des possibilités de financement qui s'offrent à eux. Le présent rapport, intitulé Réparation du béton dans une optique rentable - Recherches, enquêtes, analyse et mise en oeuvre, passe en revue les caractéristiques propres à la production de béton durable, les facteurs propices à la détérioration, les méthodes d'essai, courantes et nouvelles, d'évaluation de l'état du béton, ainsi que les techniques de réparation communes et les aspects financiers liés aux pratiques de réparation du béton. Il récapitule brièvement la genèse des recherches sur le béton, en ce qui a trait à la détérioration et la réparation, de même que l'évolution des normes de conception et de construction qui ont été établies pour en arriver à un matériau plus durable.

Le rapport a pour objet, entre autres, de réunir la myriade d'informations qui existent déjà sur la durabilité et la réparation du béton dans un compendium de consultation facile pour les situations où la performance du béton laisse à désirer. Il y a bien des façons dont le béton peut se détériorer, et bien des causes également. Il y a aussi bien des manières de faire face au problème, la solution retenue étant aussi bien fonction des aspects financiers et opérationnels que techniques. En l'occurrence, les motifs et techniques de réparation du béton ont fait l'objet d'un grand nombre de recherches depuis une trentaine d'années. Les progrès réalisés relativement aux matériaux, à la démolition, à la réparation et aux techniques de protection ont été beaucoup plus rapides que les analyses de coûts du cycle de vie et les évaluations du rapport qualité-prix dans le contexte des mêmes travaux de réparation. Le rapport a été écrit selon le point de vue du propriétaire confronté à des travaux de réparation de béton. Il examine différentes méthodes de réparation à la lumière des besoins de propriétaires différents et fournit des exemples de la manière dont le propriétaire peut se servir des données sur le coût des réparations et d'instruments d'analyse des coûts pour faire une évaluation économique.

La Société canadienne d'hypothèques et de logement a commandé le présent rapport pour favoriser l'intégration des aspects techniques et économiques de la réparation du béton, afin d'aider les propriétaires et administrateurs d'immeubles et leurs experts-conseils à comprendre leurs préoccupations respectives. Le rapport compte quatre chapitres et neuf annexes. Dans les quatre chapitres on trouvera des renseignements sur la durabilité du béton et les mesures à prendre pour évaluer l'ampleur, la nature et l'à-propos des interventions du propriétaire, de l'administrateur et de l'expert-conseil. Les neuf annexes fournissent plus de détails sur les matériaux et constructions en béton, les principaux facteurs favorisant la détérioration du béton, ainsi que les nouvelles méthodes de protection, le coût du cycle de vie et les frais de réparation. Ces renseignements de base devraient se révéler utiles aux propriétaires et administrateurs d'immeubles lorsqu'ils discuteront des diverses méthodes de réparation avec l'expert-conseil. Le rapport est structuré de manière à ce que le lecteur puisse acquérir des connaissances générales à la lecture des quatre chapitres et, au besoin, d'approfondir certains points en consultant les annexes, la bibliographique et les autres lectures suggérées à la fin du document.

L'examen des divers points et la synthèse des recherches que renferme le présent rapport devraient aider les propriétaires et administrateurs d'immeubles à mieux comprendre la nature et la portée des causes de dégradation, et les diverses techniques de réparation, pour ainsi leur permettre de prendre une décision plus éclairée. Nous comptons aussi que les chercheurs, concepteurs et experts-conseils qui ont compris pourquoi le béton fait défaut sous certaines conditions seront sensibilisés aux aspects non techniques de la réparation et donc mieux en mesure de seconder les propriétaires et administrateurs d'immeubles à évaluer l'état matériel en tenant dûment compte des contraintes financières qui sont rattachées à la remise en état du béton.

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CHAPTER 1 CONCRETE DETERIORATION

HOW FAULTS OCCUR IN CONCRETE

Concrete is basically a mixture of two components: aggregates and paste. The paste, usually comprised of Portland cement and water, binds the aggregates, usually sand and gravel or crushed stone, into a rocklike mass as the paste hardens. The hardening process is the result of a chemical reaction between the cement and water. Supplementary cementing materials may also be included in the paste. Aggregate occupies roughly three-quarters of the volume of concrete, the remaining volume being cement paste (cement and water) and air voids. The properties of the concrete can be modified by changing the properties and proportions of the major constituents, cement, water and aggregate, and by the addition of other materials to the cementitious matrix. Concrete can withstand substantial forces in compression but it is a brittle material with little strength in tension, requiring the addition of reinforcing materials to allow it to be used in structural members.

The designs of concrete mixes and the designs of structures including concrete and reinforcing steel have evolved considerably in the last 100 years. Research has sought ways to make reinforced concrete a stronger more efficient product to use and a more durable product for the long term. For instance, by adding special chemical admixtures, less water and, thus, less cement is needed reducing the cost; by improving on the design techniques, thinner, stronger structures have been possible; and by the introduction of supplementary cementing materials such as flyash, slag cement and micro silica, other desirable properties can be enhanced. Each of these changes has made reinforced concrete a better material. Yet, despite the efforts by designers and materials suppliers to improve the performance of reinforced concrete, the actual concrete that forms much of our buildings is affected by varying degrees of quality control on the construction site. The resulting concrete will, thus, resist deterioration to varying degrees.

When used above-grade as the structural frame of a building, in a climate controlled and dry environment, concrete failure is very rare. Concrete that is not exposed, such as the interior structural columns, beams, walls and floors that make up the frame of a building, can be expected to last at least the design life of the building with essentially no remedial work. However, the experience of building owners with structural concrete that is used below grade has not been as good. For example, concrete used for parking floors in multi-level garages has suffered from chloride-ion incursions and corrosion-induced delamination and spalling; concrete used in some exterior wall components above grade, such as precast concrete wall panels or as exposed flat slabs above grade such as balcony floors or the edges of structural floor slabs, have deteriorated due to moisture, carbonation of the concrete and freeze-thaw reactions. This experience shows that the incidence of the problems tends to depend on environmental factors such as moisture, chloride and low temperature. Owners have learned that concrete exposed to these elements will require periodic maintenance to repair deterioration, to restore its appearance, and to retain desirable properties such as strength and watertightness. Accordingly, for reasons of construction quality and exposure, some parts of structures that were previously in good condition, now exhibit deterioration and now present building owners with substantial costs for maintenance and repair.

For property owners to begin to understand the reasons why concrete deteriorates and, therefore, properly assess what should be done about its cost effective restoration, each deterioration situation must be put into its correct perspective.

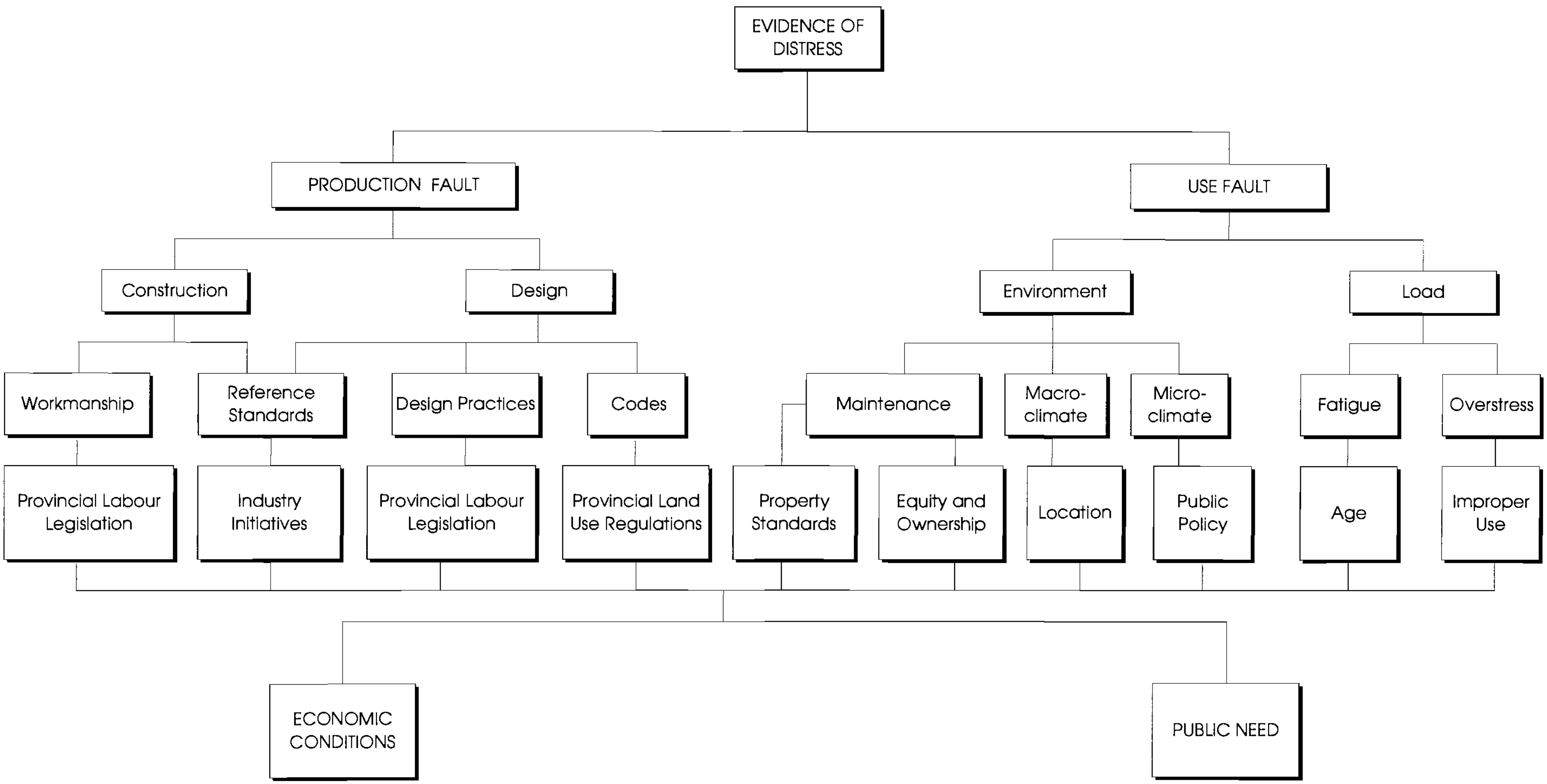

Figure. 1-1: Factors Influencing Building Condition

Figure 1-1 is provided to illustrate many of the factors which culminate in evidence of distress. Since concrete is designed and constructed to have various levels of resistance to deterioration it is able to withstand damage or decay in environments of various degrees of severity. If deterioration has occurred, the cause lies either in the way the concrete was designed and constructed, a *Production Fault* or in the way it has been put to use, a *Use Fault*, or some combination of the two. When a durability problem arises, it is a consequence of a mismatch between the issues of production and use.

As Figure 1-1 shows, within the two primary categories of *Production* and *Use* there are four broad subcategories that could lead to concrete distress; these are, *Construction*, *Design*, *Environment* and *Load*. Understanding these four technical issues is critical in any property maintenance discussion. These issues also tend to be central to technical arguments concerned with legal action involving concrete deterioration.

Each of the four subcategories noted above is dependent on certain influences. For instance, on the *Production* side of the chart, construction quality is determined by *Workmanship* and by the *Reference Standards*, the design of concrete is controlled by *Design Codes* and, constituent materials are, as well, controlled by *Reference Standards.* These factors, which determine the level of resistance to deterioration that concrete has, at the time it is produced, are examined in this report. Sufficient detail is provided so that building owners may have a good grasp of what is required to make concrete durable and thereby provide some guidance as to the properties that should be restored during repair.

Once the building is occupied, factors on the *Use* side of the chart also begin to effect durability. For instance, the amount and type of *Maintenance* during the life of the building is established by each owner's operating and capital expense allocation. Owners also determine the *Load* applied to the structure. The *Micro-climate*, i.e., moisture and salt application in freeze-thaw conditions, which may cause the concrete to deteriorate, is less controlled by the owner but, nonetheless, is a factor that is within the *Use* category.

Each of the controlling *Production* and *Use* factors exists within a régime of some other influences. For example, *Land Use Regulations* permit the Codes to dictate the minimum requirements of buildings; *Property Values* influence the degree and type of maintenance and *Public Policy* determines the use of deicing chemical on most roads in Canada. The two prime motivators driving all of the above factors, *Economic Conditions* and the general *Public Need* are identified as the basis for Figure 1-1.

To properly repair concrete it is necessary that the proper fault be addressed. Even so, the repairs that owners of buildings undertake usually only partially address the fault because, once the concrete is produced, only the *Use* side of the "fault tree" (Figure. 1-1) can be changed. Repairs, therefore tend to be required on a recurring basis.

Subsequent resolution of the faults must address both *Production* and *Use*. This generally results in changes to design and construction standards, changes to the building codes and education of the owners of the buildings in the proper care and maintenance of their properties. The first step leading to effective change, is research into the nature and causes of the distress and the effectiveness of the repairs.

Research Into Concrete Durability and Repair

The need to understand what causes concrete to deteriorate, and to determine the action needed to rectify the problems have been the impetus for extensive research over the past 30 years in Canada. Concrete deterioration has caused many owners of concrete structures to spend significant funds on repair of the structure when expenditures of a lesser magnitude were expected over the life of the structure. This chapter, thus, examines the primary deterioration processes that have become the focus of much of the research to date.

Much of the early research in the 1960s was initiated by the industry supplying the cement product. Accordingly, many of the older references are from the dominant industry group, the Portland Cement Association. Later, in the 1970s, the major users of concrete became involved with research into causes of distress. These were, principally, the highway and bridge authorities in Canada and the United States of America who began to view distress of the concrete in their structures as significant, both as short and long term problems. A good deal of effort was spent by highway and bridge authorities in Canada and the United States in developing potential protective measures for new structures in addition to assessment of the causes of the existing problems. Much of the knowledge so gained was readily adaptable to buildings, particularly garage slabs.

In the early 1980s, Canada Mortgage and Housing Corporation, took an active interest in the condition and repair of concrete in publicly-funded residential buildings (high-rise buildings primarily). Their motivation was founded on an interest in sustaining the value of the mortgaged buildings through effective repair of the deteriorated concrete. That interest spawned some of the research currently available to building owners in Canada. Appendix A summarizes key contributions to concrete research developed by numerous industry groups, transportation authorities and other owners of concrete structures. It is presented to give building owners and managers an overview of the evolution of concrete research work, its general findings, the direction that concrete research appears to be headed and the significance of this research in cost-effective concrete restoration.

Primary Concrete Deterioration Processes

Visual evidence of deterioration is far more obvious than the processes that caused the deterioration. Some of these processes are quite complex and not fully understood even by the engineering and research community. However, in order to properly repair concrete, owners and their consultants must understand why the concrete failed in the first place. This section of the report outlines the various processes that cause concrete to deteriorate. To facilitate understanding of the processes by those not familiar with concrete deterioration, it is organized under headings that are the terms commonly used to describe the evidence that is seen. More information about the deterioration processes is given in the Appendices at the end of this report.

Deterioration of concrete in and around buildings takes several typical forms, the most prevalent being:

- Scaling
- Disintegration
- Cracks (narrow and wide)
- Leaching and Efflorescence
- Delamination and Spalling (salt-induced and carbonation-induced)

SCALING

Scaling is typified by deterioration and pitting of the surface of concrete, such as is often observed on outdoor concrete sidewalks. It also occurs on curbs and gutters, and floors subject to moisture and freezing and thawing. Scaling is also common on walls, columns and planters or feature walls near grade in similar exposed conditions. The severity of the scaling can be greatly exacerbated by the addition of deicing salt, either placed directly on the surface, transported by vehicular traffic or air borne through road spray. Figure 1-2 shows a typical example of scaling of concrete that was buried and thought by the designers to not be exposed to freeze-thaw conditions. In this case, the cause of the distress was a *Design Fault* (see Figure 1-1).

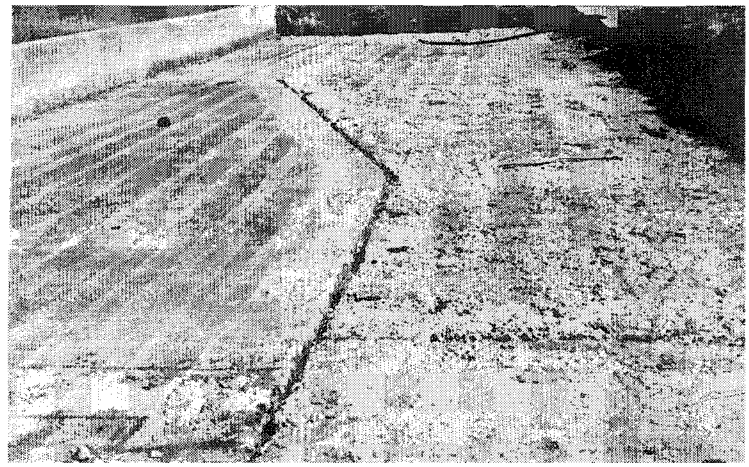

Figure 1-2: Scaled Concrete

Scaling appears as a surface deterioration of the cementitious matrix, sometimes exposing coarse aggregate. In the most severe cases, large aggregate may be dislodged and the surface will be very rough and weak. While freeze-thaw cycles are the primary inducement for scaling, wetting and drying cycles can cause the same form of deterioration, though this is far less commonly a problem.

Scaling occurs because concrete is interlaced with microscopic channels (capillaries) and air spaces (pore voids) which can hold water. The moisture in these channels and pores expands when freezing, causing pressure in excess of the tensile strength of the concrete. The concrete matrix begins to rupture and is eroded away at the exposed surface, giving the scaled appearance.

Concrete producers have learned how to entrain air voids into the concrete that are of the right size and spacing to release the pressure created by the freezing moisture. While these air voids are actually very small, in the microscopic scale in which scaling occurs, they act as large vessels to accept the water being displaced by the formation of ice (see *Appendix C Air-Entraining Admixtures* and *Appendix D Freeze-Thaw Damage and Scaling*). The entrained air voids are small enough and spaced close enough to ensure that they are in close proximity to sources of pressure buildup. Accidentally entrapped air and larger, more widely spaced voids are too few in number to offer protection and improve concrete durability.

In some situations, scaling may occur even if the concrete includes a suitable air-void system. These cases typically arise when the aggregate itself does not have adequate frost resistance. Scaling may also happen if the concrete contains excessive amounts of supplementary cementing materials such as blast furnace slag. However, this concern is generally found to be rare, relative to problems with lack of suitably entrained air.

Freeze-thaw resistance of any concrete can be adversely affected by the initiation of cracking due to some other process. Through research, we know these to be: alkali-aggregate reactions, sulphate attack or shrinkage *(see Appendix C Constituent Elements and Durability)*. Any increase in the moisture content of the hardened concrete will increase the potential for freeze-thaw damage, allowing larger volumes of water to permeate the concrete. In applications where concrete will be subjected to freeze-thaw cycles, air entrainment alone will not successfully prevent frost damage if the concrete becomes saturated due to cracking caused by other processes or, because of poor construction procedures.

The magnitude of the internal pressures due to ice formation is so great that the use of concrete of sufficient strength alone to resist these forces is impractical and uneconomical. Nonetheless, greater amounts of cement are used to achieve long term freeze-thaw resistance. While this additional cement does increase the strength of the concrete, the reason the additional cement is used is to decrease the permeability of the concrete by lowering the water-cement ratio. However, suitable air-entrainment is still needed to protect against freeze/thaw damage.

Repair of scaling requires removal of the damaged portion of the concrete and replacement with a suitably freeze-thaw resistant and air-entrained concrete. Various replacement materials are described in CHAPTER 4 REPAIR AND PROTECTION TECHNIQUES.

CHEMICAL DISINTEGRATION

Volume changes in hardened concrete can occur within the constituents of the concrete mix, causing cracking and its eventual physical disintegration. The two primary processes in this category are related to chemical reactions, one between the cementitious materials and the aggregates in the concrete, known as alkali-aggregate reactivity, and the other, a reaction between the cement and sulphates in the moisture of adjacent soil or groundwater.

Research has provided consultants with the means to determine if the potential exists for alkali-aggregate reaction. Now, standard physical and chemical tests are performed on the rock forming the aggregate to screen out alkali-reactive aggregates. Similarly, standard tests are performed on ground water to determine if sulphate attack on the hardened cement is likely. The results of alkali-aggregate reactivity, sulphate attack or volumetric changes due to improper cement are typically not problems that the average building owner or manager must face. Failure to select the correct materials initiates a *Design Fault* which now rarely occurs.

However, if reactive aggregates or sulphate attack do cause disruption of the concrete, it can be very serious. Usually, the failing concrete will have to be replaced with concrete that is not susceptible to this form of deterioration.

CRACKS

Cracks occur when stresses in the concrete exceed its tensile strength and are a consequence of all forms of deterioration. Some cracks occur independent of the other deterioration processes described in this chapter. These are induced by initial drying shrinkage of the concrete, response to changes in ambient temperature, and structural stresses due to excessive loads.

Initial drying shrinkage of the concrete increases microscopic cracking of freshly placed concrete. This cracking increases the permeability of the concrete which can lead to other problems that require moisture, such as corrosion and scaling. This is usually a *Construction Fault.*

Thermal movement in response to changes in the ambient air temperature results in significant expansion and contraction of large concrete elements such as long beams and slabs. The coefficient of thermal expansion of concrete ($10x10^{-6}$ mm/mm/deg. C) can result in roughly 10 mm of movement in a 30 m length of concrete exposed to a 30 deg. C temperature swing. If the ends of the concrete are restrained, these movements can be of sufficient magnitude to crack the concrete. Designers try to control the size of these cracks through the use of reinforcing steel and try to prevent random cracking through the use of expansion and contraction joints. Inserting crack control joints to predefine the location of any cracks that may develop is a standard design and construction practice. Cracks that occur due to inadequate reinforcement or absence of joints are likely a *Design Fault;* although, it is not uncommon that cracks occur anyway due to poor construction of the joints.

Crack control measures should also be taken into account any concrete repair efforts.

LEACHING AND EFFLORESCENCE

Leaching and efflorescence generally occurs at cracks and holes on highly permeable walls or slabs that are exposed to moisture on one side. This moisture dissolves free lime [$Ca(OH)_2$] in the cement paste and carries it out of the concrete as the moisture moves through. If the moisture evaporates from the outer surface, the dissolved salts are left on the surface as efflorescence.

Typical examples are the cracks and form-tie holes in the foundation walls of underground parking garages, cracks on the underside of garage floor and roof slabs and cracks in exposed balcony floor slabs. On some buildings having highly permeable concrete and no or very poor waterproofing on the 'wet' side of the concrete element, the leaching and efflorescence may appear as a larger wet or stained area. This would indicate more widespread transfer of moisture through the slab (Figure 1-3).

The leaching at cracks and holes may be intermittent, perhaps seasonal, with fluctuating ground water or exposure conditions. Thus, the surface of the crack or hole may not always appear wet but, it will frequently exhibit a stain and encrusted salts or efflorescence.

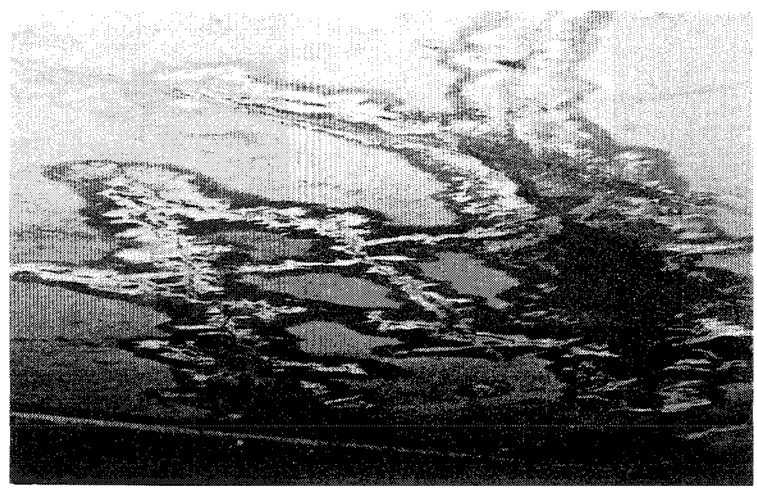

Figure 1-3 Leaching and Efflorescence

There are no standard tests applicable to leaching or efflorescence in concrete and it is generally not of significant structural concern; however, over time, the leaching results in a loss of lime in the paste and leads to a slow deterioration of the concrete with possible loss of structural integrity. The loss of free lime in the hardened concrete may also decrease the pH of the concrete as the alkalinity produced by the free lime is lost. This can result in localized corrosion of the embedded metals.

The more common issue that property owners must deal concerns municipal property standards which generally require that leaching into buildings be stopped as it affects the use of the property. If leaching is occurring at a structural wall or column, the situation should be examined by an expert to assess the significance of the leaching and to determine the type of repair that would be suitable.

Delamination and Spalling

Delamination is a separation of the concrete along a plane essentially parallel to the outer surface of the concrete. This separation plane is generally located at the level of the reinforcing steel and is associated with the corrosion of the reinforcement. *Spalling* is the creation of a depression resulting from the separation and removal of the delaminated concrete. Delamination is not necessarily visible on the concrete surface, whereas spalls are visible. Delamination can be detected by a characteristic "hollow" sound heard if tapped by a hammer.

Although delamination, and the ensuing spalling of concrete, is most often associated with corrosion of the embedded reinforcing steel, delamination may also be caused by load induced cracks parallel to the surface; however, this is less often the case. The electrochemical activity that produces the corrosion has been explored by a variety of research bodies over the past thirty years. While a summary of the processes that cause the damage to concrete is given in Appendix E, its occurrence in buildings is described below under the headings: *Salt and Moisture Induced Corrosion* and *Carbonation Induced Corrosion*.

Salt and Moisture-Induced Corrosion

Corrosion of reinforcing steel is greatest in warm, moist, high-chloride environments. While it was understood by designers and researchers that off-shore structures and bridges were moist, high-chloride environments, it was less well understood that concrete in buildings and, in particular, parking garages could also suffer the effects of corrosion (Figure 1-3).

The combined effects of the lack of knowledge of the potential problems combined with inadequate quality control during the heavy building periods of the 1960s and 1970s generated concrete buildings without adequate protection against corrosion. This resulted in building owners having to undertake major repairs to restore the delaminated and spalled concrete and to add the previously absent protection against chloride ions from road salts and moisture from snow and ice carried in by cars.

The initiation of corrosion and the rate at which it proceeds has now been studied extensively. The general conclusions are that good concrete construction practices, using mixes with low water-cement ratio and designs having greater cover over the reinforcing steel, will delay the onset of corrosion; however, even uncracked, high quality concrete provides inadequate long term protection[4].

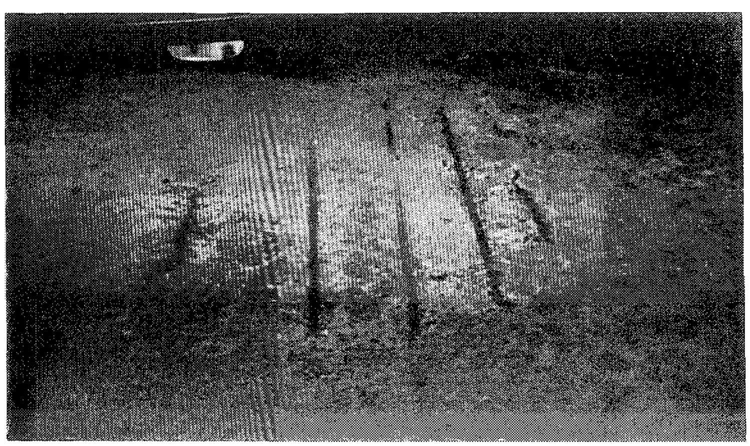

Figure 1-4: Spalling Induced by Corrosion of Reinforcement

Corrosion occurs most often in concrete exposed to deicing salts; however, it has also been known to occur in other exposed concrete following carbonation of the cementitious matrix. The naturally occurring phenomenon of carbonation is a world-wide problem and has been investigated and reported on by CMHC with regard to buildings in Canada.

Carbonation-Induced Corrosion

Concrete contains large quantities of free lime (over 20 percent of the hydrated paste). Under moderately humid service environments the lime readily combines with atmospheric carbon dioxide (CO_2). This process is called carbonation. The rate of carbonation is highly dependent on relative humidity, with humidities in the range of 50 to 70 percent producing the highest rates of carbonation[14,15]; however, even in normal dry air, which contains 0.03 percent CO_2, carbonation can occur.

Under normal atmospheric CO_2 conditions, the depth of carbonation may progress at a rate of approximately 2 to 3 mm/year. In higher level CO_2 exposures, moist atmospheric conditions and weaker concrete, this rate can increase significantly. A literature search performed by CMHC[14,15,38] has suggested that for Canadian conditions a relationship can be drawn between the depth of carbonation, time, and a factor related to concrete quality and environmental conditions. Exposed building elements most susceptible to carbonation would be balconies, which are in a moist condition. Vertical concrete wall elements would be less susceptible. Precast wall panels are the least vulnerable of the exposed concrete, due largely to the higher quality concrete typical of the precasting process.

Highly permeable concretes, having high water-cement ratios, are most likely to exhibit deterioration by this mechanism, as the CO_2 penetrates the hardened concrete more readily. The carbonation process itself decreases the permeability of the affected paste; however, the carbonated concrete paste is weak, so cracks occur and the overall permeability of the concrete is increased.

Of greater significance, however, is the fact that carbonation neutralizes the natural alkalinity of concrete. In the carbonation reaction, carbon dioxide reacts with hydroxides in the concrete forming a weaker paste with an associated drop in the pH of the concrete. Concrete normally is alkaline having a pH of 13. The alkalinity of concrete reacts with the iron of the reinforcing steel creating a protective film, rendering it "passive" and protecting it against further corrosion. The loss of alkalinity that follows carbonation effectively depassivates the reinforcement and corrosion of the steel is more probable, leading to delamination and spalling of the concrete.

A simple, cost-effective test for determining the presence of carbonation in concrete that has been employed for many years involves the use of phenolphthalein - a pH indicator. Thermogravimetric testing is more complex, but more accurate in determining the presence of carbonation in progress at depths greater than that found by the phenolphthalein pH indicator solution.

CMHC reports that carbonation induced corrosion is thought to not yet be a major problem in Canada. If corrosion has occurred, it is generally related to the depth of concrete cover over the reinforcing steel and the quality of the concrete. In these cases, the concrete should be locally patched.

Impact of Design Changes on Deterioration

It has been suggested that another contributing factor to the occurrence of distress may be the design criteria stipulated in building codes of different years[10]. Older structures which pre-dated the 1970 design standards appear to be in better condition than those constructed more recently (1970s and 1980s). Indeed, the design criteria for structural concrete have changed over the years to utilize improvements in concrete quality, reinforcing steel strength, and design philosophy. The result has, in some instances, been thinner structural members. The structures designed using thinner members comply with the structural design codes and were considered to be safe, good quality structures when built; however, it could be argued, for example, that the thinning of a parking deck slab does not allow as great a tolerance in the positioning of the reinforcing steel within the slab. The result is that otherwise completely suitable structural concrete been made less forgiving in the harsh salt and moisture saturated environment to which it could be subjected.

The increased sophistication of design and the use of improved concrete materials and stronger, more reliable reinforcing steel strength should have been accompanied by increased attention to quality control and workmanship; however, quality control and workmanship did not make corresponding improvements. The result was structures with improperly placed reinforcement, weak, permeable concrete and poor drainage. In addition, ideally, the design and construction codes of the time should have included clearer requirements for protective measures such as waterproofing, drainage, concrete cover, concrete quality, etc. Such was not the case, though, until the 1980s when structural concrete deterioration became common.

Design Changes in Response to Deterioration

Epoxy Coated Reinforcing Steel

One response to the corrosion of reinforcing steel has been the use of epoxy-coating on the reinforcement. Epoxy coated "green" (usually) reinforcing steel is protected from corrosion due to the electrical isolation or dielectric effect offered by the epoxy. CMHC's Technical Builders' Bulletin T-7 of September 21, 1984 gave new construction requirements for parking garages received NHA financing which, among several other protective measures, included the requirement for use of epoxy coated reinforcing steel for all bars, tie wires and supports. Epoxy coated reinforcing steel is now required by CSA Standard CAN/CSA S413-87 for all steel in ramp slabs and exposed roof slabs of conventionally reinforced parking decks and areas where plows may damage a waterproofing membrane. In addition, post-tensioned parking deck slabs that employ a sealer for surface moisture protection must have epoxy coated bars or, if a waterproofing membrane is used instead of a sealer, all bars in the top 100 mm of the slab must be epoxy coated.

Epoxy coating has now been widely adopted, but, it has generated new concerns. The process of fusion bonded coating reached its stride in the early 1980s in Canada when it was beginning to become applied to reinforcing steel for parking and bridge structures. The coating is supposed to be continuous; however, the presence of small nicks and scratches caused by transportation, stock piling and construction, is unavoidable. These defects, or "holidays" in the coating are potential points of corrosion. The question of the effect of using both coated and non-coated (black) reinforcing steel together in one slab was assessed in a study performed in 1983[35]. That study tested epoxy coated steel with holidays and combinations of epoxy coated steel with holidays and black steel under various conditions and concluded that pitting of the reinforcing steel at holidays and the combination of "green" and "black" steel was not a concern in respect to corrosion.

A second concern arose out of the apparent excessive cracking of slabs constructed with epoxy coated reinforcement. A study for CMHC in June 1989[37] evaluated the occurrence of such cracks. The findings confirmed that parking decks constructed with epoxy coated steel are prone to excessive cracking and that the cracking is not related to corrosion but to a reduction in bond strength between the coated steel and the concrete. In addition, since moisture penetration at cracks is a concern, parking decks with epoxy coated steel should also be waterproofed.

It has been suggested by one researcher[36] that epoxy coating of reinforcing steel should be discontinued as a viable primary protective system, as it has been shown by testing to exhibit corrosion at defects, separation of coatings, and poor correlation of results of performance tests between different suppliers of epoxy coated steel. The recommendation given in that opinion promoted the use of other protective systems and the assurance of electrical continuity of the epoxy coated reinforcing steel at the time of construction so that a cathodic protection system (see Appendix H) could be later installed. While there is merit in the use of additional protection, it is this author's opinion that abandoning the use of epoxy coated reinforcing steel in new construction removes a key aspect of corrosion protection and that considerably more evidence on the detrimental effects of epoxy coating, as opposed to apparent evidence of lack of absolute success, should be systematically gathered before halting its use in concrete exposed to salt induced corrosion.

The directive given by CMHC in 1984 to use epoxy coated reinforcing steel has been considered by some designers to be an overreaction to the problem of salt-induced corrosion. Design changes that respond to the probable effects of deterioration must err on the conservative side if the long term durability, use and maintenance costs of structures are to be fully considered. In this regard, while design evolution for strength and safety and the effective use of upgraded high performance materials can assure high quality new construction, long term protection against deterioration, in addition to epoxy coating should include: high quality, dense, air entrained concrete; increased concrete cover; proper slope and drainage; good detailing and construction that resists collection of debris and moisture; waterproofing; and durable traffic wearing courses.

AIR ENTRAINMENT

As a result of deterioration in parking garages, CMHC, through Bulletin T-7, in 1984, followed by the Ministry of Housing in 1988 and the CSA Standard for Parking Garages in 1987 began to recommend that parking garage floors contain air entrained concrete. Use of air-entrained concrete is now standard construction practice for garages. Owners of buildings, parking garages in particular, that are found to be constructed using non-air entrained concrete should take steps to protect the existing concrete from freeze-thaw damage. This would involve the application of sealers or waterproofing, draining ponded areas of slabs and reducing the use of deicing chemicals.

DRAINAGE

Ponding of water in parking garages has been addressed, again, through CMHC. Floors are now required to have a 2 percent slope to drains. Since it is impractical to adjust the slope of floors of existing structures, those that are poorly drained can often be improved through the use of new drains selectively located where water ponds.

PROTECTION SYSTEMS

All new garage floors and roofs are now required to have a minimum protection system through the application of waterproofing or sealers, depending on the type of structural reinforcing system and the use of epoxy coating. Repaired structures should also be protected using waterproofing or sealers. There is considerable debate over the value of epoxy coating existing reinforcing steel in repaired structures. Epoxy coating, repair concrete and protection systems are discussed in detail in CHAPTER 4 REPAIR AND PROTECTION TECHNIQUES.

CHAPTER 2 TEST AND EVALUATION PROCEDURES

GENERAL

Property owners are generally not technically equipped to perform meaningful evaluations of their structural concrete. They react only when obvious distress or user inconvenience occurs. At this point, usually, a consultant specializing in evaluation of structural concrete is called in to perform the necessary evaluations and tests.

After the distress is identified, evaluations and tests are generally performed in three steps. During the problem identification stage, information is gathered to permit an assessment of the nature of the deterioration. Following identification of the types of distress, information should be gathered to quantify the extent of the problem and its significance relative to the use of the affected elements. Finally, information should be gathered to permit detailed and accurate design of corrective measures. These three steps, *identification, quantification* and *specification* involve the use of test procedures that provide data from which evaluations can be made about the structure and its condition.

Figure 2-1 summarizes the tests performed to evaluate reinforced concrete in buildings. In addition to the test, the primary causes of the listed forms of distress are noted.

● Usual

○ Occasional

	Examination/Tests									Primary Cause(s)											
Distress	Visual Unaided	Visual Microscopic	Hammer/Chain Sounding	pH	Electro-potential	Cover meter	Chloride Content	Absorption	Special Non-Destructive Tests	Moisture	Chloride	Sulphate	Carbon Dioxide and Moisture	Unprotected Embedded Metals	Misplaced Reinforcing	Thermal Stress	Load	Deleterious Aggregate	Initial Shrinkage	Poor Air Void Distribution	Poor Placing/Curing/Finishing
Scaling and Pop-outs	●	●						●		●	●							○		●	○
Disintegration	●	●							○	●		○						●			
Wide Crack	●								○						○	○	●				
Narrow Crack	●								○						○				●		●
Leaching & Efflorescence	●							○		●	●	○					○		○		
Reinforcing Steel Corrosion	●		●	●	●	○	●	○		●	●		○	●	○						
Post-Tension Cable Corrosion	●								●	●	●			●							
Carbonation	●		●	●				○		●	●		○		○						○

Figure 2-1 Test and Evaluation Procedures Related to Deterioration and Primary Causes

The test and evaluation procedures used can be broadly grouped as:

- Visual Examination
- In-Situ Non-Destructive Tests
- Laboratory Tests

VISUAL EXAMINATION

Visual examination of the structure should always be the first step in evaluating the existing conditions. Such inspections can focus subsequent testing efforts on the problems that are the are most significant to the particular structure and can permit preliminary assessments of structural condition.

Generally, the visual examination involves the use of hand tools, including: hammers, heavy chains or bars to "sound" the concrete for hollow spots (delamination), chisels to remove small samples for closer visual examination or examination with the aid of a 10x or 15x magnification hand lens; a knife to scrape the surface; and, perhaps, phenolphthalein solution to test for pH level.

Information gathered during the visual examination stage about the condition of in-situ materials is combined with a review of the structural design. Such concerns as design parameters, reinforcing steel type and location, specified concrete strength and materials properties are addressed. An assessment is made of the physical conditions, dimension, tolerances and workmanship and a mapping provided of the visible distress. The result is a preliminary and sometimes sufficiently accurate understanding of the nature, extent and severity of the deterioration. This information can also be used to arrive at quantities for repairs that may be necessary.

Should the deterioration be of a nature that the simple tests performed during visual examination are not able to arrive at the extent or severity of the distress, certain non-destructive tests may be of assistance to the investigator to better define the nature and extent of the deterioration.

At this point in the investigation process, preliminary discussions should be held with the owner to outline the possible repair options and relative costs.

IN-SITU NON-DESTRUCTIVE TEST METHODS

GENERAL

During the 1970s and 1980s, the research into non-destructive testing (NDT) of reinforced concrete increased dramatically. The research grew in response to the needs of more rapid concrete construction practices, such as continuous slip forming, which demanded faster assessments of in-place strength. Methods were also developed to respond to a growing need to evaluate existing structures for in-place strength and the onset of corrosion of reinforcing steel. Advances in digital signal analysis and computerization during the 1980s allowed the development of equipment that was more portable and which gave consultants a quick response to characteristics of the reinforcing steel or concrete. The data provide an indirect measurement of the properties that were in question[30]. Generally, the indirect measurement techniques require some form of calibration of the characteristic measured with the property in question. Use of non-destructive testing equipment thus requires knowledge of the properties of reinforced concrete and the technology behind the equipment being used.

Consultants using NDT equipment must be prepared to explain the significance of the data to enable property owners and managers to be part of the risk assessment process. Costs of the repairs and the reliability or durability of the repairs will depend on the joint agreement between the owner/manager and the consultant as to the significance of the data.

The earliest NDT methods concentrated on in-situ strength and several tests based on impact resistance (e.g. the rebound or Schmidt hammer and the powder actuated pin penetration or Windsor probe test). These procedures remain useful to compare the relative in-situ resistance to the test of similar concrete at different locations. They should not be used to assess actual in-situ strength.

Over the past 10 to 15 years, pull-out and break-off strength test procedures have been developed to give approximations of the in-situ strength of the concrete. These methods are occasionally used both during construction and in the evaluation of deteriorating structures. Generally, the in-situ strength tests are not performed at locations where the concrete appears to be deteriorated as that concrete is obviously inadequate. In-situ strength tests are useful to assess the strength of concrete where conventional core extraction and subsequent laboratory compressive strength testing is impractical. For example, in-situ strength tests can be used to evaluate the structural adequacy of the concrete walls and columns in occupied spaces to determine if the design loads can be increased.

More recently, non-destructive test method development has focused on the need to assess: the presence of anomalous conditions in the concrete; conditions related to corrosion of reinforcing steel and post-tensioning cables; and the nature and extent of cracks or voids in the concrete. These NDT methods include:

- Infrared Thermography (IR scans)
- Ground Penetrating Radar (GPR)
- Ultrasonic Pulse Echo (Pulse Velocity)
- Half-Cell Electropotential (Half-Cell)
- Pachometer (Cover meter)
- Time-Domain Reflectometry (TDR)

Before commissioning a testing program, owners should make a general assessment of the amount of technical data that they need to make a decision about repair needs. If repair needs can be defined without the data provided by a testing program, there is little added value to the owner if the consultant performs the tests. For instance, if it is clear to the owner and the consultant that cracks in a column are caused by structural movement, the results of pulse velocity tests to determine the exact depth and location of the cracks are of no particular use. In this case, a structural analysis would be far more useful in developing ways of relieving the stress causing the cracks. As a rule, building owners and managers should be convinced by their consultants as to the value of performing any tests.

Not all of the NDT procedures lend themselves readily to use in buildings. For instance, IR scans require that the anomaly create a discernible temperature difference at the surface of the concrete. The technique has successfully located delaminations in bare concrete bridge decks. Most often, however, the lack of temperature differences in such locations as concrete parking decks makes thermography impractical.

The GPR technique is best suited to location of reinforcing steel, by virtue of the strong reflections created by the dense steel.

More common is the pulse velocity test. In this application, the time between sending and detecting sound waves is used to locate cracks and voids in concrete. If the concrete is deteriorating, however, perhaps due to freeze-thaw damage, this method would likely provide poor data as the voids and micro cracks, combined with the heterogeneous nature of cement and aggregate, would result in a multitude of spurious reflections of the sound waves.

The majority of non-destructive testing performed on the concrete components of buildings attempt to find details about the occurrence of corrosion of the embedded reinforcing steel.

HALF-CELL ELECTROPOTENTIAL TEST

One of the more common non-destructive tests for evaluation of deteriorating concrete in buildings is the half-cell electropotential test. These tests have been used routinely over the past 15 to 20 years and, due to their simple nature, have become relied upon to assess corrosion and position of reinforcing steel in buildings. ASTM test method C876 describes the test procedure in detail.

The electropotential half-cell test (Figure 2-2) is performed on a close grid pattern, generally 1.5 m, or closer, between readings. To perform the test, a low resistance multi-strand wire is attached to the reinforcing steel mat and to a voltmeter. From the voltmeter, the wire is connected to a copper rod immersed in a saturated solution of copper sulphate (the half- cell). The bottom of the half-cell is open to the slab through a porous stone and wet sponge. When touched to the top of the reinforced concrete slab a potential voltage is read on the voltmeter. Values numerically greater than minus 350 mV (millivolts) are considered to represent a high probability that the reinforcing steel in the test area is actively corroding. Values numerically less than minus 200 mV, similarly, represent high probability of no corrosion (passive).

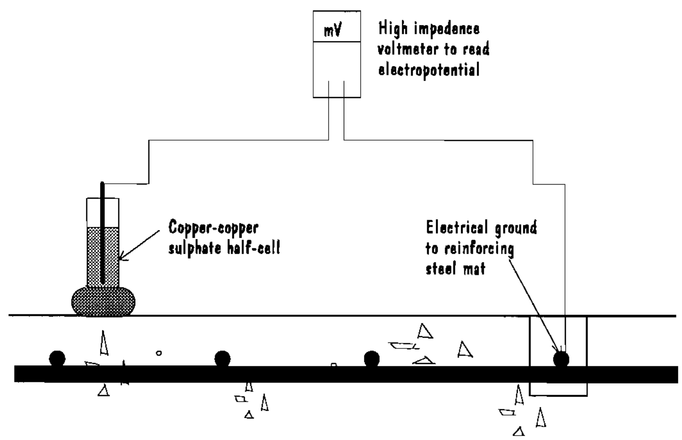

Figure 2-2: Electropotential Half Cell Test Configuration

Values in between minus 200 and 350 mV represent an uncertain corrosion condition. If the test results indicate a large proportion of the values in this range, it would be prudent for the owner to monitor the corrosion potential at least annually and, possibly, take steps to mitigate the corrosion potential by application of a protective membrane or other suitable means.

When coupled with the results of chain-drag surveys and visual examinations, the half-cell and cover meter tests can give considerable data about the condition and, likely, longevity of the structure (usually floor slabs in parking garages). Recent studies[19] seem to indicate, though, that the heretofore reliance on the half-cell test as a measure of corrosion potential is unfounded and that it should be discontinued as a routinely applied test. One major drawback of the test is that the results indicate only the 'potential' for corrosion. The results do not give information about the rate of corrosion. Further, the presence of values significantly greater than minus 350 mV should not be interpreted as indicative of a greater amount of corrosion. The half-cell test has also not been demonstrated to be useful in assessing small increases or decreases in corrosion potential over time as the repeatability of test results is poor. Still, the half-cell can be used in conjunction with visual inspection and laboratory tests as one of a series of tools to provide data leading to the condition assessment of the structure.

The proposal to discontinue the test as a standard practice when assessing corrosion potential warrants examination. Certainly a test method that provides no useful information should not be used; however, the use of the test method must be assessed in conjunction with the suitability of the structure to provide appropriate values as data. For example, the voltmeter must be grounded to the reinforcing steel. Since the half-cell values are recorded over a large area and the ground point is possibly quite remote from some test locations, the reinforcing steel is usually checked for electrical continuity by recording a value at one ground location, measuring a small number of electropotential values and then moving the ground connection to another location and verifying the potential values at those same test locations. If poor correlation is found, the ground connection is moved and the potential checked again. Erroneous values will arise if the test is performed over reinforcing steel that is not electrically connected to the ground point. The layout of the reinforcing steel of concrete flat slabs is fraught with opportunities for poor electrical conductivity between the upper and lower mats and between mats at different columns and mid spans. Thus, the test method requires considerable verification of continuity of reinforcing steel, a difficult and tedious process; however, once continuity is established, it has been this author's experience that the procedure provides useful information about corrosion potential - information that often relates directly to the presence of rust and delaminations.

There are definite limitations to the use of the test procedure. Since measurement of corrosion potential using the half-cell relies on electrical continuity within the reinforcing steel mats and between the reinforcing steel and the concrete, the test method is not useful if the reinforcing steel to be tested is coated with epoxy, a dielectric that isolates the reinforcing steel from the corrosion process as well as the test. It is also not useful for structures designed and built using post-tensioning cables that are often sheathed in a grease-filled sleeve.

COVER METER

Cover meter tests are performed using commercially available metal detectors. A scale or calibrated charts provided with the instrument allows readings to be translated to depth of reinforcing steel for known bar sizes. The battery powered instrument generates a magnetic field at a probe that is moved across the surface of the concrete. Each bar of the reinforcing mat interrupts the magnetic field given off by the hand held probe. This distorts the intensity of the magnetic field that is recorded on the scale. The maximum deflection of the analog gauge is related to the bar size and depth. The information can be used in structural calculations and to establish shallow concrete cover and thus susceptibility to corrosion induced by chloride ions or carbonation.

TIME DOMAIN REFLECTOMETRY

As corrosion of post-tensioning systems might result in sudden tendon or anchor failures, CMHC has begun evaluating the use of a method of assessing cables in place using Time Domain Reflectometry (TDR). The use of this technology is particularly important in western Canada, where many recently built structures employed post-tensioned concrete. TDR tests involve exposing one end of a post-tension cable at the anchorage point. A high frequency electric pulse is sent into the cable through the connection point, most commonly through the anchor. The pulse travels along the surface of the wires in the strands to the end of the strand where it reflects back and is detected at the source point. The signal can then be recorded or read on an oscilloscope. If there are no flaws in the wires, there should be one "blip" in the signal representing the end of the wire. Defects or changes in environment, affecting the wires in the post-tensioning cable, cause intermediate reflections that are also recorded.

The process is in its developmental stages at present and presents some difficulty in screening out the extraneous reflections caused by reinforcing steel in contact with the cables and anchorages and spurious reflections from adjacent post tensioning cables not electrically isolated from the cable being tested.

LABORATORY TESTS

GENERAL

Samples taken from the site for laboratory tests also provide data that can be used to better understand the nature and extent of evident and pending deterioration. Typically, laboratory testing of concrete is associated with strength tests of cored samples removed from the structure. As is the case for in-situ strength evaluations, the strength test of cores does not provide useful information about material deterioration. Strength tests can, however, be used to assess strength of apparently sound concrete for comparison with loading requirements. They are really only useful as data for structural analysis and not as a material evaluation tool.

The laboratory tests commonly performed to assist in establishing the nature and extent of deterioration of concrete involve assessment of the air void distribution, as related to freeze-thaw resistance, and the assessment of chloride ion content, as related to corrosion of reinforcement. In conjunction with studies on the potential problem of carbonation, CMHC has recently sponsored research into laboratory evaluation of carbonation based on water absorption[16]. These test procedures are reviewed in the following.

AIR VOID PARAMETERS

The air void distribution assessment requires the use of concrete cutting and polishing equipment to prepare samples for viewing under a stereoscopic microscope. The prepared samples are placed on a coordinate table or stage that can move the sample through the field of view in increments along a traverse. A high intensity lamp, placed at a low angle of incidence to the sample, causes the air voids to cast shadows and thus become clearly visible. ASTM test procedure C457 describes the method of examination, characteristics to be measured and calculations to be made, to determine the total air content, spacing factor and specific surface of the air voids - such being the important characteristics of air entrained concrete (see Appendix B and Appendix D).

Since there is no widely used test to determine the suitability of air-void parameters in fresh concrete, owners of newly constructed buildings would be able to use this test as a means to assess the freeze-thaw resistance of concrete used in their structures. Owners of structures having deteriorating concrete can use the test to help them to determine if the deterioration is the result of freezing and thawing or some other process of disintegration. Microscopic examination can also be used to assess the presence of reaction materials formed by alkali-aggregate reactivity, though, this is not common in buildings.

CHLORIDE CONTENT

The chloride ion content of samples of concrete taken from buildings is frequently determined to assess the amount of chloride that has penetrated to various depths in parking garage floor slabs. Typically, a 75 to 100 mm diameter core sample is removed from the floor to a depth of likely 150 to 175 mm in a 200 mm thick slab. The sample is then prepared by cutting 10 mm thick slices of concrete from the core sample at the depths for which the chloride content is to be determined, generally a section at the top 10 mm, a section at the depth of the top reinforcing steel and a section near the bottom of the core. See Figure 2-3.

It is unlikely that chloride has penetrated the full depth of the slab so chloride content at the core bottom should represent the amount of chloride present at the time the concrete was placed (background chloride) resulting from admixtures and aggregates. The slices of concrete are each pulverized, put into solution and treated with a chemical that permits calculation of the chloride ion content present. Typically, water soluble chloride is measured. Chloride in the concrete that is not water soluble is not a concern as it does not contribute to the corrosion process to any significant degree.

The resulting chloride concentration profile with depth assists the investigator in determining the severity of the chloride-induced corrosion problem.

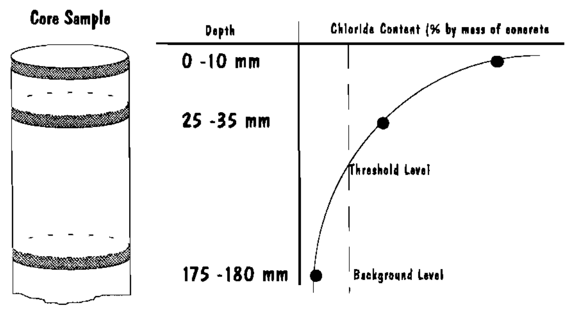

Figure 2-3: Chloride Ion Test Results

Occasionally, powder samples are obtained instead of cores by recovering the drilled dust from various depths. When sampling in this manner care must be taken to prevent contamination of deeper samples by material scraped off the sides of the drill hole by the drill bit. Several samples representing each level must be taken and combined to represent the combined impact of cement paste and aggregate at the test level. Spurious test values could arise from a single drilled powder sample that encountered a particle of high chloride content coarse aggregate.

In cases of widespread deterioration of slabs, the depth of chloride ion penetration can be used as an indicator of the volume of contaminated concrete that likely has to be removed and replaced. This data, together with chain drag and half-cell surveys, is used to calculate the costs for repair.

CARBONATION

Carbonated concrete is conducive to corrosion of embedded metals. Owners of buildings that have exposed balconies and walls that were previously considered to be free of long term deterioration processes should be concerned about the effects that carbonation has had on their structures. Typically, the depth of carbonation has been tested using phenolphthalein solution as a pH indicator. The test requires that the solution be applied to a freshly exposed face of the concrete to avoid spurious results caused by surface carbonation. The test indicates whether or not the concrete has a pH of more than 9 or 10 by staining the concrete pink in the high pH areas. Lower pH concrete is unaffected by the phenolphthalein solution. There is no gradient of pH and, thus, no measure of the relative amount of carbonation that has occurred.

A test method that attempts to relate concrete quality, in terms of its water absorption capabilities, to the depth of carbonation was employed in a study for CMHC as part of their overall efforts in assessing the potential problem of carbonation in Canada. The test involved oven-drying concrete core samples then soaking one surface in distilled water and measuring the mass gained at close time intervals. When mass gain occurs at a uniform rate, the "sorptivity" of the material is determined. Each sample had been previously tested using a phenolphthalein solution to record the depth of reduced pH which indicates carbonated concrete. The tests indicate a good correlation between sorptivity and carbonation.

Owners may use the test to determine to what extent carbonation has affected their structure and, thereby, better plan for repairs.

Typically, sorptivity values can be expected to vary depending of the surface tested, (e.g., formed or trowel finished) as the surface density will differ. Concrete density, water-cement ratio, curing practices and finishing techniques should be considered when evaluating the results of sorptivity tests.

CHAPTER 3 REPAIR ALTERNATIVES AND COST IMPLICATIONS

PROBLEM DEFINITION

The repair or treatment of deteriorated concrete will likely differ between different building components. For instance, the repair of delaminated slabs in a parking garage differs from the repair of cracks in walls. Repairs may also differ between buildings of different ages. For instance, the suspended slab of a parking garage constructed during the building boom of the 1960s will require a different approach to repair than the parking floors of a building constructed in the 1980s; not only is the building of the 1980s newer and, thus, the problems with the performance of the concrete are, hopefully, less severe, but as a result of new design practices, construction techniques and building code requirements, the floors are constructed differently. Finally, repairs may even differ between buildings of similar age due to differing ownership and equity positions. In fact, each building puts forward a different set of parameters concerning age, structural design, exposure conditions, maintenance, finance and ownership. These differences are valid and should be considered when identifying the problem and deciding how to respond.

The identification of a problem, in the performance of concrete in a building, sets in place a series of questions and the way in which those questions are answered will determine how the problem is resolved. Since concrete is used for a variety of both structural and non-structural components of buildings, the action taken can follow many different paths, each leading to a different way of dealing with the performance problem. Further, the property owner must appreciate the full scope of the problem. To investigators, the problem of concrete performance may be a technical or safety issue involving how to identify and repair various forms of concrete distress. To building owners and managers, the problem will include how to pay for the repairs and whether the repair cost is a sound use of those funds.

When assessing the need for repair, one must also consider the significance of a failure. That is not to say that impending structural collapse is the only cause for concern. Failure can mean that the concrete can no longer serve its intended function: for instance, a concrete surface exposed principally for aesthetic reasons, such as a precast concrete cladding element, that shows minimal surface deterioration, may create an unserviceable state because it no longer serves its purpose as an aesthetically pleasing surface. If the concrete is used in a "semi-structural" element, such as a slab-on-grade, where smooth finish is important for traffic purposes, surface deterioration may render the element unserviceable because cars can no longer comfortably use the floor. If the concrete is used in a structural element, deterioration causing loss of strength of the concrete or strength of embedded reinforcing steel could destroy the serviceability because of resulting deflections or inadequate structural strength. The significance of these situations depends on how critical the loss of use of the non-performing concrete is in relation to its intended use.

What to repair, how to repair it, when the repairs should be done, and why the repairs should be done are questions that must be addressed before performing any work. Figure 3-1 depicts many of the considerations that may weigh in the decision to do repair work. Some are more applicable to privately owned buildings, some to co-operative housing or condominiums and some to public non-profit housing. Each building owner or property manager responsible for the sound condition of the facilities should fully examine the various opportunities and constraints appropriate to their property.

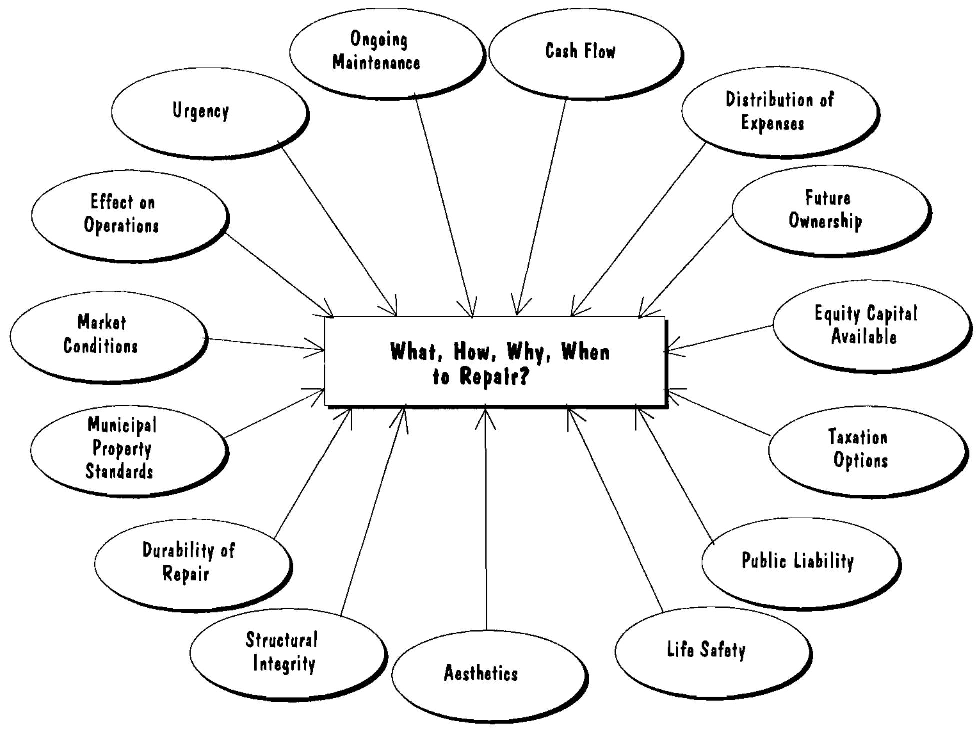

Figure 3-1 - Owner's Concerns

ECONOMIC ASSESSMENTS

Alternative repair options are available in virtually all cases of failed or deteriorated concrete and each alternative repair option has a different cost associated with it. It is, therefore, necessary to link engineering assessments of the evidence and cause of the distress with economic assessments of the financial feasibility of the alternative repair strategies.

This economic analysis is accomplished by using one or a combination of three common types of economic analysis procedures which, for the purposes of this review, are called: *cost-benefit analysis, compliance analysis* and *life-cycle cost analysis*. The first two processes, cost-benefit and compliance analyses are less rigorous than the life-cycle costing process. As a result, they are often performed without formal consideration by owners and their consultants.

COST-BENEFIT ANALYSIS

Cost-benefit analysis is a comparison of the known costs of one or more repair alternatives with the expected benefits obtained from that repair over a set period of time. Both tangible and intangible benefits can be included in the assessment, meaning, those benefits which can be measured with some confidence, such as financial gain or loss, and those benefits which can not be readily measured, such as possible good will generated by improving the condition of the concrete. The analysis can involve simply listing the benefits that are expected and making a subjective judgment as to whether or not the cost for the alternative is warranted. Since cost-benefit assessment includes some intangible elements, the process tends to rely heavily on application of a realistic weight to the intangible benefits.

COMPLIANCE ANALYSIS

This procedure is a comparison of one or more repair alternatives of known cost against a set of prescribed objectives. To properly complete this type of analysis it is vital that the objectives of the study be clear, reasonable and that they be established before the alternatives are developed, as the alternatives are formulated with the intent of meeting most or all of the objectives. The process can be a simple matter of listing the objectives and identifying the points at which each repair alternative falls short of those objectives. Failure to comply with one or more objectives need not disqualify a repair alternative. Application of the appropriate weight to achieving each of the objectives will assist in selection of the most appropriate repair alternative.

LIFE-CYCLE COST ANALYSIS

This procedure is a comparison of the total cost of design, remediation, maintenance and subsequent iterations of that process over the deemed life of the structural element for one or more repair alternatives. The life of the element could be assumed to be the same as the life of the building, the life of the particular element or any other defined period. For instance, the concrete sidewalks in front of a residential high-rise building could be considered to have a deemed life of 40 years, the same as the building. The life-cycle cost for the sidewalk would therefore include total replacement at some point during the life of the building as the sidewalks would deteriorate, crack and be subjected to accidental damage. The deemed life could instead be the life of the sidewalk itself, up to the point at which it should be replaced or, the life could be deemed to relate to some event in the life of the building, perhaps the sale of the property or another predetermined planning horizon.

While life-cycle costing involves known costs for the work, as is the case when making cost benefit and compliance analyses, unlike the other analyses, life-cycle cost assessments involve assumptions about the future value of money and rates of interest that could be earned on money if it was invested in some asset other than the repair work. However, considering the analyses are comparisons between alternatives, errors in forecasting interest and inflation or discount rates are not significant.

The expense involved in some types of concrete repair can amount to a substantial proportion of the value of the building. The work that owners must do to correct deteriorated concrete, therefore, should not only make technical sense, it should also make financial sense. Appendix G works through a case study which shows how different owners may employ different repair schemes to accommodate their particular financial needs. The basics of life-cycle costing are described below.

When applied to repair and maintenance of structural concrete, life-cycle cost analysis must consider two basic ideas. These are:

1. Money has a time-dependent value and owners should look for ways to optimize their repair expenditures. For instance, for some owners, it may be more cost effective to perform repairs of a minor nature on a more frequent basis as an operating cost, rather than perform less frequent, larger repair programs as capital expenditures.

2. Concrete repairs are required usually after several years of satisfactory use; however, concrete often deteriorates at increasing rates with time. For instance, the exposed concrete balconies of high-rise residential buildings sometimes do not exhibit deterioration for as long as 20 years after initial construction; however, the effects of carbonation and corrosion can cause the slab edges to delaminate quickly once the deterioration processes begin.

Reliable life-cycle cost analyses depend on good data to use as a basis for assumptions and predictions of future conditions. Professional engineering consultants' assumptions about the future condition and the expected life of repair strategies can be relied upon to prevent unsafe or unsound conditions, as these professionals will consider public safety to be a first priority. Unfortunately, consultants' predictions about future rates of deterioration are, most often, based on the conditions observed at one point in the history of the deterioration. As consultants usually do not have the opportunity to measure the rate of the deterioration over time, they usually infer future conditions based on experience and information provided by the owner or manager of the property. In such cases, the alternatives for repair should be presented within the context of at least two possible future deterioration scenarios. These would be selected by the consultant and reasonably predict the probable maximum and minimum rates of deterioration for the particular concrete element needing repair.

One or more of the above noted economic analysis tools should be integrated with the engineering condition analysis. Failure to do so will inevitably result in too narrow a view of the technical problem and will produce repair solutions that may not meet the particular current and future needs of the building owner.

TERMS USED IN LIFE-CYCLE COST ANALYSES

Present Principal (P)

The Principal amount is the single sum of money owing at the beginning of a term of payments, in which case it is called the Present Principal "P". In life-cycle cost evaluations, this would be the estimated cost to perform repair work if the work was all done immediately.

Inflation (e_1) and Escalation (e_2)

The value of money is dependent on when it is to be spent (or received); for example, an item that cost $10,000 today may cost more, in current dollars, if purchased one year from now. This is known as inflation. It is usually expressed as a percentage of a known previous amount, i.e., "4 percent inflation rate". Inflation is the term "e_1" in life-cycle equations and is shown as a decimal, i.e., "0.04".

Deterioration of concrete adds another cost escalation factor to the normal inflation rate applied to money. Since deterioration increases with time, a factor is needed in life-cycle costing to account for increases in deterioration should the repair work have to be staged over several years or, should owners wish to know the effect of deferring repairs on the costs for the work. For our purposes, this factor will be identified as "e_2" to distinguish it from normal inflation which will be identified as e_1. It is also shown as a decimal. For instance, increased delamination in a parking garage slab that occurs at a rate of 10 percent per year would have an escalation factor e_2 of 0.10.

The combination of inflation and escalation can add greatly to the increased cost for repairs. For instance, if the inflation factor is 4 percent (0.04), repairs that cost $10,000 today and are expected to increase by 10 percent per year (0.10) until repaired would cost $11,440 the following year.

$$\begin{aligned} &\$10{,}000 \times [(1 + e_1) \times (1 + e_2)] \\ &= [\$10{,}000 \times (1.10 \times 1.04)] \\ &= \$11{,}440 \end{aligned}$$

Interest (i)

If $10,000 were invested today in an interest bearing account or other investment, it would accumulate interest. This is also expressed as a percentage in conversation and as a decimal, "i", in life-cycle cost equations. In life-cycle cost assessments, it is sometimes necessary to know whether it makes financial sense to temporarily put money, that could be spent on repairs, into an interest-earning investment and defer the repair work. In the example of the $10,000 repair, if the owner could put the $10,000 into an investment that earns 12 percent per year for one year, the total amount earned in interest would be $1,200.

$$\begin{aligned}&\$10{,}000 \times (1 + i)\\&= \$10{,}000 \times (1.12)\\&= \$11{,}200\end{aligned}$$

Since the interest earned ($1,200) is less than the escalation costs ($1,440), the owner would be financially better off doing the repairs now rather than investing the funds and waiting a year.

Future Principal (F)

The Future Principal amount "F" is the single sum of money that would be paid after a prescribed period. In life-cycle cost evaluations, this would be the cost of the repair work (in current dollars) if the work were deferred. This factor includes the effects of inflation and escalation in deterioration.

Interest Periods (n)

The length of time between the start and end of the payments applied or interest earned is broken into a number of periods referred to as "n" in life cycle cost calculations. If the period is five years, n=5. If the period is 60 months, n=60. The number depends on how often the interest and inflation are to be added into the calculation. The calculation of n is often used to determine the payback period as a measure of the soundness of the investment.

Initial Payment (A)

Often, cash flow is of concern when accounting for repair costs and, as such, the maximum regular payment (or income) needs to be known or established. This is referred to as the Initial Payment "A" which will be made over "n" periods.

USING LIFE- CYCLE COSTING

If an owner wants to know which of two or more alternative repair strategies is the most cost effective for him, it will be necessary to perform a life-cycle cost assessment. To assist in that assessment life cycle-costing equations are used. The nature of the calculations performed depends on the economic factors that are most critical to the particular owner.

For example, an owner that wants to minimize the total life-cycle costs for the repairs should calculate the value of the present principal (P) also known as the present worth of each of the alternatives. The least value of P would be the economically favoured alternative. On the other hand, an owner may have a restricted income and, thus, must have tight control on the cash flow associated with repairs; that owner would want to calculate the amount of each payment (A) for each alternative to be assured that it is in line with expected income. As a third case, an owner may wish to know if the interest earned on the money that could have been spent doing repairs now reduces the total amount, in current dollars, if the repairs are deferred.

For reference and use by owners in their own calculations, Appendix G gives the life-cycle cost equations needed to find the various factors and simple examples of how life-cycle calculations can help owners decide which repair options are financially best for them.

SITUATIONS TO AVOID

The owner of a building is well advised to develop a clear understanding of the anticipated results of a repair strategy, including the life of the engineered repair alternatives proposed, and the financial and operational opportunities and constraints facing him. While this report should assist owners to understanding the technical requirements and methods of repair, before repairs are undertaken, the specific merits and limitations of each applicable repair alternative must be understood. Owners should be informed, either by their consultants or by using their own knowledge of their building, about the long term implications of each repair alternative. Owners should also know the initial capital and ongoing maintenance costs for all viable alternatives.

Owners are cautioned against accepting advice that depicts only one repair alternative, particularly if the owner's financial and operational considerations are not explicitly satisfied by that alternative or are not mentioned at all in the consultant's discussion on the merits of the proposed repair strategy. For example, the repair of the sidewalk noted above could involve the patching of cracks and isolated replacement of concrete. The repairs would be effective and relatively inexpensive; however, if in the owner's opinion, the appearance of the sidewalk was vital to the perceived quality of the building, patch repairs would be inappropriate, as they would not be aesthetically acceptable.

Owners should also avoid situations wherein they feel pressured into effecting a repair strategy that is based largely on a perceived structural or safety inadequacy. If a structural safety inadequacy exists, it should be fully explored by the engineering consultant and alternative engineered solutions developed to deal with the structural aspects of the repair. Frequently, structural repairs are possible which modify the existing structural support system in such a way as to deal with the safety issue and, thereby, avoid extensive reconstruction of the failed concrete. For example, rather than replacing a parking garage floor slab which, due to deteriorated concrete and construction errors in the original placement of reinforcing steel, is now considered to have insufficient support around columns, it may be possible to provide additional support at the columns through the use of more reinforcing steel, drop panels below the slab or capitals at the tops of the columns. This would allow the owner to solve the structural problem and simply repair the deterioration. Aesthetic issues would necessarily form part of the engineered alternatives as well as the economics of repair against replacement.

Finally, owners should avoid circumstances which force them to make decisions based on incomplete information. If an economic assessment is not performed, either by the owner or the consultant, decisions about the repair strategy will be based on only some of the necessary facts. Whether the absent information is of a technical, financial or operational nature, owners must feel that they are making decisions based on a complete understanding of the repair alternatives and cost implications associated with each alternative.

ACTION PLAN

The repair of concrete in buildings is the final step, in a series of steps, that should be followed by the owner. These steps apply regardless of building ownership, age and financial situation. The approach, depicted in Figure 3-2, can help owners work with their consultants and, possibly, avoid overlooking an important consideration.

The eight step procedure shown describes the process of concrete repair from identification of a performance problem through the selection of consultants, to investigation, specification and, finally, completion of the repairs. Some of the steps are taken by the owner, some by the consultant and some jointly between the owner and the consultant. At each step there are certain issues that must be resolved.

It should be noted that, if the owner or manager is comfortable in dealing with the assessment of the technical needs of the repair, preparation of appropriate specifications and administration of the contracts, the owner may elect to not retain a consultant. In general though, the consultant does add value to the work through his experience. The consultant also acts as the interpreter of the specifications for repair and administrator of the contract between the owner and the contractor. The owner's role is by no mean diminished by the hiring of a consultant. It is the owner or manager, not the consultant, that has available, all the non-technical input required for the evaluation of what, how, when and why to repair. Only the building owner or manager has the needed information on operational and financial implications of the various strategies for repair. These non-technical arguments must be no less persuasive than the technical arguments.

The eight steps, shown in Figure 3-2, could be considered to be milestones in a road map to identify repair needs and establishing a repair strategy. The steps are described below:

STEP 1. IDENTIFY PROBLEM

Detection of deterioration may result from a complaint, visual evidence, building code or by-law requirement. As discussed in Chapter 1, the deterioration can usually be seen but the cause may not be obvious. Development of solutions to the problem requires knowledge of the cause of the deterioration.

STEP 2. DEFINE OBJECTIVES

Immediately after a problem is identified the owner must establish the objectives of the repair as well as any constraints that would affect the implementation of the repairs. The objectives and constraints will usually include: aesthetics; funding; limitations on the amount of repair work that can be performed at one time; the life expectancy of the repair; and, the maintenance that the owner intends to do once the repairs are complete.

STEP 3. OBTAIN NECESSARY EXPERTISE

If the owner elects to proceed with the repair work, the owner must decide if an expert consultant is required to assist in the development of repair strategies and/or the coordination of the repairs. If the assistance of an expert is required (as is generally recommended), selection of the expert should be based on their relevant experience with similar evaluation and work and resources available.

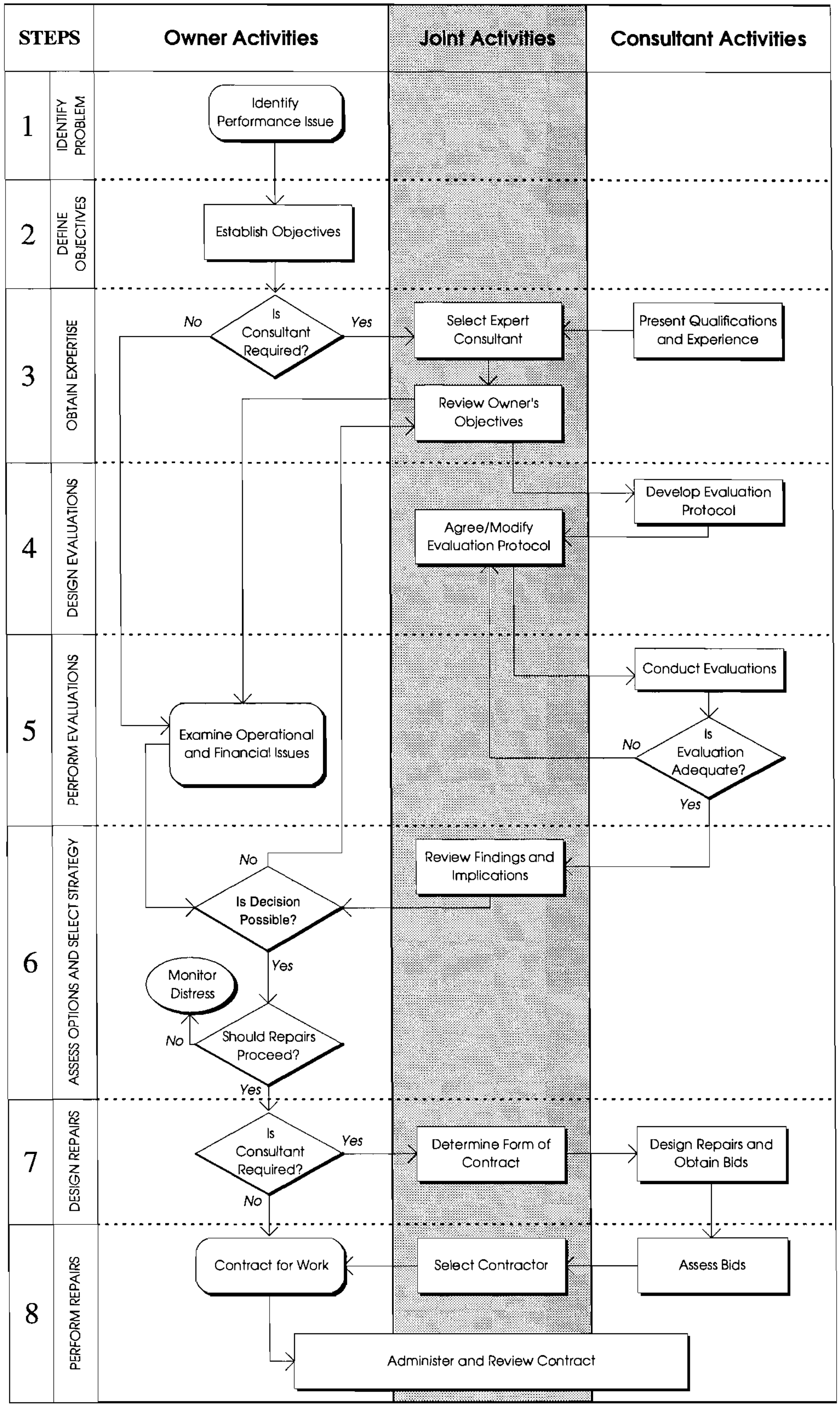

Figure 3-2: Roadmap to Concrete Repair

The owner and the consultant must come to a mutual agreement and understanding of the objectives and the constraints, including the financial, operational and the technical considerations that will impact on the methodology of, not only the repairs to be performed, but the level of effort and investigation techniques that the consultant will employ to assess the repair needs. The level of effort and the expertise of the consultant are prime determinants of the professional fees that will be charged by the consultant.

STEP 4. DESIGN THE EVALUATION PROTOCOL

Based on the agreed objectives, the consultant will develop an evaluation protocol. This should be reviewed and agreed upon by the owner before the consultant begins. Some forms of test may impact on the use of the owner's facility. It is, therefore, important to both parties that the nature and purpose of the various examinations and tests employed by the consultant be fully understood by the owner. At all times, the consultant's protocol must reflect the owner's objectives.

Evaluations can be performed in stages. In this way, it may be possible to minimize the cost of the evaluation step by eliminating some of the tests that become unnecessary as information is gathered.

STEP 5. PERFORM EVALUATIONS

During the evaluation stage both the owner and the consultant have assessments to make. The consultant must complete the necessary evaluations to determine the cause, extent and implications of the various forms of deterioration and provide alternative methods of repair in keeping with the owner's objectives. The owner must determine the most appropriate alternatives for financing the repairs, the impact that the owner is willing to have on ongoing operations and the future use of the property.

During the course of these evaluations the consultant may determine that additional information is required beyond that which can be obtained using the agreed protocol. In such cases, the additional evaluation steps should be presented to the owner and agreed upon as before.

STEP 6. ASSESS OPTIONS AND SELECT A REPAIR STRATEGY

The owner and the consultant should jointly review the completed technical assessment and the implications of the alternative courses of action. The owner can then combine the technical considerations with the operational and financial considerations to make a decision as to which repair strategy is most appropriate for their situation. The owner must base this decision on all of the available information. It is possible that the repair is deemed to be unnecessary at this time, in which case, the deterioration should be monitored for changes that would signal the need for another evaluation or remedial action by the owner.

It is also possible that, even at this stage, there is insufficient information available to make a decision. Such being the case, the owner and the consultant together should determine what additional information is needed.

STEP 7. DESIGN REPAIRS

If the repairs are relatively simple and the owner is able to develop a specification and contract documentation for the needed repairs and administration of the repair contract, the owner may elect to proceed without the assistance of the consultant. In most cases though, a consultant will be retained to develop the technical design of the repairs, including the selection of the repair materials, the description of the techniques to be employed to implement the repairs and the locations that the various repairs are to be performed. The owner and the consultant together should agree on the form of contract that would best suit the proposed repairs.

Two forms of contract that are commonly used are the *Stipulated Price* contract and the *Unit Price* contract. The Stipulated Price contract bases payment on a lump sum value for the repair work. This may seem attractive to owners; however, it demands that the scope and extent of the repair work be very well detailed in the technical specifications and drawings for the repairs; such is not always possible for repair of existing structures that have hidden conditions. The Unit Price contract bases payment on the actual amount of repair work done to correct the deterioration. This allows the total cost for the work to be adjusted to accommodate hidden conditions. This type of contract demands more on-site review of the work by the consultant to assess the extent of the work needed and to be paid for under the Unit Price contract.

8. PERFORM REPAIRS

Once the specifications and the contract documents are prepared, the owner or the consultant request bids from contractors to do the specified repairs. Contractors should be prequalified to do the repair work. This means that, desirably, only contractors having experience with repair work of a similar scope and type should provide bids. Once the bids are received, the owner and consultant should jointly review the bids and select a contractor. Then, the owner and the contractor formalize the selected contract and the repairs can begin.

The owner and the consultant each have duties and responsibilities during the repair work concerning payment for work done and protection of lien rights under the Construction Lien Act.

Professional consultants are required, under provincial Acts, to review the repair work in process for general conformance with their design. Frequent review by the consultant that designed the repairs can also resolve technical issues that arise during the repair process and verify the quality of the repair work, as it is being performed.

OWNERSHIP AND FINANCING OF REPAIRS

One of the issues that must be addressed jointly by the owner and the consultant is the method of financing the capital cost of the repair work. In this regard, owners of different types of property have different avenues available to them to obtain and recover the costs for capital expenses. Figure 3-3 summarizes the various alternatives generally available to four primary types of owners, i.e., private, co-operative housing, condominium and public non-profit.

Financial Option	Type of Owner: Private	Type of Owner: Co-operative Housing	Type of Owner: Condominium	Type of Owner: Public Non-Profit	Considerations
Recovered from Rent	❍				- Possible rent control legislation - Rental market conditions
Mortgage Refinancing	❍	❍			- Equity position - Taxable income reduction
Special Assessment of Owners/Shareholders		❍	❍		- Financial situation of owner/shareholders - Housing market
Replacement Reserve Fund		❍	❍	❍	- Fund balance - Assignment of funds to repair item
Capital Improvement Budget				❍	- Availability of funds
Operations/Capital ExpenseTransfer	❍	❍	❍	❍	- Impact on cash flow

Figure 3-3: Ownership and Financing Options

REPAIR FREQUENCY AND ASSOCIATED COSTS

Appendix F provides curves that describe the condition of concrete that is deteriorating over time due to environmental factors. Deterioration of a parking garage floor slab due to combined delamination and scaling could be described by such curves. Figure 3-4, below, is a simplified version of one such curve depicting deterioration over time.

Initially, the loss of structural strength is of little concern since there is a large margin of safety between what loads the structure can resist and those that it experiences. Later, as deterioration progresses, the risk of structural inadequacy increases and, finally, the structural element could fail. Of concern to owners as the deterioration progresses, is not only the risk of failure, but, that the rate of deterioration is very likely increasing as time passes.

At some time, between points B and C on Figure 3-4, the owner must take action to correct the deterioration. That point would be the threshold, beyond which, the concrete no longer meets the owner's minimum desired level of performance. Factors that determine the minimum desirable level of performance vary among owners, but can usually be ascribed to elements of the owner's concerns shown in Figure 3-1.

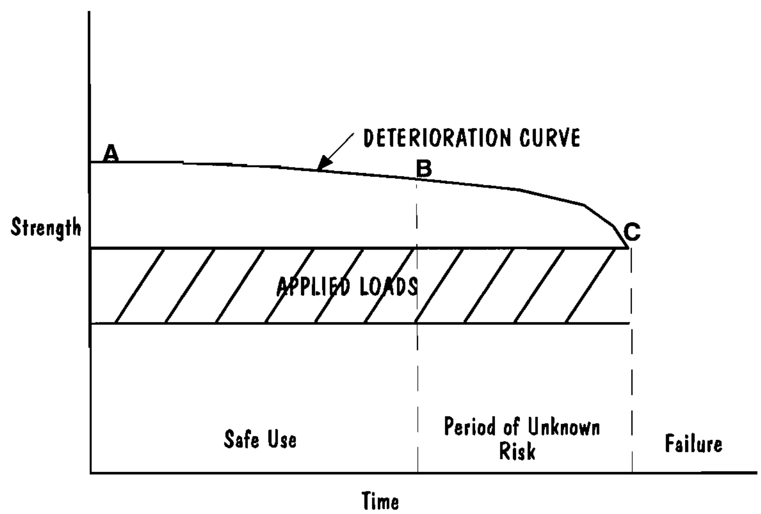

Figure 3-4 Reduction in Resistance to Deterioration over Time

If the desired level of performance is close to that of the original construction, the owner will likely implement maintenance and repairs at frequent intervals, thus, retaining the desired high level of performance. On the other hand, if the owner either can not or does not wish to maintain the concrete at or near the initial level of performance, repairs will be less frequent. Figure 3-5 depicts such a scenario. Owner A, for example, maintains the concrete in a condition closer to its initial condition and performs adequate maintenance and frequent repairs to maintain the state of the concrete above the minimum desired level of performance. Owner B, on the other hand, does not maintain the concrete as frequently, perhaps because of financial impediments or because Owner B does not demand the same high level of condition and performance.

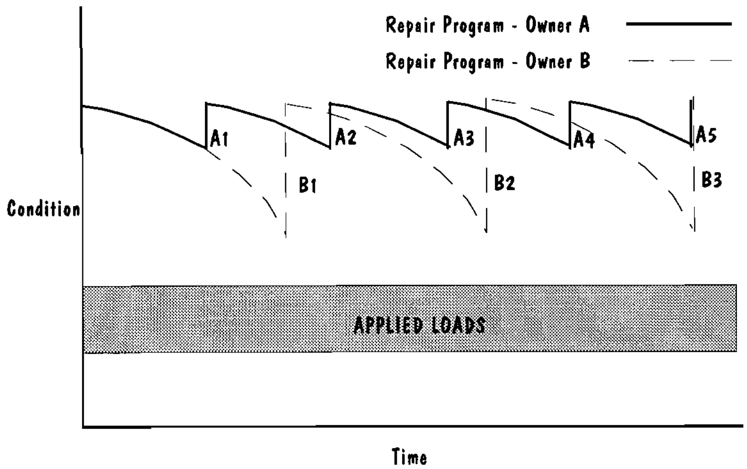

Figure 3-5 Alternative Repair Strategies

In this example, Owner A repairs the concrete five times and Owner B repairs the concrete three times over the same period. Presuming that there is not a structural concern in either case, both repair programs are acceptable ways of managing the technical repair concerns. However, regardless of the tolerance that Owner B may have for concrete in poor condition, it may make financial sense for Owner B to effect repairs more frequently. If, due to the accelerating rate of distress, the cost for Repair Program A (A1+A2+A3+A4+A5) is less than the cost for Repair Program B (B1+B2+B3), Owner B should perform work more frequently. Life-cycle cost analysis would demonstrate whether or not it makes economic sense to do the repairs as Owner A would or as Owner B would.

CHAPTER 4 REPAIR AND PROTECTION TECHNIQUES

The development of repair materials for concrete has been remarkable over the last decade. Historically, concrete was repaired by removal of the affected area and replacement with similar concrete. At present, the repair material products available for concrete are several hundred in number and include crack repair materials, specialty cements and repair systems with reinforcing fabrics, meshes or fibres. The majority of the most-used products, however, can be broadly grouped by their mode of use into *replacement materials, crack fillers and adhesives,* and *surface stabilizers.*

The following discussions of the three categories of the noted repair materials give some insight into their use and limitations. However, it is first important to understand that, regardless of the repair product used, there is a difference between the work required for repair and patching the surface of a concrete structure and the work required to strengthen or restore strength to the structure.

STRENGTHENING AND PATCHING

Addition of structural strength can be accomplished without necessarily involving the restoration of deteriorated materials. For example, restoration of the strength lost by a deterioration of a reinforced concrete beam may be best accomplished by addition of new adjacent steel or concrete beams. On the other hand, repair of materials affected by deterioration should not be considered to automatically restore strength lost due to that deterioration. For example, deterioration of concrete around the columns of a parking garage structure, caused by corrosion of the reinforcing steel reduces the strength of the slab at the column. Simple repair of the concrete by removal of the delaminations and patching does not assure that full structural strength is restored. One can not generalize about recovery of strength loss; however, most engineers are not alarmed by the notion that some strength loss exists if the repair techniques used involve only patching. Each structural deterioration situation must be examined individually by a structural engineer, skilled in the repair of concrete, to determine the contribution to the strength of the building element being made by the repair.

It is generally recognized that the bond strength between the old or substrate concrete and the new patch concrete is crucial and that durability of the bond is an issue that requires further exploration[32].

REPLACEMENT MATERIALS

Durability problems that take the form of surface scaling, spalling, freeze-thaw damage or damage due to dehydration or from excessive heat (e.g., a fire in the building) can be repaired by removal of the weakened or broken volume of concrete, to expose the sound substrate and, subsequent application of a repair material suitably bonded to reform the original lines, grades and levels of the element. The goal is to replace the damaged material with another material with adequate properties to be able to withstand the ongoing detrimental effects of the environment that caused the element to suffer the initial problem.

One of the key concerns related to durability of the replacement material is the possibility of differential shrinkage between the old concrete and the new patch. Depending on the environment and requirements of the exposure, several types of replacement products are available, some with special procedures for placement, bonding or curing. The more commonly employed types of materials for replacing concrete are discussed below.

Normal Portland Cement Concrete Patches

As the name implies, this repair material is simply Portland cement concrete placed to provide original profiles. One of the most important factors determining the success of the repair is the quality of the bonding between the existing and the repair concrete. In order to provide sufficient moisture for hydration, the existing concrete is wetted and a slurry of *neat* paste (cement and water) is used as a bond coat. This also ensures that all voids in the roughened substrate are adequately filled. The fresh concrete patch mixture is then placed on the moist, tacky bond coat using standard concrete construction procedures. Standard finishing and curing procedures are also employed.

Mix proportions should be selected to produce patches that will have characteristics similar to the existing concrete (i.e., if the strength of a slab, as determined by compressive strength tests on cores, is 25 MPa, provide 25 MPa strength concrete in the patches). It is a misconception among many owners and some contractors that the stronger the concrete in the patch, the better the patch will be. This is not so. Matching the properties of the repair concrete to the substrate will provide for similar behaviour and reduce the possibility of bond failure and cracking of the patch.

Latex Modified Concrete

The addition of a suspension of synthetic rubber compounds or latex, to concrete mixtures improves the impermeability and elasticity of the mix over that of normal Portland cement concrete. To be most effective, the latex should be non re-emulsifiable, that is, unable to return to solution. These products are usually styrene butadiene or acrylic-based and are provided in a suspension having 47 to 50 percent latex solids by weight in water. For best results in thin cross-section applications (less than 25 mm), the ratio of latex solids to cement (known as the solids-on-solids ratio) should be 15 to 20 percent. In the mixture for latex modified concrete, a portion of the mix water is replaced by the latex emulsion. Lesser concentrations of latex solids are usually provided in proprietary mixes.

Some latex modified mortar (concrete excluding coarse aggregate) applications include a fibrous reinforcement either as a woven fabric to be sandwiched between layers of latex modified mortar or as discrete glass or polypropylene fibres from 20 to 40 mm long. These fibres redistribute stresses that can result in cracks in the repair material.

When using latex modified concrete, it is preferable to employ a mixture of Portland cement and latex as a bonding agent. Placing, finishing and curing should follow practices recommended by the supplier of the product. Latex has the ability to entrain air into the mixture, sometimes to excess. Since excessive amounts of air will reduce the strength of the mix, it is required that latex suspensions be prepared with a defoaming agent in sufficient concentration to limit the entrained air to 6 percent.

Epoxy Modified Concrete

Epoxy modified concretes are principally supplied as proprietary products. Their performance depends on the amount of epoxy introduced into the mixture. Such mixtures look and behave very little like concrete when compared to the Portland cement and latex modified replacement materials. Since the cementitious material may be almost wholly replaced by epoxy, epoxy modified concretes are by far the most impermeable of the replacement materials. They are rarely employed as a general replacement material, though, as these products tend to be costly. The most common use is small patches on vertical or overhead surfaces.

MAGNESIUM -PHOSPHATE CEMENTS

Normal Portland cement is a calcium-silicate cement that cures by way of a reaction with the mix water. Proprietary products employing calcium-magnesium cements also cure by reaction with water; however, the reaction is much faster - as little as one hour - after which the repaired element may be put into service.

The exceptionally fast set exhibited by these products is accompanied by a highly exothermic reaction. Thus, thin sections are susceptible to shrinkage and cracking. Cracking due to high heat gain is also possible for thicker sections. These products have best success in replacement patches having relatively regular plan dimensions. Because they have fast setting properties, these products can be used to repair spalls in parking garages that can not be out of service for the extended periods of time needed for setting and curing of normal Portland cement patches.

CRACK FILLERS AND ADHESIVES

Virtually all crack fillers and adhesives are of a proprietary nature. Work with these materials should be restricted to specialty trades licensed or approved as applicators of the material by the supplier. The use of these products is either to fill cracks in an effort to seal the opening or glue the sides together, wherein the product is an adhesive.

CRACK FILLERS

To allow for thorough mixing and ease of placement, fresh concrete always has far more water than needed to complete hydration of the cement. Eventual loss of the excess water through drying causes the concrete to shrink. This shrinkage is often compounded by thermal effects and if such shrinkage is restrained, the concrete cracks. While these cracks are generally not significant to the serviceability of the element, in a moist environment they can result in unwanted water leaks, leaching and efflorescence. Sealing of the cracks using injected materials has become common and is often successful in eliminating water penetration into the crack. The crack filler is injected into the crack through an adjacent, drilled injection port (Figure 4-1).

The crack filler materials can be grouped into two categories: *chemical crystalline,* and *chemical gels*. Both require water as a catalyst to initiate the process which seals or fills the crack.

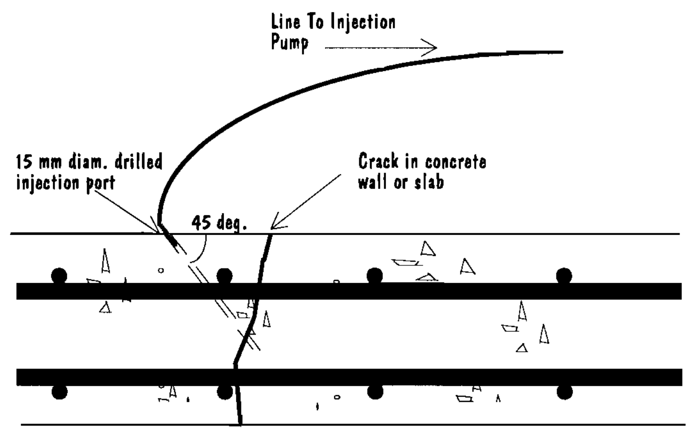

Figure 4-1: Injection at Crack

Generally, the use of the crack fillers is determined by the nature of the crack. If the crack continues to open and close due to thermal or load related forces, it is best to use a sealant that can accommodate the movement. These would include the chemical gel type sealants that remain somewhat flexible. If the cracks are not expected to open and close, the more rigid chemical crystalline sealants may be sufficient to stop water ingress.

The *chemical crystalline* type must be prepared as a paste and implanted into a recess created in the crack. The seal is generated by growth of crystals to fill the crack. Water leakage may recur at such sealed cracks until sufficient further crystal growth takes place. Subsequent widening of the crack again allows water penetration. Thus this method is not recommended for situations where no further water ingress is permissible.

The *chemical gel* type of crack filler is generally based on polyurethane or acrylic formulations that expand when in contact with water. These materials are injected into cracks and upon reaction form a flexible sealant within the void space. If water is removed entirely, these materials can dry and shrink and allow water to by-pass the seal perhaps years after injection. They are, however, able to accommodate moderately small movements in the crack width - depending on the initial crack geometry.

ADHESIVES

For the most part, adhesives used for concrete repair are rigid epoxy formulated for this application and the pressures under which the injection will occur. Epoxy is non-elastic, and thus, does not move with moving cracks, but in its capacity as an epoxy, does provide adhesion to the crack walls far superior to that of any of the crack filler materials. The epoxy formulation to be used depends on the location and orientation of the crack (i.e., wall, floor, column, the depth of the crack and its width) and should be so specified.

Some epoxy adhesives are essentially self-leveling and can be employed in floor cracks; whereas, some are much more viscous and more suited to injection under high pressures or into wider wall cracks. The selection of the epoxy must be matched to the orientation of the application and the characteristics of the crack to be filled with adhesive. Failure to properly fill a crack will reduce the effectiveness of the repair attempt to restore the strength of the element before the crack occurred.

In the majority of cases, the cracks requiring adhesive injection are the result of a structural overload. Adhesives should not be used to seal moving cracks against water ingress. Attempts to do so often fail, either because the adhesive fails or because a crack forms in the concrete again at another location. Ability to stop water ingress is, thus, a secondary effect rather than a requirement of adhesive injection.

SURFACE STABILIZERS

Occasionally, owners may wish to reduce the permeability of concrete or make it more resistant to chemical attack or weathering but do not want to change the appearance of the concrete, as would happen if a membrane was applied. In such cases, a surface stabilizer may provide the needed protection.

The term surface stabilizer refers to a material that penetrates the surface of the concrete and forms chemical bonds. Two types of products are discussed, representing the broad range of available products: *micro silicas* and *impregnated polymethylmethacrylate.*

Micro silica is an extremely fine silicate mineral product that, when applied to the surface of concrete, can react with the calcium compounds to form new calcium silicate minerals. Due to the fineness of the micro silica, it is capable of filling the interstitial spaces within the cement gel, thus decreasing its permeability and strengthening the surface. Commercially available products employing micro silica have been used for only a few years, thus their long-term performance is untested.

Polymethylmethacrylate (PMMA) impregnation can be used to slow the damage to ornamental concrete caused by acid rain or other aggressive chemicals. It is, however, rarely performed on buildings due to the cost and complexity of the process. Impregnation requires drying of the concrete substrate using heat prior to application of PMMA over the surfaces. The PMMA is drawn into the concrete pores to fill the voids, thus densifying the surface layer. Some chemical bonding occurs between the carboxylate of the PMMA and calcium of the free lime in the cement matrix, thus further decreasing permeability.

While each of the above product types will reduce permeability, none of these surface stabilizers should be considered to be waterproofing materials.

Material Selection and Repair Design

The performance and durability of repair materials directly follow from the matching of the repair to the cause of the failure, since, without such careful design and selection of materials, the repair is likely to fail. Accordingly, understanding the primary cause and mechanism of failure and the structural significance of each situation must precede the selection of any repair material or the repair effort will be frustrated.

The basic technical premise behind any repair undertaking is that the repair should last as long as the element to which it is applied. Selection of the appropriate repair materials for the application, therefore, must be based on the desired properties of the repair, i.e., strength, adhesion, resistance to abrasion, movement capability and time available to perform the repair. Once the repair material has been selected for the application, the method of application likely governs the durability of the repair.

Factors that can be controlled in the design of the repair are:

- removal techniques
- repair geometry
- surface preparation
- bonding

These parameters are discussed as follows:

Removal Techniques

Procedures for removal of the deteriorated or failed concrete can directly impact the durability of the repair. The two most popular techniques involve the use of chipping hammers and the use of high-pressure water to remove the unsound materials. It should be noted that either technique can be successful if correctly applied.

Chipping away of unsound concrete must be performed with the correct amount of energy applied by the hammer at the various depths of removal. Lighter hammers should be used toward the final stages so as not to further damage the substrate concrete to which the repair material must be bonded.

High-pressure water removes both sound and unsound concrete depending on the water pressure at the instant of impact with the concrete. It is more difficult to control than removal by chipping hammers, but typically provides a rougher texture to the surface that contributes to bond strength. High-pressure water jets can also be used to remove large areas of uniform thickness. If considering high-pressure water removal of concrete, the overall design of the repairs must accommodate drainage of the water from the repair areas and filtering of the water after use as it may contain fine suspended particles of a size that will not readily settle out in holding tanks and would not be permitted in municipal sewers.

REPAIR GEOMETRY

Of particular importance to the patching of cracks and larger surface areas is the shape of the edges of the repair itself.

A process of "veeing" out cracks to receive the repair compound provides a far less reliable repair shape than a "square" or "dovetail" configuration. The principal difference between the two shapes lies in the angle of the sides of the repair; the 'V' shape offers no resistance to popping out of the crack repair material whereas the shape of the 'dovetail' shape offers both resistance to loosening of the patch material from the parent material and a greater surface area for the bonding of the patch material.

The process of "feather edging" of patches also generally results in progressive edge failure of the very thin patch material. A quite successful procedure (Figure 4-2) allows for a 90 degree shoulder at the edge of the repair at least $1^1/_2$ times as deep as the largest aggregate in the patch and in no case less than 10 mm deep. The vertical edge of the repair area is accomplished by saw cutting prior to chipping and the remaining removal accomplished by chipping. If high pressure water cutting of the concrete is used, the edges can frequently be cut sufficiently vertical as to prevent feather edges in the repair concrete.

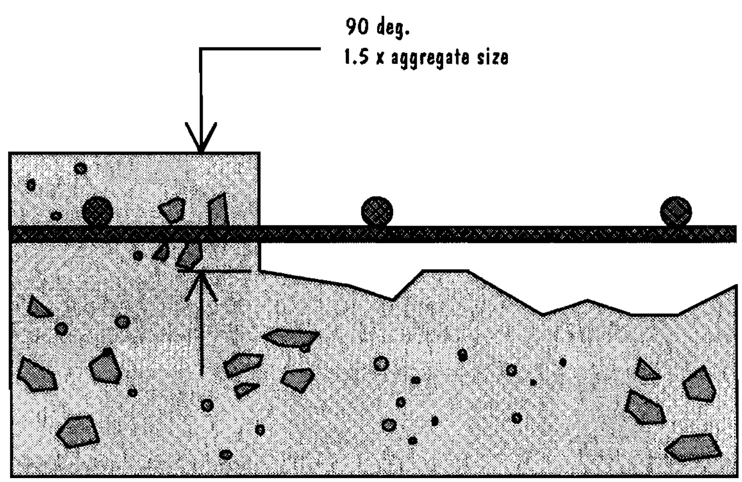

Figure 4-2- Shoulder Detail in Concrete Removal Area

SURFACE PREPARATION

Surfaces to which a repair is to be applied must be clean, sound and compatible with the intended repair material. At all times, consideration must be given to the nature of the repair material. Cleaning is usually accomplished using high pressure water, chemicals or abrasive blast as is appropriate for the circumstances. Usually water will clean the surface sufficiently; however, more stubborn surface contaminants may require the use of detergent or chemicals. The detergent should be selected on the basis of the contaminant to be removed (for instance, oils can be removed by detergent). Cleaning by use of water and detergent may be performed if bonding is required on a previously weathered and dirty concrete surface. The detergent should be thoroughly rinsed away or it may contaminate the patch concrete, reducing its strength or impairing the bond of the patch.

Chemicals, such as acids, actually disintegrate the concrete surface on which they are applied. Flushing of the chemical with water is required as chemicals will reduce the strength and bond of the patch material. Acid solutions should not be used where reinforcing steel corrosion is of concern and, in particular, never on post-tensioned structural concrete.

Abrasive blasting, which uses either a mineral grit or small steel shot, removes loose surface particles and the cement matrix around aggregate thus removing the soiled concrete surface. Abrasive blasting can create substantial dust so areas receiving this form of cleaning should be isolated by tarpaulins. There are also health risks associated with some abrasives and the dust they create.

BONDING

Bonding of patch materials to the substrate may be accomplished either by using adhesives, or by relying on properties of the patch materials. A bonding agent might provide better results as it fills the interstices in the parent concrete and provides a transition zone to the repair material. The preparation of the surface prior to application of the bond material is critical to the strength and long term durability of the bond. The substrate concrete should be wetted to prevent it from sucking water out of the bonding agent, as this will reduce the bond strength. However, the patch substrate should not contain free-standing water, as the bond is also reduced by absorption of water into the repair material from the substrate. If a cement slurry is used, it should have a water-cement ratio of 0.40 to 0.50, generally a creamy consistency is correct, and it should be vigorously scrubbed into the substrate concrete to ensure that all voids are filled.

Without a bonding agent, such as cement slurry, latex-cement slurry or epoxy, there is a risk of the patch material failing to knit with the substrate. This knitting together can also be adversely affected by improper use of the bond material, for instance, allowing epoxy to become tacky or latex slurries to film before applying the patch material. Manufacturer's directions for use should be followed when using proprietary materials.

Research has found[32] that bond durability is a function of many parameters and that there currently exists no acceptable test to assure long term bond strength in all circumstances. Use of bonding materials must consider the repair strategy as a whole. For instance, if cathodic protection is contemplated, epoxy bonding agents should not be used as they are non-conductors of electricity and would interrupt the distribution of the current to the reinforcing steel.

SUMMARY OF TYPICAL REPAIRS

The nature of the repair selected will depend to a large extent upon the type of concrete distress and its significance to the deterioration of the structure. Figure 4-3 summarizes the types of repairs that are typically performed for the various types of distress observed.

● Usual

○ Occasional

		Significance							Remedial Action													
Distress	Location	Aesthetic	Minor Serviceability	Major Serviceability	Minor Structural	Major Structural	Monitor	Seek Expert Advice	Do Nothing	Clean	Surface Patch - Normal Portland Cement	Soffit Patch - Normal Portland Cement	Polymer Modified Mortar <25 mm	Rout and Seal Cracks	Epoxy Inject Cracks	Chemical Inject Cracks	Strengthening	Seal Surface	Waterproof Surface	Replace Component	Excavation of Overburden	Formwork and Shoring
Scaling	Suspended slab	●	○						○				●					●	●		○	
	Column, Wall	●			○				○		○		●					●				
	Slab on grade	●	○						○				●					●				
Disintegration	Suspended slab	●		○		●		●												●	●	●
	Column, Wall	●				●		●												●		●
	Slab on grade	●		○		●		●												●		●
Wide Crack	Suspended slab				○	○		●						○	○		○					
	Column, Wall	○			○	●		●			○				○		○			○		
	Slab on grade	●	●	○			●		○					○			○					
Narrow Crack	Suspended slab				○		●		○					●								
	Column, Wall				○	○	●		○													
	Slab on grade	●					●		●								○					
Leaching and Efflorescence	Suspended slab	●			○		○			●								●	●		○	○
	Column, Wall	○					○			●						○		○	○			
	Slab on grade	○					○		○	●												
Delamination and Spalling	Suspended slab			●	○	●		●			●	●	●				○	○	●	○	○	○
	Column, Wall				○	●		●			○		●				○	●		○		
	Slab on grade	●		●					○		○		○							○		

Figure 4-3 Remedial Work

It is important to note that the repair, for a particular situation, may differ from that identified in this summary, due to specific site conditions. For instance, it may not be wise in all cases to apply a surface sealer to concrete that is exhibiting efflorescence (the white deposit left after evaporation of moisture containing dissolved salts from the concrete) as the source of moisture may not be external to the sealed surface. Moisture may be migrating due to capillary tension or vapour pressure from behind or below the concrete element, such as may be the case of a slab on grade. Application of a sealer under these conditions would trap the salts under the sealer and cause the surface to spall.

PROTECTION AGAINST FUTURE DISTRESS

The concrete, now repaired, will often require protection against the aggressive environment that first initiated the distress. Figure 4-3 identifies protection in the form of application of sealers or waterproofing for structural components that have exhibited: scaling, delamination, spalling, leaching or efflorescence. These additional treatments are required to prevent moisture from penetrating the concrete, thus reducing or eliminating the possibility of reoccurring distress. Vertical surfaces, as well, should be protected after repair, for the same reason. To protect columns and walls from salt and moisture splashed onto the surface or piled up as salt laden snow, the bases of these elements are often coated with a sealer at the time slabs are sealed or waterproofed.

WATERPROOFING MEMBRANES

Waterproofing of the surface of slabs subject to the deteriorating effects of moisture and salt not only prevents the ingress of moisture into the slab but also prevents leaking through cracks that penetrate through the slab. CMHC-sponsored research[19] suggests that concerns involving the effectiveness of waterproofing over a salt contaminated deck are unfounded.

Wearing courses applied to membranes are typically system specific. Hot-applied asphaltic membranes have asphaltic wearing courses and cold-applied elastomeric membranes have any of a variety of polymers. The polymers used by the different suppliers often incorporate fine sand as a grit for traction; however, the wearing course over an asphaltic membrane could be mastic asphalt, latex modified asphalt or conventional asphalt, depending on the system.

Six elastomeric waterproofing systems were tested in a research program sponsored by CMHC[33]. Membrane performance varied when subjected to a series of twelve different tests. While all membranes resist water ingress, resistance to heat, antifreeze and ultra-violet radiation is a concern for some of the membranes tested, as is low temperature crack bridging and elongation properties. Further, the porosity of the concrete substrate seems to have a significant affect on adhesion of the membranes, with higher porosity resulting in reduction of adhesion strength.

The opinion presented in the research noted above[33] and in the author's opinion, considerable efforts are required to develop meaningful standards for the waterproofing systems applied to parking decks. Presently, the thickness of membrane, the use of reinforcing materials and the overall performance criteria varies considerably between the various systems. This leaves the consultants and the property owner unclear as to the long-term performance of thin, elastomeric waterproofing systems.

Selection of the waterproofing system should consider the use of the structure. For instance, additional tear resistance may be desirable for applications of waterproofing in smaller, residential garages with tight turning requirements. On the other hand, larger garages with long aisles and heavy traffic areas should have wearing courses with high resistance to abrasion and wear. If the slab is heavily cracked, the waterproofing and traffic topping should be selected on the basis of their combined ability to span cracks.

SEALERS

Like waterproofing systems, there is presently a lack of standards that would assist owners in selecting the appropriate sealer for their garage; thus, particular care must be exercised in evaluating the specific product against the application.

Most of the sealers used for concrete are coatings that do not penetrate to any appreciable degree. Those that do penetrate the surface more than 2 to 3 mm are called "penetrating sealers". Some sealers employ silane and react with the cementitious matrix to a significant degree. These products are capable of penetration to a depth of roughly 2 to 3 mm or more depending on the initial porosity of the concrete. There are several commercially available *silane based sealers*. These products are employed principally to control moisture ingress. As all pores and capillaries are not filled by the sealer, some water penetration is possible.

A study of 12 generic types of sealers, involving 62 different products was conducted at the National Research Council [38]. That study reported that the effectiveness of the sealer is not wholly dependent on the product type. There is variation in performance within the generic product group; thus, different silane based products will protect to different degrees. The properties of the concrete substrate also affect the ability of the sealer to perform. Concrete with a high water-cement ratio will not be as well protected as concrete with a low water-cement ratio that receives the same treatment.

Regardless of the product type and quality of the concrete, the effectiveness of the sealer in preventing water ingress is largely related to the time the water stays on the surface; thus, if drainage is good and ponding does not occur, sealers may be an appropriate method of reducing, but not preventing, moisture ingress. There are examples of good performance shown by garages that have employed sealers.

If used as moisture protection for a slab, sealers are most applicable if the slab is free of cracks, as sealers have very limited capacity to span cracks. Post-tensioned structures and pre-stressed structures with good drainage are the best candidates for sealer application. It should be noted that current design standards for parking garages allow the use of sealers on parking deck floors only if the reinforcing steel of the upper mat is epoxy coated and the slab is post-tensioned rather than conventionally reinforced. Precast, pre-stressed designs (uncommon in residential construction) may have a reinforced, bonded topping. The topping may also benefit from a sealer. All conventionally reinforced parking floors require waterproofing.

Cathodic Protection

The application of cathodic protection to suspended parking garage slabs is also available as a means of providing protection against further corrosion-induced delamination and spalling. It is frequently necessary to waterproof the slab in conjunction with the application of cathodic protection or to locally seal leaking cracks to prevent leaching through the slab. Most cathodic protection systems are installed on the underside of the slab and employ a conductive paint in the protective system. Supplementary benefits include the migration of chloride to the anodic coating on the soffit; however, this can also affect the bond of the coating and reduce system effectiveness. A more detailed description of cathodic protection is included in Appendix H of this report.

The use of cathodic protection is not widely accepted as initial costs are high and the application, operating and maintenance procedures are not standardized. There are some concerns about the durability of the conductive coating: leaching through the slab can deteriorate this coating and reduce the effectiveness of the protection. In addition, the cathodic protection process causes the evolution of hydrogen at the steel surface; resulting in hydrogen embrittlement, particularly of post-tensioning or prestressed reinforcement is, thus, a concern. Some testing has also found that reinforcing steel bond strength is reduced[34]. Post-tensioned structures are not suited for cathodic protection.

Cathodic protection systems must be monitored and maintained for the life of their installation and there remain unresolved conflicting views as to the installation practices and performance criteria in this rapidly evolving industry as it relates to protection of reinforced concrete. It is clear, though, that cathodic protection does reduce corrosion of reinforcing steel. It should be combined with waterproofing to prevent water that passes through cracks from damaging the cathodic protection system.

FIELD EPOXY COATING OF EXPOSED REINFORCING STEEL

Epoxy coating of reinforcing steel exposed by removal of deteriorated concrete, is sometimes performed in an effort to electrically isolate the reinforcing steel that corroded and caused delamination. The coating is applied either by spray or brush application. There is no standard practice concerning the in-situ coating of uncoated reinforcing steel prior to placing repair concrete. Some consultants and corrosion specialists claim that partially coated reinforcing steel accelerates corrosion of uncoated reinforcing steel in the adjacent areas. There is no substantial body of evidence to support this claim.

Reasons for not epoxy coating exposed reinforcing bars include: reduced restoration costs; exclusion of epoxy material from the bond area and possible reduction in patch adhesion; poor electrical continuity should cathodic protection be considered in the future; and elimination of the question of reduced adhesion between steel and concrete due to the epoxy. While it has not been the practice of the author to epoxy coat exposed, previously uncoated bars, despite these concerns, there is insufficient evidence to recommend that the practice of epoxy coating be discontinued by others.

PERFORMANCE OF REMEDIAL WORK

There is very little documentation on the performance of remedial work. Generally, in the author's experience, performance is highly dependent of the quality of the design and workmanship and less so dependent on the materials used. This does not mean that certain materials may be inappropriate for use in some circumstances or that certain procedures are appropriate in all cases, but that, by and large, the restoration of concrete follows a set of characteristic procedures using a selection of typical materials. These were described earlier in this section.

There has been some research work done to assist owners and managers in maintaining their structures. A recent study by IRC [19] on the deterioration of parking structures describes the performance of 62 garages monitored over a four year period. The results of that study indicate that the repair of concrete structures requires ongoing attention from building owners and that the commonly employed methods involving patching and waterproofing of garage floors seems to be adequate if care is taken in the bonding of the repair concrete to the existing concrete. The apparent weakness in the practices employed involve the durability of the wearing courses of the waterproofing systems and in lack of standardization in the sealers used.

The report also suggests that while the rate of deterioration of structures continues to vary (0.6 to 5.8 percent per year) in the 62 repaired structures under review, even the parking garages constructed without adequate protection can be kept serviceable, with the addition of appropriate protection, until the end of their design life. Essentially, owners should begin to treat garage structures as assets requiring ongoing maintenance. In addition, annual inspections should be performed by consulting engineers with log books kept of condition and actions taken.

Without question, the repair and maintenance of concrete in buildings requires ongoing attention from building owners. Good repair techniques and materials technology are readily available as is qualified engineering support and financial costing assistance. Prudent owners can use all of these resources to make economically sound and technically responsible choices concerning the upkeep of the concrete in their buildings. Alignment of the cost and performance of the repairs with owner's particular needs should lead to owner satisfaction.

APPENDIX A RESEARCH INTO CONCRETE DURABILITY

RESEARCH AND TECHNOLOGY TRANSFER

To understand the circumstances that can cause lack of durability in concrete and to recognize the symptoms of the deterioration, it is important to know about some of the processes that cause deterioration. To assess concrete with possible problems, it is important to be aware of the tests used to detect often hidden deterioration. To determine what to do about the distress to correct the damage and protect against further occurrence, it is important to consider the alternatives and their applicability in various situations. Thus, the goal of durable concrete has spawned considerable research into the physical and chemical causes of concrete distress, ways of detecting problems, ways of delaying or preventing the onset of deterioration, and ways of providing repair and protection of deteriorated concrete.

This research has been fostered through the efforts of organizations such as the American Concrete Institute (ACI), the Portland Cement Association (PCA and CPCA), the Cement and Concrete Association (CCA) - now the British Cement Association and, in Canada, by Canada Mortgage and Housing Corporation (CMHC), the Canadian Institute of Public Real Estate Companies (CIPREC) and the National Research Council - Institute for Research in Construction (NRC, IRC), formerly the Division of Building Research (DBR). Publications of the research results by these organizations can be obtained at nominal or no cost. Many of these documents are, however, rarely produced in a form useful to building owners and managers. The references and the list of other reading attached to this report include published research by both public and private organizations, selected papers, conference proceedings and text books. This literature is by no means complete, but, it does provide a reasonably good overview of the study of concrete performance in buildings.

There has also been a vast amount of knowledge gained through the individual efforts of building owners, restoration contractors and consultants having experience with building restoration. Unfortunately, the information learned by these segments of the industry is also not produced in a form that is readily accessible to building owners and managers. In some cases, the information is of a "non refereed" nature; that is, it has not been reviewed for technical accuracy or bias. It may also be incomplete.

Consequently, while there are a good number of organizations undertaking the needed research work, owners and managers of buildings are generally unaware of the findings and thus rely upon their consultants and contractors to advise them as to the best approach to their particular problem.

The transfer of knowledge about concrete durability technology from the research community to the users is a fairly large step. It requires that the science and technology produced in separate papers and studies be reduced to a form suitable for practical application. With regard to design and construction, this has properly been the responsibility of the standards and code writing bodies who, with input from a wide cross section of stakeholders, put down in formal, simple terms, the requirements for materials, design and construction that are known to result in safe, durable construction. In Canada, the development of these codes and standards is a fairly involved process and one that should be understood to fully appreciate the problem of translating research findings into buildings that perform. Appendix B describes the initiation of building codes and standards and the purpose of these in the construction industry. Building owners and managers, however, are less concerned with the requirements for new construction than the requirements for maintaining the existing structures. Fortunately, momentum is building in research and standards writing organizations to accommodate these maintenance concerns.

Building owners who become familiar with the various aspects of building *Production* and *Use* (Figure 1-1) can take better control of the short term and long term costs for building repair and maintenance. In the long term, while the owners and managers of buildings typically don't have direct input to design or construction of buildings, the information that owners and managers possess about the performance of their buildings should be considered by the design community as vital to the development of new buildings. By introducing their experience about concrete performance through studies of deterioration and rehabilitation, building owners and managers could help to shape the technical requirements of new concrete construction. This type of feedback would be very valuable to building owners in the future.

BRIDGING THE GAP BETWEEN RESEARCH AND PRACTICE

EARLY INITIATIVES

Chloride-ion induced corrosion of steel in concrete is a phenomenon well known to researchers and to some designers for many decades. The potential broader benefit of the findings of research initially undertaken by the highway authorities in Canada and the United States of America has, however, to some extent, been limited because it was not seen to be applicable to buildings. The parking garage problems developed because of a misjudgment in the design and code writing community as to the impact of salt brought into garages by vehicles and the increase in the use of salt by those responsible for maintaining roads. As such, the codes and standards influencing the production of concrete garages did not keep pace with the factors influencing their use. Today, the exposure and thus the processes resulting in deterioration of parking garages is acknowledged by most designers to be similar to that for bridge decks.

In an effort to expedite the introduction of needed improvements to the general design and construction community, researchers and others who have been investigating performance problems and potential remedial action, have published articles and papers with information and recommended courses of action that could be voluntarily adopted by the construction industry. In some situations, public bodies have required quick corrective action of a mandatory nature. The corrosion-induced deteriorating condition of parking structures designed prior to the early to mid 1980s, for example, was of such concern that CMHC in 1984[1] and the Ministry of Housing in Ontario in 1988[2], each published design requirements pertaining to garage structures designed under their purview. To its credit, the Ontario Chapter of the American Concrete Institute (ACI) produced a document on the relevant aspects of deterioration, design and construction of parking garages as early as August 1981. These interim standards were often adopted by private developers in an interest to protect their investment and reduce liability in the face of an ever increasing litigious environment. To some designers and builders, these interim requirements, which included such current recommendations as epoxy coated reinforcing steel, improved slope to drains, waterproofing and air entrainment of the concrete, were perceived to be an over-reaction to the problem. These recommendations have, however, proven to be effective in preventing the onset of deterioration of the garage concrete. The overall added costs of these protective measures were not large, amounting to an initial capital cost of 8 to 10 percent more than the traditional practices. When compared to the life-cycle cost for repairing the traditionally designed and constructed garages, which, in some cases, involves replacing the garage floor slabs, the cost increment becomes even less significant.

The problems experienced by parking structures in Canada were by no means unique. The corrosion of steel in concrete, the possible use of corrosion inhibitors, improvements to concrete quality, required cover over the reinforcing steel and the effects of permeability, concrete alkalinity and chloride content on the rate of corrosion were under investigation through the 1960s by the Portland Cement Association (PCA)[3]. These early papers were followed by research published by the Federal Highways Administration (FHWA) in the early 1970s that addressed the effects of mix design and construction on corrosion[4], the use of electropotential data in assessing corrosion potential[5], types of protection necessary against salt[6] and possible use of galvanized steel[7].

BORROWING FROM HIGHWAYS AND BRIDGE EXPERIENCE

The (then) Ministry of Transportation and Communications in Ontario (MTO) was also actively researching the nature of the distress caused by corrosion. Corrosion protection requirements, including epoxy coated reinforcing steel, increased concrete cover, higher quality concrete and thicker structural members, were introduced in the 1970s and again reviewed in 1986[8]. Their research documents show that, as early as 1963, MTO was identifying deficiencies in concrete bridge decks beneath the asphalt pavements, the most serious of which was not corrosion, but scaling. This led to the practice of constructing bare concrete decks from 1965 to 1972 using air entrained (freeze-thaw resistant) concrete so that the decks could be inspected more readily. However, by the early 1970s the corrosion of reinforcing steel and the major rehabilitation costs associated with repair was becoming obvious. The MTO investigated waterproofing membrane systems through the rehabilitation process and in 1972, air-entrained high quality concrete with waterproofing under asphalt wearing courses became the standard for new bridge decks in Ontario. MTO concluded from their experience that there was a need to pursue a more conservative approach to the design and construction of concrete exposed to harsh environments, and in particular, Ontario's bridges.

As a consequence of the research by MTO and FHWA, results have been made generally available. These results continue to make a significant contribution to the understanding of concrete deterioration and repair in buildings and in the development of standards and guides for design and construction of concrete for buildings.

CONCRETE DETERIORATION IN BUILDINGS

In the late 1980s, with new design standards in place for parking garage construction[9], the focus of attention on parking structures turned to the existing stock of building structures. The building booms of the 1960s and 1970s created a new type of high-rise housing stock. To increase density, new residential apartment buildings were built taller, ranging into 20 to 30 stories or more, in some cases, and parking garages became multi-level structures incorporating as many as three or four structural parking decks. Since the parking decks below grade were considered to be protected from exterior conditions, they were generally constructed without resistance to freeze-thaw or road-salt induced corrosion damage. The residential towers of the 1950s, 1960s and 1970s were also typically designed to have exposed balcony slabs that were extensions of the floor slab concrete. These floors added to the area of unprotected structural concrete unwittingly exposed to deterioration. The condition and thus value, maintenance and rehabilitation needs of these buildings was a growing concern, particularly to CMHC which was in the position of having financed or insured mortgages on many of these properties. The problems of structural deterioration of parking garages began to dominate the research related to concrete through the 1980s, including that performed by CMHC.

The scale of the problem of delamination, leaking and structural decay of garages was known to be enormous; however, the decay rate and the predictability of the degree of structural strength loss was (and remains) a conundrum. One of the first documents to appear that addressed the scale of the concrete deterioration problem was a survey of 84 parking structures in Ottawa. A CMHC report published in November 1981[10] determined that those 84 structures alone represented approximately $10 million in repair costs in 1981 dollars.

Ongoing maintenance was a concern addressed by the Parking Consultants Council of the National Parking Association that published a "pocket-book" format manual in 1982[11]. The manual included a recommended maintenance list together with a discussion of the 15 items to be checked on a routine basis.

In December 1984, CMHC expanded this survey sample from 84 to 300 parking structures, including the Montreal, Ottawa and Toronto areas[12]. Elements of the design and construction of the garage structures, such as expansion joints, protection against corrosion, drainage, concrete quality, etc. were assessed in relation to the durability of the garage structure. The sample study found that design practices including good drainage, crack control, high quality expansion joints, adequate concrete cover and waterproofing improved durability. Similarly, proper construction practices which prevent cracks, allow for good drainage and maintain concrete cover over reinforcing steel sustained durability. For guidance, maintenance procedures (reprinted from the Parking Consultants Council document of the National Parking Association) were attached to that CMHC document.

A third study by CMHC published in March 1986 summarized the causes of deterioration of parking garages, the relevant investigation procedures, repair strategies and maintenance procedures[13]. That report was among several synopses of concrete deterioration, investigation and repair that were available in the mid 1980s; however, most of the available references dealt with bridge structures. The CMHC-sponsored research, though, was geared to the evaluation of the deterioration phenomenon in concrete directly related to its use in buildings, those structures in which CMHC had a vested interest in long term performance of the concrete and suitable retention of property value.

In the late 1980s, the scope of research conducted by CMHC began to extend beyond the issue of parking garage deterioration to include the other exposed concrete in buildings. In Europe and Australia, widespread damage due to carbonation of the concrete was a recognized problem; whereas, in Canada, carbonation had received very little attention. CMHC commissioned studies aimed at assessing the risk of deterioration of exposed concrete walls, balconies and parapets of high-rise buildings. The first report, a state-of-the-art literature review published in April 1987, determined that ambient relative humidity in the 50 to 75 percent range would likely be more conducive to carbonation[14]. The second report, in January 1990[15], examined carbonation at 28 buildings in Toronto, Ontario (one of the cities considered to be at risk). That study found balconies appeared to be the most vulnerable to carbonation, whereas vertical elements such as walls were of less concern. The third report [16], in August 1992 described a method of test that correlated the rate of water absorption to carbonation depth.

CMHC sponsored a survey [39] of buildings in Halifax, Calgary, Edmonton, Vancouver and Victoria found that the characteristics of the concrete element, such as the poor cover of concrete over the embedded metal, vertical orientation of the element, intermittent exposure to wetting and, proximity to combustion products, all contribute to the rate of carbonation. That study also found that, while phenolphthalein tests for depth of carbonation are useful and cost-effective, thermogravimetric testing (a direct measurement of the lime and carbonate content of the concrete) found that the carbonation 'front' in the concrete may actually be 10 to 20 mm deeper than the phenolphthalein indicates. The research to date on carbonation indicates that it does not appear to be as significant an issue to owners with buildings in Canada as it is in other parts of the world. It is still not clear why concrete in Canada appears to be less affected by carbonation.

More recently, CMHC also turned its attention toward specific types of repair, protection and the evaluation of new test procedures. In March 1990, CMHC published a report to give building owners and managers a general understanding of cathodic protection as a means of protection against corrosion of reinforcing steel[17]. While there have been successful applications of cathodic protection to parking garages dating from the early 1980s, CMHC's report notes that the application of cathodic protection to reinforcing steel in concrete is not well documented. In addition, structural integrity, water leakage, drainage, etc. are not addressed by the use of cathodic protection and specialized knowledge of the design, installation and operation of cathodic protection systems must be sought should these be considered in the restoration of the garage structure. The Technical Committee of the National Reinforced Concrete Cathodic Protection Association (NRCCPA), an industry association, also published a reference document in October 1991[18]. Appendix H gives a brief overview of cathodic protection as applied to reinforced concrete.

In December 1991, CMHC extended the examination of cathodic protection to the various types of systems available and the application and costs of these systems[31]. The performance evaluation of the protection offered and the durability of the systems were also considered with the conclusion that the evaluation techniques and performance criteria are a topic of debate and should be addressed through further research.

Protection of reinforced concrete, both in new and repaired structures was becoming common in the early 1980s so CMHC, the Ontario Ministry of Housing (MOH), Public Works Canada (PWC), the National Research Council (NRC) and the Canadian Institute of Public Real Estate Companies (CIPREC), sponsored research into the elastomeric (thin, cold-applied) waterproofing systems used for parking garage protection[33]. Generally, the study found that these systems are effective in preventing chloride and moisture ingress; however, system selection can not be based on standards of performance as such standards do not exist. Similar benefits and difficulties in standards development apply to the asphaltic (thick, hot-applied) waterproofing systems.

The study of a wide variety of parking garage deterioration issues is given in a CMHC/MOH/PWC/NRC/CIPREC report commissioned in 1987[19] that looks at the performance of 62 garages over a four year period. The report again found that deterioration rates varied between structures of the same type; however, the deteriorating structures can be maintained in a serviceable condition, likely to the end of their design life, if frequent repair and maintenance are performed. The report concludes that no single repair scheme is technically superior to another and that early repair is the most effective way of dealing with concrete deterioration. Recommendations from that report include the maintenance of a log book for each parking garage, annual inspections by consulting engineers and the development of standards for waterproofing and sealers to be used on garages.

Most recently, CMHC has been exploring the problem of corrosion of post-tensioning cables in structures. This form of structural reinforcement became popular in the early 1980s in Ontario for high-rise residential structures and has been used extensively for reinforcing in buildings in Alberta for a longer period. The test technique known generally as time domain reflectometry (TDR) is being evaluated as a means of isolating areas of probable corrosion of the cables. A brief discussion of TDR is given in Chapter 2.

As the results of research into new testing and repair techniques develop, the owners and managers of buildings can be assured that the data and information produced will be available via the usual sources of published reports, seminars and technical papers. Application of the new techniques to each particular situation will require merging of the technical and financial opportunities available to each owner. While organizations such as CMHC can publish information on technology, the operations and financial situations unique to each building are best known only to the owner or manager of these buildings. Accordingly, repair choices should be made jointly by owners and their consultants.

APPENDIX B DURABILITY AND CONSTRUCTION STANDARDS

The attributes of a concrete element are the result of its constituent materials and its manner of construction; therefore, the properties and durability of the concrete are significantly influenced by the construction procedures. To enable the creation of concrete with the specified intended properties, many rules have been developed to identify and ensure correct construction procedures. These rules, procedures and test methods are presented in various standards written and published by the Canadian Standards Association (CSA). These standards become legal requirements when referenced by provincial building codes and they are an excellent guide to the means of creating good concrete structures.

DEVELOPMENT OF NATIONAL STANDARDS AND BUILDING CODES

The development of the National Building Code of Canada (NBC) was brought about in 1937 under the joint sponsorship of the National Housing Administration Department of Finance and the Codes and Specifications Section of the National Research Council. To a large extent the impetus to develop a national standard was the entry of the Canadian Government into the housing field and the recognized need to produce a uniform standard in Canada.

Since Provinces have authority under the Constitution Act to develop laws governing building in Canada, the NBC is not a legal requirement but a model standard that can be adopted by the provinces to govern the construction of buildings. However, it has been adopted by most provinces with little modification and, under the auspices of a provincial regulation, it has become the legal requirement for design and construction. The current NBC is now prepared by the Canadian Commission on Building and Fire Codes and is published by the Canadian Codes Centre.

The Code has essentially been a document that sets down the minimum requirements for pubic health, fire safety and structural sufficiency of buildings; requirements unrelated to these aspects are kept to a minimum. It is important to recognize that the provisions in the Code are minimum requirements in a typical environment and typical use. If the environment is expected to be more severe than anticipated, additional measures must be taken in the design and construction of the building.

To a considerable degree, the Code references the standards for design and construction that are prepared by standards writing bodies. In particular, the Codes reference standards that govern material properties, mix designs and structural design for concrete. Those standards are produced by the construction industry through organizations such as CSA, the Canadian General Standards Board (CGSB) and the American Society for Testing and Materials (ASTM). A full listing of the referenced standards is given in the code document. Some standards are approved by the Standards Council of Canada as National Standards of Canada and, thus, bear the letters CAN before the standard's designation. For instance, CAN/CSA-A23.1-M90 is the standard referenced in provincial and national building codes as a national standard on Concrete Materials and Methods of Construction.

CONCRETE STANDARDS IN BUILDING CODES

Various standards have been developed in relation to the design, construction and testing of concrete materials and concrete structures. These standards are listed in the national and in provincial codes.

CAN/CSA-A5-M88	Portland Cement
CAN/CSA-A23.1-M90	Concrete Materials and Methods of Construction
CAN/CSA-A23.2-M90	Methods of Test of Concrete
CAN/CSA-A23.3-M84	Design of Concrete Structures for Buildings
CAN3-A266.1-M78	Air-Entraining Admixtures for Concrete
CAN3-A266.2-M78	Chemical Admixtures for Concrete
CAN3-A438-M84	Concrete Construction for Housing and Small Buildings
CAN/CSA-S413-8	Parking Garages - Construction

In addition, there is a standard in production that will affect the restoration of concrete:

CSA Standard S448.1	Repair of Reinforced Concrete in Buildings

Expected to be available in early 1994, the repair standard addresses the repair of structural, conventionally, reinforced concrete for buildings. The standard is to be applied under the direction of a professional engineer qualified in the repair and protection of the reinforced concrete of buildings. Investigation tools and techniques are listed for use at the discretion of the professional engineer. Requirements for technical specifications, repair practices, inspection and testing of the repair materials are also listed in the standard. The standard also gives guidance as to the maintenance of structural concrete.

Successful implementation of repairs that would use this concrete repair standard would still depend heavily on the experience and knowledge of the professional engineer assigned to perform the work and on the participation of the owner in the decision as to which repair strategy is right for their purposes.

CONCRETE MIX PROPORTIONS

Whether for new concrete construction or for repair concrete, selection of the material proportions for the concrete mix must address a number of objectives, the principal of which are:

- adequate workability or consistency of the plastic concrete to permit proper placement, generally addressed by specifying slump;
- adequate strength in the hardened state to provide the structural strength assumed in design, generally addressed by specifying the 28-day compressive test strength;
- volume stability, meaning limited shrinkage during hydration;
- impermeability and other properties, such as entrained air, to provide durability in the service environment;
- economy of use of the materials;

To a significant extent, these are interrelated items because the achievement of one will aid in the attainment of another. However, there are some significant factors to consider in order to produce durable concrete. The major factor is the control of the water/cement ratio in the mix. Generally, provided a workable plastic mix is created, the lower the water/cement ratio, the greater is the durability, the strength, and the volume stability.

Durability often relates indirectly to concrete strength, and although strength may not be an issue for structural reasons, attainment of a minimum concrete strength is important for some elements to provide a level of durability. For example, for sidewalks, curbs and gutters, many authorities request a specified concrete strength of 30 MPa. The strength of repair concrete should reasonably match that of the substrate concrete.

TRANSPORTATION, PLACING AND CURING CONCRETE

TRANSPORTATION

Whether site-mixed or produced off-site and delivered, it is important that concrete be placed in its final position as soon as possible after mixing is completed. Agitation may prove necessary to maintain the mix uniformity. Moreover, because plastic concrete has a limited usable life and its strength and durability properties suffer if hardening is delayed beyond that usable life, the time allowed between batching and complete discharge of the material to its final location is limited by CSA standards.

PLACING

Proper handling and depositing of the plastic concrete is important to achieve not only a good looking finished concrete element, but also one with the properties sought in the mix design. The concrete may be distributed by buckets, chutes, conveyors, pumps or other means and consolidated in its final location by tamping or vibrating, but the methods used must maintain the uniformity of the plastic concrete by eliminating segregation of materials and retaining the specific qualities sought, such as air-entrainment.

Delays in transporting and improper placing sequencing can result in *cold joints* in the concrete. These occur at locations where the previously placed concrete has hydrated sufficiently to prevent mixing of the interface between that concrete and the subsequently placed concrete. The result is a discernible bond line. Occasionally, there is segregation of the second pour at the bond line in the form of honeycombing (extremely porous concrete, generally lacking in cement paste and fine aggregate). The result is poor appearance, weakened structure and often, increased likelihood of water penetration through the concrete at the bond line.

Finishing of the surfaces that are not formed should not begin before the majority of the excess water in the concrete has come to the surface and evaporated. Finishing too soon can greatly reduce the surface strength. This effect often appears as scaling and pitting of the concrete. To achieve dense, hard and durable concrete floor finishes, an experienced floor finishing tradesman is necessary.

CURING AND PROTECTION

Owners and their consultants need to consider the 'down time' involved in properly placing and curing repair concrete. To rush through these steps in the repair of concrete in order to more quickly return a floor to use, will result in a less durable repair. In fact, improper placing, curing and protection practices used during the initial construction often contribute to the deterioration of concrete. The extent of the deterioration resulting from such poor practices should be assessed by the consultant and considered in any repair and protection recommendations.

Concrete transforms from the plastic state to the hardened state by a process called hydration, in which water reacts chemically with the cement. In freshly-placed concrete, the loss of moisture on an exposed surface because of evaporation creates a porous, permeable and weak concrete surface because of the incomplete hydration of the surface concrete. The aim of curing is to maintain a moist surface condition to promote adequate continued hydration in order to provide a more durable concrete surface and at the same time reduce plastic shrinkage effects, further improving durability. Curing practices may involve either "water-adding" (i.e. spraying, ponding, wet burlap cover) or "water-retaining" (i.e., plastic sheet cover, curing membrane) techniques. Proper curing is vital to optimize the strength, impermeability and durability of the concrete. These curing and protection practices are equally important for repair work as for new work.

CAN/CSA-A23.1 describes requirements for curing and protection of concrete for a variety of exposure conditions and concrete uses. The basic period of 3 days at a minimum temperature of 10 deg. C or for 35 percent of the specified 28 day strength should be extended where the concrete is to be exposed to aggressive environments, abrasion or other severe exposure. Additional curing may also be needed to achieve a certain strength.

In cold weather, the addition of water should be terminated 12 hours before the end of the protection period to avoid the risk of damage due to freezing. Protection of the concrete during cold weather (at or below 5 deg. C) requires the use of insulating blankets, heated enclosures or other means to protect the concrete against freezing.

In hot weather (greater than 27 deg. C), "water-adding" type curing is required to minimize overheating during the initial hydration period and thus reduce the risk of crack formation. Protection during hot weather using windbreaks, sunshades, water spray (fogging) or delaying placement until the evening when evaporation is lessened, is intended to protect the concrete from excessive surface drying and the loss of the water needed for hydration. Shrinkage cracking is also reduced.

Maintenance of suitable protection against excessively cold and hot temperatures during the hydration stage of the early life of concrete is essential for a durable product.

Quality Control During Construction

The objective of quality control, applied to new concrete, is to monitor the supply and discourage the variability of the "plastic" concrete as delivered. "Quality control" generally involves testing samples of the concrete for consistency (slump), air content and sampling for subsequent compressive strength testing. These tests measure the physical parameters that describe the quality of the concrete and thus its probable long-term durability. All tests are described in CSA Standard CAN/CSA-A23.2 Methods of Test for Concrete.

Owners should be aware that, while quality control during the construction may have ensured that the materials conformed to the specifications, the specifications used and the design of the concrete elements in existing structures have proven to be inadequate to resist deterioration. Quality control to ensure conformance to the specifications, therefore, does not assure durable concrete unless the production factors,(i.e., the *design* and *construction* of the concrete element), meet the demands of the use factors (i.e., the *environment* and *loads*).

The procedures described in the following are fundamental checks on the specified properties of both new and repair concrete.

SLUMP

The test is performed by compacting the fresh concrete into a standard shaped cone and measurement of the amount that the concrete, so formed, "slumps" after the cone is lifted away (Figure B-1).

The testing of slump is a convenient measure of consistency and workability which is related to total water content in the concrete mix. Since water/cement ratio has direct bearing on the strength and since excessive amounts of water can greatly increase cracking due to drying shrinkage, slump control within the specified range, is important. Slump can also be affected by the use of admixtures and by the shape of the aggregate. These characteristics should be considered in the specification of slump. Workability, and thus slump, can be increased by the addition of certain types of chemical admixtures to the concrete. These admixtures make the transportation and placing of low water/cement ratio concrete easier and their effects must be taken into account when this test is performed.

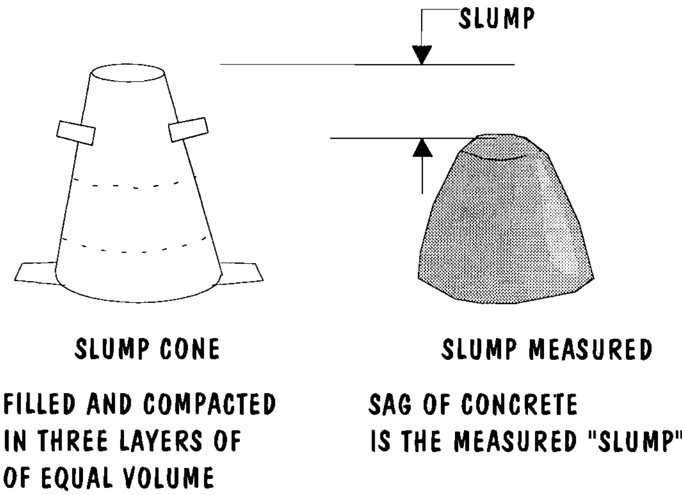

Figure B-1: Slump Test for Concrete Consistency

AIR CONTENT

The testing of total air content of the plastic concrete assesses the total air content prior to placing. Variations in combination of water, admixtures and cementitious materials will affect the total air content of the plastic concrete which in turn affects resistance to freeze-thaw cycles. The two test methods defined by CAN/CSA-A23.2 and used in routine construction measure only the total air content. These tests, A23.2-4C and A23.2-7C, describe procedures for sampling and measurement of total air content of plastic concrete using the "Pressure Method" and the "Volumetric Method", respectively. Both procedures provide quick, accurate, repeatable test values; their primary drawback is their failure to measure air void distribution. Thus, while the total air content of the concrete placed may be within the specified limits, the distribution of the air voids may be inadequate to protect the concrete from freeze-thaw damage.

The determination of total air content and air-void distribution of plastic concrete in place has, however, become the subject of much recent research. The development by the Strategic Highway Research Program [25] (SHRP) of total air content and air-void distribution test procedures is a welcome response to the need for rapid in-situ tests for air content. One new test procedure involves a fibre-optic device and the analysis of reflected light intensity to assess air content. Another new test is being developed which associates the distribution of the air bubbles of various sizes to change in buoyancy of a sample of fresh concrete suspended in a liquid that frees the air bubbles from the sample. The latter test may necessitate development of new parameters other than specific surface and spacing factor against which quality air entrainment would be measured.

A test of the air content of the plastic concrete should be performed on concrete supplied for repair purposes where air content for durability is a requirement. At this time, testing of plastic concrete is limited to total air content.

Consultants can determine the air void distribution of hardened concrete by extracting core samples and performing microscopic analysis in accordance with CAN/CSA-A23.2-17C. The results of that test could assist the consultant in determining whether or not the concrete in place is susceptible to freeze-thaw damage and to help determine the repair and protection procedures needed.

STRENGTH

Compressive strength tests performed on standard samples of the concrete, when assessed statistically with other tests of the same mix design, not only confirms that the concrete as supplied to the construction site meets the structural design criteria, but also can reveal potential variability in quality of supply. Since it is very rare that all individual batches of concrete are tested in the field for slump and total air content, statistical comparison of this test data is useful to assess trends in supply quality and to identify anomalies in the material supplied. The strength of specimens is generally measured after 28 days of curing. These specimens must be cast, cured and tested as specified by the CAN/CSA-A 23.2 standard test procedures. Improper treatment can give spurious results.

As noted above, the strength test is useful as a value in a statistical evaluation of the uniformity and measure of strength of concrete supplied to the construction site. There exists no standard test to assess in-situ strength potential for concrete while it is in the plastic state.

INSPECTION

The tests described above, being relatively simple and inexpensive to perform, typically form a basis for concrete construction quality control. To be assured of meaningful results, the standard CSA specified procedures must be rigidly followed. Testing of concrete for quality by inspectors, tradesmen or professionals who are unskilled in the procedures is inappropriate and should not be accepted. Procedures improperly performed and samples improperly obtained or cured, disqualify the test and invalidate the result.

Testing of concrete properties should not be confused with inspection of concrete during construction or during a repair program. Proper inspection of the concrete construction or repair process for the purposes of quality control does not rely on standard procedures but on the knowledge of the professional performing the inspection. The need for this expertise deserves more consideration than normally given. Failure to properly place, consolidate, finish and cure concrete leads directly to poor durability.

While there are standard tests for concrete products that can be used to check quality of the supplied materials, many of the proprietary products do not readily conform to the test procedures. In such cases, inspection of the use of proprietary materials, against the intent of the specifications for the work and the suppliers directions, can detect inadvertent misuse and thus, prevent inadequate long-term performance of these special products.

When performed properly, the design of the repairs for a building considers the unique needs of the structures being repaired. These needs, having been translated into a technical specification, should be monitored by the designer at all stages to verify that the repairs will be effective.

APPENDIX C DURABILITY AND CONCRETE COMPOSITION

Concrete is a composite material consisting of particles or fragments of a relatively inert mineral material bound together by a cementitious binding material. These principal components are normally termed *aggregate* and *cement paste*. The cement paste undergoes a transition from the freshly mixed *plastic* state to the *hardened* state because of a chemical reaction between the cement and the water, termed *hydration*. Due to the occurrence of volume reduction of the paste in the hydration process, entrapment of air during mixing and placing, and the presence of water in excess of that required for hydration (water is added for workability of the plastic concrete), the hardened concrete contains pores interconnected by channels. Hence, hardened concrete is a porous material.

Currently, the most frequently used concrete in buildings is that containing Portland cement. Portland cement was first developed in the early 19th century and derived its name from the fact that the concrete product it created resembled building stone from the island of Portland, in England.

Within the past 30 years, many new products and types of concrete have been introduced. New chemical additives have been marketed, but the major change is the increased use of supplementary cementing materials in Portland cement concrete. The range of concrete building products has expanded and it is fortunate that the range of knowledge of durability about these products has expanded as well.

THE QUALITIES OF DURABLE CONCRETE

Production of concrete building products and components involves the proportioned combination of materials that must be mixed, transported, placed in forms and suitably protected during the curing process while hydration proceeds to create the desired hardened concrete. If any of these aspects (the base materials, the mix proportions, the transportation, placement and curing techniques), are not properly completed, the performance of the concrete will suffer, both in strength and in durability.

In buildings, concrete elements are subjected to a variety of service environments, such as:

- exterior wet conditions (i.e., balcony slabs),
- exterior moist conditions (i.e., exposed soffits),
- exposure to deicing chemicals (i.e., sidewalks, garage floor slabs),
- exposure to sulphates (i.e., foundations),
- interior moist conditions (i.e., frequently washed floors), and
- interior dry conditions (i.e., slabs, walls and columns).

To remain durable, concrete must be designed and constructed to be able to withstand these conditions. The constituent materials, mix proportions, and construction practices each contribute to the initial and ongoing level of durability. In order to withstand the service environments imparted by the intended use, and thus be durable, concrete must be sufficiently impermeable, have sufficient strength and it must have sufficient protection against aggressive environments.

Characteristics of the concrete that produce these attributes and the current procedures for verifying that these attribute are present are described as follows.

LOW PERMEABILITY

Hardened concrete is a porous material. Because the voids are interconnected to some extent, concrete is permeable to gases and liquids. Many of the processes causing disintegration of the concrete material itself, or materials embedded in it, involve penetration of gases and liquids into the concrete, so, the degree of permeability of concrete is a fundamental issue in determining its likely durability.

Generally, considering normal Portland cement concrete, the lower the permeability of the concrete by fluids, the more durable it will prove to be. Mix constituents and proportions, including aggregates, admixtures and water-cement ratio, are a determinant of the degree of permeability, but the construction procedures - placement, compaction and curing - are also important factors in determining the degree of permeability of the product.

In spite of the importance of mix proportions and permeability to the long term durability of concrete, test procedures for direct measurement of these properties are not well developed. The principal means of assessing the adequacy of the mix and the permeability characteristics of concrete has been by indirect tests for other aspects, for example, strength testing of cylinders cast of fresh concrete or cores extracted from hardened concrete. Results from tests give no direct information about the ability of the concrete to withstand corrosion-induced delamination and the attendant structural strength loss. The slump test performed on the fresh (plastic) concrete only loosely establishes the likelihood of excessive water in the mix and, hence, the probability of an unacceptable water-cement ratio and attendant strength loss and increased permeability. Slump is also affected by the admixtures added to the fresh concrete to improve workability. Even the more recent attempts at assessing the probable permeability of concrete using the chloride penetration tests, AASHTO T277 and the AASHTO chloride pond test are "indirect" tests, of permeability. These are good methods, but they are not broadly available. Simpler methods would be desirable for use by a greater number of owners and their consultants.

STRENGTH

While the strength of the hardened concrete contributes to resisting the forces of disintegration, strength is not the only issue. Of greater significance is that for the usual range of concrete strengths used in buildings, a stronger concrete has other inherent properties that render it more durable than a weaker concrete. This is the rationale for CSA Standard CAN/CSA-A23.1, requiring concrete, intended to be exposed to aggressive environments, to have minimum specified concrete compressive strengths or the related maximum water-cement ratio.

PROTECTION AGAINST AGGRESSIVE ENVIRONMENTS

The design life of buildings is usually 40 to 50 years and the concrete elements, if properly proportioned, mixed, placed, finished and cured, can certainly meet this requirement. In benign environments in a building, the serviceable life will be much longer. However, in hostile and corrosive environments, such as that experienced by an exposed concrete slab subjected to freeze-thaw cycles and application of deicing chemicals, the useful life may be shorter than 10 years if proper protection is not provided. In such environments, it has become the practice to protect the concrete through the use of special cement additives that retard the deterioration, and by the addition of coatings.

Protection normally afforded concrete in freeze-thaw environments is through the incorporation of entrained air in the cement matrix. Intentionally entrained air should not be confused with random porosity that results from the inability of the hydrated gel to fill all the space that was previously occupied by water. Gel is the term used to describe the compounds that form when cement reacts with water. Unlike naturally occurring porosity, entrained air will not have an adverse effect on the durability of concrete.

Structural concrete must crack in order to take advantage of the flexural strength characteristics of the reinforcing steel. The "cracked section" is a basis for design of flexural elements. However, the impact of the cracks on the long term durability of the structure vis-a-vis water penetration, leaching, or corrosion is not commonly considered in the structural design of the reinforced concrete elements. In such cases, coatings are relied upon to bridge the narrow cracks that result from normal structural loads and normal shrinkage. In most instances, the thermally-induced movements are accounted for in structural design by the introduction of expansion and contraction joints. These joints demand careful attention during construction as the seals placed at these joints will likely leak.

CONSTITUENTS OF CONCRETE AND DURABILITY

Concrete is generally thought of as a durable and serviceable material. Some types of concrete continue to exist and perform after centuries of use. Concrete made with the currently popular Portland cement has the potential to last for long periods when properly designed, placed, finished and cured. In instances where one or more of these stages in concrete construction is given inadequate attention, the concrete can deteriorate. Its lack of resistance to deteriorating influences can take a variety of forms, ranging from surface defects, e.g., efflorescence, dusting, scaling and popouts, to deeper spalling deterioration, to complete disruptive disintegration of the concrete mass. Apart from failure of the concrete itself, the concrete may limit the useful life of the element it serves by failing to sufficiently protect items embedded in or attached to it.

PORTLAND CEMENT

Portland Cement is a manufactured product. The raw materials for Portland cement - usually limestone and clay - must be prepared and blended to provide the appropriate proportions of lime, silica, alumina and iron that will create the required compounds in the cement powder used to make concrete. The percentages of these principal constituents are purposely varied in the manufacture of Portland cement to create five different types of cement product, each with characteristics intended for a specific type of application. The basic ingredients used in the manufacture of Portland Cement are listed in Table C-1:

Table C-1 Major Constituents as Oxides (% by mass)

Lime,	CaO	60-65
Silica,	SiO_2	20-24
Alumina,	Al_2O_3	4-8
Iron,	Fe_2O_3	2-5

The constituent compounds of Portland cement are usually abbreviated in what is known as cement chemistry notation. In this way the chemical description for calcium (CaO) is reduced to C; similarly silicate (SiO_2) is reduced to S, Aluminate to A and Ferrite to F. The four primary compounds, sometimes referred to as Phases, are shown in Table C-2:

Table C-2 Primary Compounds in Normal Portland Cement

Tricalcium silicate,	C_3S
Dicalcium silicate,	C_2S
Tricalcium aluminate,	C_3A
Tetracalcium aluminoferrite,	C_4AF

The proportions of the main constituents of the cement determine the hydration rate and the chemical resistance of the concrete. CSA Standard A5 describes five cement types, as follows in Table C-3:

Table C-3 Types of Cement

Type 10	Normal Portland
Type 20	Moderate sulphate resisting
Type 30	High early strength
Type 40	Low-heat
Type 50	Sulphate resisting

Provided the ratios of the principal compounds in the cement are in the appropriate range, minor amounts of other materials may be present and tolerated in the cement as impurities.

Volume Stability

Percentages of some of the principal compounds identified in Tables C-1 and C-2, and the related impurities, must be controlled in order to eliminate durability problems caused by the cement. One of the potential problems arising out of improperly manufactured cement is instability in the volume of the concrete.

Lack of volume stability of hardened concrete, resulting in expansion, cracking and eventual disintegration of the concrete, can result from a number of durability factors in the cement including unsoundness, excess sulphates, excess alkali content and excessive tricalcium aluminate content.

Unsoundness

Unsoundness of cement paste is generally attributed to the belated hydration of free lime in the cement particles and to the presence of excessive magnesia (MgO), both in moist environments. CSA Standard A5 sets limits for MgO content to control this and an accelerated autoclave expansion test (ASTM C151) is used to detect unsound cement. These tests and limits are considered acceptable and unsound concrete is now rarely an issue.

Sulphate Attack

In the presence of water, soluble sulphates react with the hardened cement paste leading to cracking and disintegration of the concrete. This process is of great concern in regions where the soil is rich in soluble sulphates (e.g., Prairie provinces). Sulphate attack is a two-phase process. First, sulphur trioxide (or sulphates) from the environment combine with free calcium hydroxide (hydrated lime) to form calcium sulphate (gypsum). The gypsum then combines with calcium aluminate to form calcium sulphoaluminate. These reactions result in an increase in the solid volume that leads to cracking of the hardened concrete. In the long-term, this mechanism can lead to disintegration of the concrete and increased susceptibility to other durability concerns, such as freeze-thaw resistance and chloride penetration.

In situations where concrete will be exposed to sulphate bearing soil or water, potential problems can be avoided by using sulphate-resisting Portland cement. CAN/CSA-A23.1 gives guidance on the testing of conditions for sulphate attack and requirements to counteract it. If the appropriate investigation and testing of the ultimate service environment for the concrete are conducted prior to the mix preparation, the problem can be essentially eliminated by selection of specific cements. CAN/CSA-A23.1 effectively defines the requirements for concrete subjected to three levels of sulphate attacks. For each exposure level, the standard provides the appropriate compressive strength, water-cement ratio and cement type for the concrete.

The use of a Type 20 (Moderate) cement is typically effective in resisting sulphate attack of mild or moderate severity, while Type 50 (Sulphate Resistant) cement is used to resist high sulphate conditions. Therefore, if the sulphate conditions are suitably identified at the early stages of a project, the concrete mix can be tailored to avoid long-term durability problems. There can also be sulphates in the cement itself; therefore, CSA Standard A5 limits the sulphate content of cements, and prescribes a 14 day expansion test.

Alkali Reactivity

In moist environments, certain siliceous mineral constituents of aggregate interact with alkalis in Portland cement to cause expansion, cracking and disintegration of hardened concrete. Alkali-Aggregate Reactivity (AAR), is controlled by limiting the amount of the alkalis in the cement. However, no limits are set on cement alkali content in CSA Standard A5. Instead, normal practice involves control of the use of reactive aggregates by adequate petrographic evaluation of the aggregate.

Aluminates

High tricalcium aluminate content in the cement in moist environments with the presence of sulphates, creates disintegrating expansion of hardened concrete. CSA Standard A5 does not set an upper limit for tricalcium aluminate for normal Type 10 cement, but does for Types 20 and 50 which are normally specified for environments where the concrete will be exposed to sulphates.

Supplementary Cementing Materials

During the past 30 years, there has been increased use of "supplementary cementing materials" (SCMs) in Portland cement concretes. These are incorporated as a partial replacement for Portland cement and, when used in this manner, possess cementing properties. Supplementary cementing materials include:

- natural pozzolans,
- fly ash,
- condensed silica fume, and
- slags,

many of which are industrial process by-products.

The specific characteristics of concrete made with SCMs can vary dramatically with relatively small variations in the amount of these materials added to the concrete mix. Addition of SCMs provides better workability of the plastic concrete and reduced heat of hydration. SCMs reduce permeability and, therefore, are beneficial in avoiding deterioration processes that require the ingress of moisture or other reactive chemicals into the concrete. Due to the reduced permeability, however, the resistance to scaling is often reduced.

Research is underway to establish the actual limits on proportions and the extent of the impact that each SCM has on the performance of the hardened concrete. To date, it has been determined that SCMs can be effectively used to create concrete with superior resistance to such conditions as alkali-aggregate reaction and sulphate attack, thereby suggesting alternatives to the use of Type 20 and Type 50 Portland cements. However, it has also been found that some concretes made with SCMs have been more susceptible to early durability concerns because the slow rate of development of the concrete strength has led to a corresponding slow reduction in the permeability of the concrete.

Research regarding SCMs is ongoing and the range of knowledge is incomplete; therefore, before SCM concrete is used, it is important that expertise be sought, that the mix proportions be confirmed by testing and that suitable curing be provided.

NORMAL DENSITY AGGREGATE

Deleterious Substances

The most common substances of concern in the normal-density aggregate are soft particles such as silt, organic matter and unsound particles, such as sandstone, chert and shale.

The unsound particles create concern about: abrasion resistance of the concrete; expansion of the particles in moist and freeze-thaw environments, which leads to cracking and deterioration; and disintegration, causing pitting, pop-outs and scaling of the concrete surface.

Where the concrete will be used as the finished surface, it may be necessary to limit certain other aggregate, such as iron bearing minerals. Such aggregate can oxidize in moist, even high humidity environments, resulting in rust stains on the surface of the concrete. These iron bearing aggregates are otherwise generally not harmful and would not be considered to be damaging to purely structural concrete.

CAN/CSA-A23.1 prescribes tests to control the presence of deleterious substances in aggregates, and sets limits on allowable contents of deleterious substances. In general, careful washing and petrographic examination of the aggregate can be effective in eliminating the durability problems associated with deleterious substances

Alkali-Aggregate Reactivity

Certain aggregates that contain alkali-reactive minerals can interact with alkali in Portland cement to cause expansion, cracking and deterioration of hardened concrete. These reactions will occur even more rapidly in high temperature and moist conditions. The reaction between the alkali and the aggregate occurs at the surface of the aggregate, creating a volumetric expansion of the aggregate material. Stress is induced in the cement paste and surrounding aggregate and builds until a crack forms in the concrete. The reaction does not proceed rapidly and, therefore, leads to a long-term deterioration concern. Once cracking is initiated by the reaction, the disintegration of the concrete is accelerated by the increased penetration of water into the concrete and other associated durability issues.

Petrographic examination will usually detect the reactive minerals. The existing CSA test method, A23.2-14A Potential Expansivity of Cement-Aggregate Combinations, is reliable, but impractical because of the long duration of the test (6 months to 2 years). There are several other tests that have been developed, including a rapid test to avoid the delays imposed by the current CSA test method. Virtually all of these other tests have been found to give drastically different results for the same aggregate tested at different temperatures and different humidities[20]. As a result, the validity and accuracy of these tests are questioned by the industry and a conclusive and accurate test continues to be unavailable.

As a result of shortages of suitable aggregate sources in many areas of Canada, the reactivity testing of aggregates is one of the critical industry issues at the present time. For example, the Ontario Ministry of Natural Resources publishes and maintains reports on aggregate resources to designate sites that can produce quality aggregates[21]. The development of a reliable short-term test for aggregate reactivity is, therefore, a high priority in the research sector in order to avoid a dramatic increase in the occurrence of alkali-aggregate reactions. This is now an active issue with a CSA Subcommittee.

Freeze-Thaw Durability

Soft, porous aggregates are susceptible to deterioration if put into a moist, freeze-thaw environment. Expansion of freezing water trapped within the aggregate will crack the aggregate and, thus, eventually the aggregate will disintegrate within the hardened concrete. This can occur regardless of the freeze-thaw resistance characteristics of the cement paste. As this process continues, the remainder of the concrete will also be affected by the expansion of the aggregates, leading to micro cracking and pop-outs.

CAN/CSA-A23.1 sets limits related to aggregate freeze-thaw durability by a magnesium sulphate test for the soundness of aggregates.

LOW DENSITY AGGREGATE

In general, the durability issues of normal-density aggregates discussed above are also applicable to low-density aggregates used to reduce the weight of the concrete elements. Light-weight concrete is not normally intended to be used in harsh environments requiring highly durable materials as it can be more susceptible to durability problems due to the high absorption rate of the aggregate.

Existing Canadian codes and standards do not adequately deal with the question of the use of low-density aggregates in specific environmental conditions. At the present time, CAN/CSA-A23.1 merely defines two classes of lightweight concrete, but does not discuss their use in moist or harsh environments. In part, this may be due to the fact that the use of low-density aggregate is diminishing in the industry. Information on the use of low-density aggregates in specific environments is available in the American Concrete Institute (ACI) standard, **ACI 211.2.**

ADMIXTURES

Admixtures are added in small amounts to the concrete mix at the batching stage to create a specific desired effect. A large number of different admixtures are commercially available, each aimed at changing some aspect of the workability, set, strength or durability of the concrete. These admixtures often assist in the control of performance problems that may result from poor workability or low strength. Admixtures may also create other durability concerns which must be considered.

Air-Entraining Admixtures

The overall volume of the hardened concrete is made up of three types of voids[24].

(a) Gel pores: interstitial cavities between cement gel hydrates, generally believed to be 15 to 20 Angstroms (Å) in diameter (one angstrom is 10^{-10} metre). Water cannot freeze in gel pores above -78 deg. C, thus the water within gel pores does not directly contribute to freeze-thaw damage.

(b) Capillary cavities: unfilled spaces between aggregations of gel particles formed by uncombined water in excess of that used for hydration. Water can freeze in these pores which average 5000 Å in diameter.

(c) Entrained air: bubbles of air purposely introduced into the paste ranging from a few microns to a few millimeters diameter. These voids serve as pressure release vessels for water in a freezing environment.

Typical entrained air volume is in the order of 4 to 8 percent of the concrete volume. Concrete may also contain entrapped air due to inadequate consolidation during placement.

Entrained air in hardened concrete is the most critical factor governing the freeze-thaw durability of the material. The adequacy of air entrainment is dependent on a series of conditions, all of which must be met to produce durable concrete. The water and cement mixture, the paste, is the matrix within which air is entrained. Increased paste content, as would be the case if aggregate size decreased, requires greater total air content. CAN/CSA-A23.1 gives details of the required air content for various common sizes (nominal size) of coarse aggregate and various exposure conditions. A range of appropriate total air content is specified for each exposure and nominal aggregate size. The minimum value represents the minimum total air content below which the cement paste is not adequately protected. The maximum value represents the limit beyond which air entrainment does not provide any useful durability benefit while still reducing strength.

In addition to the total air content, the size and the spacing of the entrained voids are critical to the durability of the concrete (Figure C-1). The size and spacing of the air void system is important, as the minute air voids act as release valves to the buildup of osmotic pressures during the freezing of saturated concrete. The spacing of the air voids must be such that expanding moisture can be forced into it from the pore spaces in the cement paste. The use of an air-entraining agent and proper consolidation of the concrete in

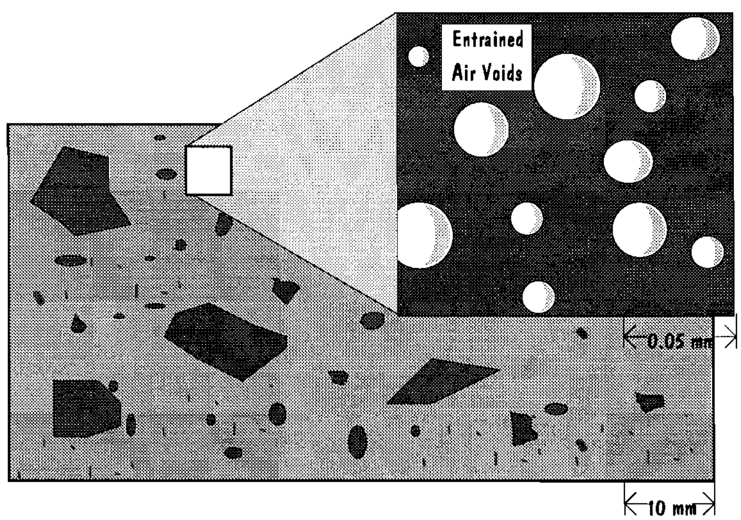

Figure C-1 Air Void System in Concrete place are required for proper air entrainment. The suitability of the air void distribution, size and quantity is, therefore, highly dependent on good construction techniques used to mix and place the concrete.

CAN/CSA-A23.1 also provides the requirements for the air-void system in hardened concrete in the form of the *spacing factor*. Spacing factor is a value representing the distance from any point in the cement paste to the periphery of an air void. These requirements, together with the total air content, are sufficient to provide durable air-entrained concrete. The total air content is measured in plastic concrete; however, there exist no common tests for measuring the other air void characteristics, such as spacing factor or *specific surface* (ratio of the interior surface area of the air void to the volume used as a measure of the size of the air voids). Thus, a problem exists with enforcing the size and distribution of the air-voids, in that the air-void system can only be determined when concrete is already in the hardened state.

At present, the only efficient and commonly available test to assess the adequacy of air entrainment requires extraction of a sample of the hardened concrete. The test for microscopic determination of the air-void system is defined in ASTM C457. The procedure determines total air content, spacing factor and specific surface. Since the distance from the site of hydraulic pressure generation to pressure release at the void is critical and since the total air content must be limited for adequate strength, it is reasonable to measure these factors in an effort to assess the quality of the entrained air. Typically, concrete requiring protection against freeze-thaw action will have between 4 and 8 percent air, depending on the size of aggregate and exposure condition, with an average spacing factor of 0.23 mm and a specific surface of 12 mm^{-1} to 24 mm^{-1}. These indicators have been found to be the best measure of the air-void system in hardened concrete.

Water-Reducing Admixtures

A second common group of admixtures, used primarily to control the workability of fresh concrete, includes the water reducers and superplasticizers (high range water reducers). Water reducing admixtures increase the workability of fresh concrete. The admixture can be used to reduce the amount of water used in the mix by reducing the surface tension of the water in the concrete mix. The effect is achieved by the creation of an adsorbed layer on the surface of the particles which is dissipated as hydration proceeds.

In general, the reduction of the water content in the fresh concrete mix improves the strength of the hardened concrete by reducing the water-cement ratio. It also reduces shrinkage of the mix by reducing the water used. The resultant reduction of the shrinkage of the concrete can be beneficial to the durability of the concrete as the permeability of the concrete to water penetration is reduced with reduced shrinkage cracking.

The use of excessive superplasticizer dosages can be detrimental to the durability of concrete, particularly if the concrete is exposed to saturated conditions and freeze-thaw temperature cycling. Research has shown that superplasticizers do not adversely affect the distribution of the air void system, but can reduce the total air content of the mix by reducing the surface tension of the water to the point that air voids cannot be maintained. In situations where air entrainment of the concrete is required for the service environment and a plasticizer has been used for workability during placement, additional air-entraining agent will be required to compensate for the loss in air-entrainment due to the plasticizer. If air-entrainment of the concrete is not required for the service environment, the use of a plasticizer does not create the need for an air-entraining agent. When super-plasticizers are used in concrete which is to be in a moist service environment, the testing for entrained air-void distribution, size and quantity becomes even more essential.

Set Control Admixtures

This group of admixtures includes accelerators and retarders. Retarders are used whenever it is desirable to extend the time period in which the concrete remains plastic. Typically, such admixtures are used in hot weather when setting time is reduced by the elevated temperature, or when delays between batching and placing are encountered. Accelerators are used to speed the normal setting process which may be desirable in cold weather concreting. Calcium chloride is a set accelerator.

In the case of accelerators, the decrease in the setting time in cold weather concreting is essential to avoid early frost damage that can cause low strength and a high degree of micro cracking, with associated high absorption characteristics. If an accelerator is used under inappropriate environmental conditions, a flash set can occur which can be extremely damaging due to shrinkage from rapid drying and associated high absorption characteristics. Flash set conditions depend on temperature, relative humidity and wind speed. The use of accelerators above about 25 deg C should be carefully monitored. On a dry windy day, flash set can occur at lower temperatures.

The use of retarders in hot weather concreting is often essential to the avoidance of a flash set. If a retarder is used under cold environmental conditions, freezing of the fresh concrete prior to set can occur which can also be damaging to the final product. When the average daily temperature is less than 5 deg. C and the temperature is less than 10 deg. C for more than half of the day over a three day period, then concrete is susceptible to freezing and a retarder should not be used.

Calcium Chloride

Calcium chloride has been widely used in Canada as an accelerator for winter concrete work. The use of calcium chloride has been linked to increased initial drying shrinkage (and hence increased micro cracking of the concrete)[22] and to increased pore pressures during freeze-thaw cycling of the concrete[23]. A greater concern exists with respect to the corrosion of steel embedded in concrete, which is initiated and accelerated by chlorides in the concrete. Whether the added calcium chloride combines chemically during the hydration process, and is therefore, unavailable as free chloride ion to depassivate the reinforcing steel, or whether some proportion remains available, is in debate. In general, the use of calcium chloride is restricted to non-pre-stressed concrete and concrete that will not be exposed to freeze-thaw or corrosion-inducing environments.

CSA Standard CAN3-A266.4 contains cautions regarding calcium chloride in several clauses and provides a section that precludes its use in the following:

- In large masses of concrete placed in mild or hot weather.
- In concrete subjected to sulphate attack, or in reinforced concrete exposed to sea water.
- In pre-stressed and post-tensioned concrete.
- In nuclear shielding concrete.

CSA Standard CAN3-A23.1 also places limits on the amount of water-soluble chloride in the cement material, particularly for pre-stressed concrete and for concrete to be exposed to moisture and deicing chemicals.

The use of calcium chloride under the following conditions should only be undertaken after careful consideration of the environment of the concrete and the likelihood of corrosion of embedded metals occurring:

- In reinforced concrete where stray electrical currents may occur.
- In reinforced concrete subjected to steam curing.
- In concrete in which combinations of dissimilar metals are embedded.
- In concrete floor slabs with a metallic finish.
- In concrete slabs supported on permanent metal forms.

APPENDIX D FREEZE-THAW DAMAGE AND SCALING

MECHANISMS OF DETERIORATION

Scaling, the deterioration of the cementitious matrix, is the result of inadequate resistance to freeze-thaw cycles when in a moist environment.

Since there are voids in concrete, it is permeable and water can be adsorbed onto the surfaces of the gel pores, capillary cavities and entrained and entrapped voids as discussed in Appendix C. Adsorbed water collects on the surface of the void whereas, absorbed water penetrates into the body of the cement matrix. Water that freezes in a capillary expands and displaces the unfrozen water. Subsequent thawing and refreezing creates pressure on the walls of the capillary. Since re-absorption of water into the cement gel is extremely slow, the pressure increases until the cement paste ruptures or the pressure is otherwise released[24]. If the concrete is at or near saturation, the freezing water within the concrete creates internal pressures due to constrained volumetric expansion and disintegration will occur. Concrete deterioration can accelerate due to propagation of fine cracks caused by the cyclic expansion of moisture in the pore voids. Thus, the extent of cracking in the concrete increases in a progressive manner.

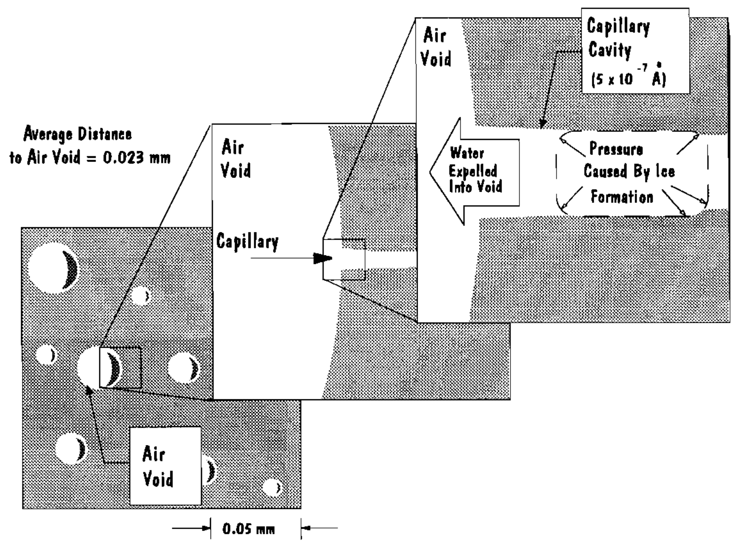

Figure D-1 Relief of Pressure During Ice Formation

There is a higher build up of pressures in concrete that is subjected to freeze-thaw cycles than that which occurs during one cycle. The higher pressures are a result of residual stresses in the concrete at the end of each cycle. It is the build-up of the residual stresses, and the progressive cracking that contributes to the deterioration of concrete in a freeze-thaw cycling environment. Figure D-1 shows how the water in capillary cavities is expelled into the air voids introduced through air entrainment.

In addition to the damage caused by expansion pressures of the freezing water, the cooling of the concrete causes shrinkage that imposes stresses. Under rapid cooling conditions, moisture pressure gradients are also created[27]. These contribute to the mechanical forces causing micro-cracking, which leads to scaling.

APPENDIX E ELECTROCHEMICAL CORROSION

MECHANISMS OF DETERIORATION

Corrosion of the reinforcing steel in concrete creates a rust product that is several times greater in volume than the steel from which it formed. This increase in volume results in pressure on the concrete adjacent to the corroding steel. Cracking and delamination occurs readily, as the pressures exerted by the rust formation exceed the tensile strength of the concrete. Reinforcing bars having large areas of corrosion can split off large areas of concrete.

At more advanced stages of corrosion, the delaminated concrete breaks free of the slab and leaves a depression or spall. Often, at this stage, the reinforcing steel has suffered severe pitting and loss of cross-sectional area. The electrochemical corrosion process may cease at this stage and atmospheric corrosion can, and may, begin.

Corrosion resulting from the electrochemical action is shown in Figure E-1. It is similar to that of a battery in that there is an oxidation-reduction reaction of dissimilar metals in an electrolyte. In the case of reinforced concrete, the small variations in the moisture, oxygen and chloride content of the concrete surrounding the reinforcement, together with residual stress variations within the steel itself, allows the steel to be sufficiently dissimilar at different points in the concrete. The electrolyte is the moist, chloride bearing concrete. Thus, the different portions of the reinforcing steel behave as the anode (negative) and the cathode (positive) and the oxidation-reduction reaction occurs at these two halves of the electrolytic or corrosion cell.

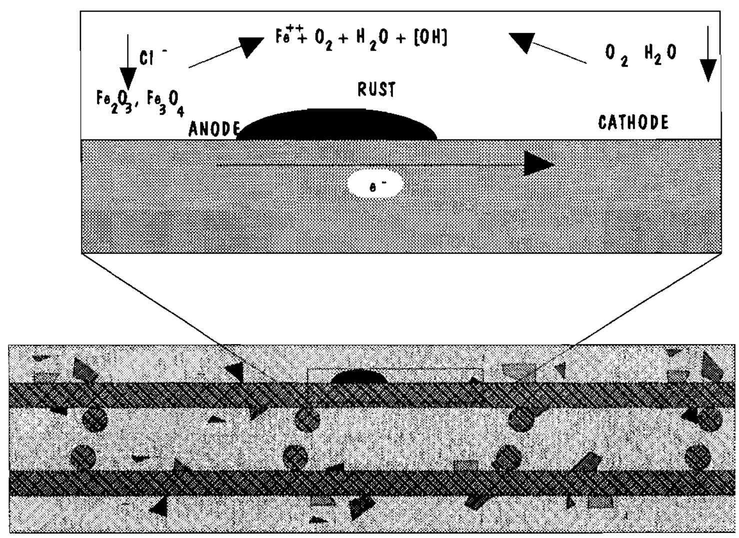

Figure E-1: Electrolytic Corrosion of Reinforcing Steel in Concrete

Before corrosion can occur, the passivating film of oxide, formed by reaction of fresh concrete with the reinforcement, must be destroyed at the corrosion site. Once the steel has lost the protective film, the oxidation-reduction reactions at the anode and cathode can proceed and rust is deposited at the anode. The reaction can be considered in three parts: *Depassivation*, *Oxidation-Reduction Reactions* and *Rust Deposition*.

1. Depassivation

   The mechanism of depassivation has not been precisely determined[29]. However, salt, water and oxygen (Cl^-, H_2O and O_2), which penetrate the concrete from the environment, are known to attack the passivating oxide film (Fe_2O_3 and Fe_3O_4). The amount of chloride needed to depassivate the steel depends on the alkalinity of the concrete[28], greater amounts of chloride being required at higher pH levels. Still, the soluble chloride concentration needed to depassivate the reinforcing steel is relatively small, generally 0.020 to 0.025 percent by mass of concrete. Chloride is not consumed in the corrosion reactions but remains in the concrete matrix contributing to the conductivity of the concrete electrolyte.

   It should be noted that chloride is necessary to initiate the reaction; however, the subsequent rate of corrosion is controlled, not by the amount of chloride, but by the oxygen, moisture and pH of the concrete.

2. Oxidation-Reduction Reactions

   Oxidation occurs at the anode. Since iron is relatively high in the electromotive force series it has a tendency to release electrons (e^-) and oxidize. The chemical equation takes the form:

   $$Fe \rightarrow Fe^{++} + 2e^-$$

   Reduction occurs simultaneously at the cathode, consuming the electrons liberated at the anode in a reaction with water and oxygen to form hydroxyl ions (OH^-).

   $$O_2 + 2H_2O + 4e^- \rightarrow 4OH^-$$

3. Rust Deposition

   The hydroxyl ions combine with iron, water and oxygen at the anode as ferrous hydroxide [$Fe(OH)_2$]

   $$2Fe^{++} + 4OH^- \rightarrow 2Fe(OH)_2$$

   which converts to ferric hydroxide [$Fe(OH)_3$] as the reaction continues.

   $$4Fe(OH)_2 + 2H_2O + O_2 \rightarrow 4Fe(OH)_3\ \textit{(rust)}$$

The two halves of the corrosion cell, anode and cathode, may be adjacent to one another on the same length of reinforcing steel, or may be on different lengths of steel provided the steel component of the electrical circuit is continuous. Thus, corrosion can proceed if oxygen, moisture, chloride and steel are present. The reaction is temperature dependent occurring at a slower rate at temperatures less than 10 deg. C.

APPENDIX F DESIGN, DURABILITY AND SAFETY

WHAT IS SAFE?

The question of safety of the concrete structure must be considered when assessing the effects of deterioration. Safe structures are those that function with an acceptable margin of safety and without damage, under the expected loads and under specific irregular loads, such as those applied by earthquakes and wind. In the design of "safe" concrete structures there are variable factors, such as the design loads, material properties, and structural dimensions and construction tolerances. These factors can alter the safety of the structural element affected by the variables. Normal randomness in these variables is accounted for in the design codes, resulting in a predictable margin of safety acceptable to the code writers. There are also factors associated with construction that affect the safety of the structure. Changes resulting from poor workmanship or shoddy materials will reduce the margin of safety and, perhaps, be undetected until some form of distress causes the owner to investigate the problem. As a result buildings have differing margins of safety, one to another, and as compared with the code defined margins of safety.

It should be expected, as well, that the actual loads applied during the life of the structure will not be exactly that which the code specified as the design criteria. Generally, the loads will be less than that initially considered by the codes; however, the applied loads may, on occasion, be more than assumed in the design of the structure.

The following simplified rationale is provided for owners and managers without an engineering background so that they might understand why failures, though rare, have occurred. The applied loads (S) and the actual resistance of the structure (R) for a group or population of structures or structural elements can each be represented by a probability distribution (Figure F-1). A particular structure will have a particular resistance to a certain combination of loads. If R remains greater than S, the structure is safe. Since, as a result of the variables in design and construction, some structures have greater resistance to the applied loads, those structures will have a greater margin of safety. This increased margin of safety can have a two-fold effect. Firstly, there is likely greater resistance to deterioration imparted to the concrete structure by such qualities as higher strength, better air entrainment, better drainage, and construction closer to design dimensions and tolerances. For instance, a slab that is thicker and of better quality concrete will be less affected structurally by freeze-thaw damage than a thinner slab of lesser quality concrete.

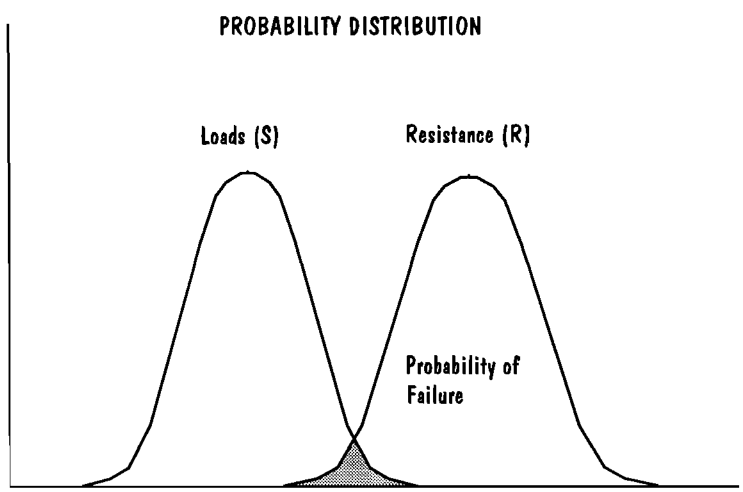

Figure F-1: Probability Distribution of Loads and Resistance

There is, of course, a cost associated with providing increased margins of safety and so it behooves designers, standards writers and code writers to provide a reasonable margin of safety at a reasonable cost. There is also a probability that the Resistance R may be less than Load S. This is depicted by the shaded area under the two probability distribution curves where the curves cross representing the chance of failure. The spread between the average resistance R and average load S can be used to describe the margin of safety.

Two important observations can be made when viewing the applied load and as-built resistance of structures in the way described in Figure F-1.

- Different structures will resist deterioration to different degrees depending on their initial margin of safety.
- With the randomness in the actual applied loads and actual resistance, there is a potential problem in predicting the actual margin of safety against design loads.

Probabilistic modeling of durability and design has been explored as a design tool[26]. The variability in design and construction results in a situation in which not all structures have the same resistance to external load effects. This includes the effects of deteriorating elements such as freeze-thaw, corrosion, cracking, etc. It should be expected, therefore, that each structure will resist the effects of deterioration to differing degrees. This may explain the considerable variation in the amount of distress found in surveys of parking garages.

EFFECTS OF DETERIORATION

It would seem reasonable that concrete with a greater initial resistance would suffer less than those with less initial resistance. Structures made of concrete with poor initial durability would, therefore, have a lesser margin of safety and so be at greater risk when deterioration begins. The structures having poorer quality concrete would also most probably deteriorate at a greater rate. Figure F-2 depicts this condition for a large population of structures, some constructed with high levels of initial resistance using better quality materials and better tolerance and dimensional control during construction (right side of resistance curve), and an equal number without those attributes (left side of resistance curve). Over time, as the concrete of the whole population of structures is exposed to deteriorating effects, the initially more durable concrete (right side of the resistance curve) moves to the left, closer to the applied loads curve. The left side of the resistance curve typifying the structures with less initial durability moves left at a greater rate. This is depicted as a skewed distribution after deterioration has occurred.

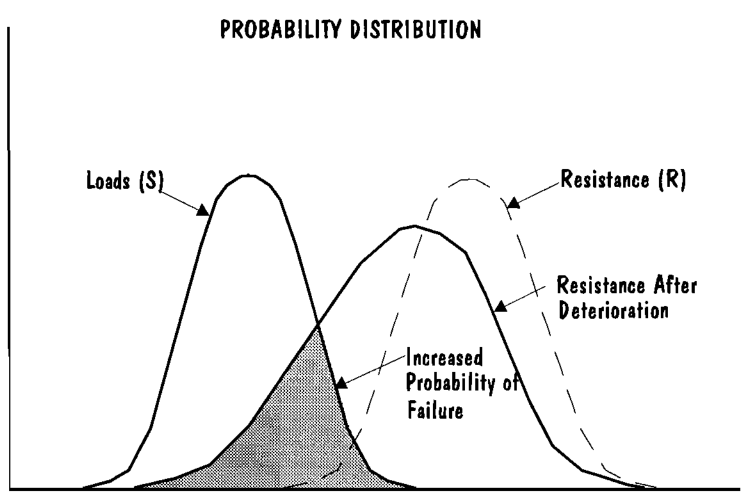

Figure F-2: Probability Distribution of Loads and Resistance After Deterioration

The state of initial durability, the randomness of applied loads and the remaining strength of the structural system after deterioration is difficult to predict or measure. This creates an obstacle to predicting the actual safety of a deteriorating concrete structure. While the loads applied can be reasonably expected to fall within a known range, the rate and impact of the deterioration experienced by a particular structural component in a building is fraught with uncertainty. Deterioration over time will reduce the resistance of the component to the applied loads; however, there is a time lapse between the onset of deterioration and the point at which the component fails.

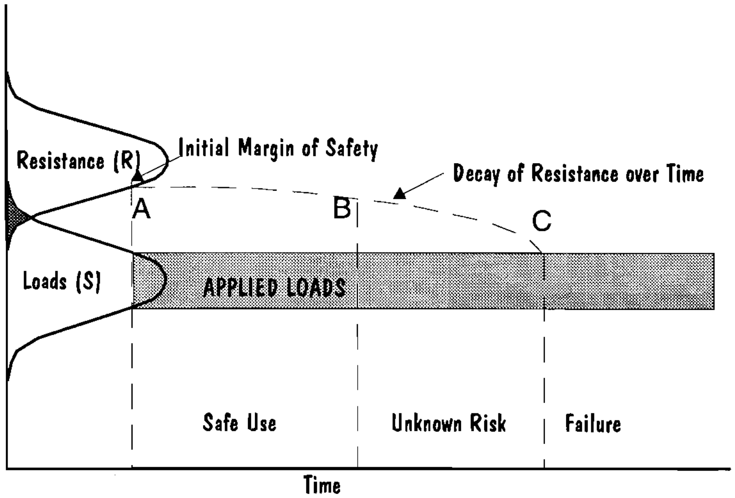

Figure F-3 Reduction in Resistance to Deterioration over Time

Figure F-3 illustrates the deterioration of a particular component that was designed and constructed to possess the resistance to loads identified on the diagram as point A. At that point, the component is safe. As deterioration progresses over time, the resistance of the structural component decreases. Thus, over time, the resistance of the component could be considered to follow a curve that progressively gets closer to the range of loads that the component experiences shown as the *load history*. At some stage during the deterioration process, a condition is reached wherein the structure no longer complies with the design code requirements. This is shown as point B. The structure may as yet not have reached a failure condition, (point C) i.e., a condition that either prevents the structural component from being used for its intended purpose or a condition that results in breakage or collapse. The period of use between points B and C is a period of unknown risk.

In this regard, engineers that are called upon to assess the safety of a building element exhibiting distress would most probably agree on the safety condition if the structural component were at either end of its life (i.e., at or near either point A or point C on Figure F-3). Uncertainty and probable disagreement would more often be the case if the structure is at or near point B, which would, in many cases, represent components in structures that exhibit distress but are still in use with minimal disruption to the operation of the facility. It is at this stage of the structure's life that decisions must be made that involve more than the structural condition appraisal, as appraisals should be expected to differ depending on the expert involved.

FAILURE MODE

Another aspect of deterioration, as related to safety, is connected to the design of the structural component. The modeled deterioration rate shown by the curve in Figure F-3 does not represent all structural components. The design of some components, such as conventionally reinforced concrete beams or the middle sections of flat slabs, rarely reaches a condition of failure without some period of warning. Wide cracks and excessive deflection or bending are signs that the components have entered the stage where continued use puts the owner at risk. Such failure modes are referred to as *ductile failure*. Structures designed using the current design codes are more ductile than those designed using earlier codes.

Structural components, such as column-to-slab connections, post-tensioning cables or support ledge beams for expansion joints, do not exhibit much warning before failure. Such sudden failure is referred to as *brittle failure*. The two types of failure are depicted in Figure F-4.

The life of the ductile element after deterioration has commenced is greater than the life of the brittle element. However, once deterioration has commenced, the rate of deterioration of ductile elements can be expected to increase rapidly.

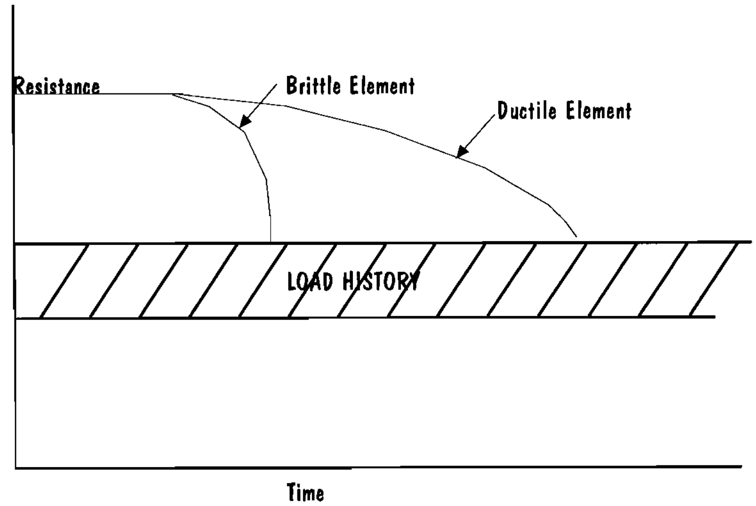

Figure F-4: Brittle and Ductile Failure Modes

No doubt, the ability to define the degree to which, for instance, a structural slab has deteriorated would be helpful to owners and managers of buildings and helpful to engineers in predicting the present margin of safety. However, there is no good source of data concerning the precise condition of structures before deterioration occurs. Although the technical know-how exists to retrieve the information about the structural adequacy of reinforced concrete, it is in most instances, impractical. There is, very little information that an engineer can rely upon to determine the remaining load resistance in structures after the onset of deterioration. Detailed evaluation of as-built conditions and load testing of structures has been performed in a few cases. It is a costly process and can be destructive.

The element of uncertainty, combined with the code of ethics of engineering associations that places public safety as paramount, typically causes engineers to make conservative judgments about structural adequacy. The question of structural adequacy, therefore, remains largely a qualitative issue, with decisions of adequacy based on the premise that deterioration results in strength loss and, therefore, reduced safety margins.

APPENDIX G LIFE-CYCLE COST ANALYSIS

LIFE-CYCLE COST EQUATIONS

This Appendix contains the basic life-cycle cost equations and an example of how some of the equations can be used to assist owners in making decisions on the financial considerations affecting the repair of their buildings. The calculations involved to determine the effects of the time-value of money can be fairly complex and not all owners will need to make such a rigorous analysis. The example given is intended to illustrate the use of life-cycle costing in three particular situations. Owners should complete the appropriate life-cycle cost analyses for their individual needs or obtain expert assistance to help them with their financial decision.

Table G-1 Life-Cycle Cost Equations

TO FIND	GIVEN	EQUATION
F	P	$P(1+i)^n$
F	P	$P(1+e)^n$
P	F	$F\left(\frac{1}{1+i}\right)^n$
P	F	$F\left(\frac{1}{1+e}\right)^n$
F	A	$A\left[\frac{(1+i)^n\left(1-\left(\frac{1+e}{1+i}\right)^n\right)}{i-e}\right]$
A	F	$F\left[\frac{i-e}{(1+i)^n\left(1-\left(\frac{1+e}{1+i}\right)^n\right)}\right]$
P	A	$A\left[\frac{1-\left(\frac{1+e}{1+i}\right)^n}{i-e}\right]$
A	P	$P\left[\frac{i-e}{1-\left(\frac{1+e}{1+i}\right)^n}\right]$
n	A, P	$\frac{\log\left(\frac{1}{1-(i-e)P/A}\right)}{\log\left(\frac{1+i}{1+e}\right)}$

Where:

P = the principal amount at the beginning of n periods
F = the future principal amount at the end of n periods
A = the initial payment of a series of n payments
e = e_1 rate of inflation expressed as a decimal
= e_2 rate of escalation expressed as a decimal
i = rate of interest expressed as a decimal
n = the number of interest periods

In order to simplify calculations, all payment or earning activities in this appendix are deemed to occur at the end of each interest bearing period.

The following case study is presented to illustrate the use of the equations.

CASE STUDY: GARAGE FLOOR REPAIRS

A consultant performs an evaluation of a 3000 m^2 suspended parking garage slab in a 15 year old residential building. The slab has an exposed concrete surface and is fully utilized by residents of the building. The consultant's condition report finds that the garage floor has delaminating concrete at the top surface of the suspended floor slab involving 40 percent of the floor area and is leaking at cracks (300 m in total). The leaks at cracks have resulted in spalling of the soffit (or underside of the slab) typically over a 300 mm width and should be repaired as localized "through-slab" removal and replacement of the concrete. The consultant also finds that there is an additional 10 percent of the underside of the slab that has delaminated, though those areas do not involve damaged concrete on the top and bottom of the slab at the same location of the floor. The consultant presents three technically viable alternatives for remediation.

The three alternatives and the associated costs of the alternatives follow. The unit rates for the costs in this case study are taken from Appendix I.

Alternative 1 - Replace Suspended Slab and Apply Waterproofing

The replacement of the slab will employ current standards for parking garage construction including good drainage, epoxy coated reinforcing steel, good concrete cover and air entrainment in the concrete mix. This new slab is expected to last 40 years. Replacement of the waterproofing will be required after 20 years. No concrete repair is expected within the design life of the new slab.

Alternative 2 - Repair Suspended Slab and Apply Waterproofing

These repairs are expected to extend the life of the slab at least another 20 years and, if diligent maintenance is performed, the slab can be expected to last another 40 years. In 20 years the waterproofing will require replacement. Ongoing repairs to the concrete top surface and underside will be required after five years and subsequently at five year intervals. The consultant estimates that 5 percent of the floor area will be involved in the subsequent repairs at five year intervals, with the exception of through-slab repairs which would not be required after the initial repair.

Alternative 3 - Repair Concrete, Seal Cracks and Apply a Sealer

Alternative 3 is expected to extend the life of the slab at least another 20 years and, if diligent maintenance is performed, another 40 years. However, repairs to the concrete top surface and underside will be required after three years and subsequently at three year intervals for the life of the slab. At those times, the sealer will need to be renewed. The consultant estimates that 5 percent of the floor area (top and bottom) will be involved in the subsequent repairs at three year intervals. Every three years there will also be additional through-slab repairs totaling 5 percent of the initial quantity and crack sealing totaling 10 percent of the original quantity.

Table G-2 Capital Cost Summary

Capital Costs

Description	Unit	Quantity	Unit Cost	Alternative 1 Slab Replacement	Alternative 2 Repair and Waterproof	Alternative 3 Repair and Seal
Sealer	sq m	3,000	$8	N/A	N/A	$24,000
Waterproofing	sq m	3,000	$25	$75,000	$75,000	N/A
Seal Cracks	m	300	$15	N/A	N/A	$4,500
Top Surface Delamination Repair at 40% of Surface Area.	sq m	1,200	$175	N/A	$210,000	$210,000
Underside Repairs at 10% of Surface Area.	sq m	300	$350	N/A	$105,000	$105,000
Through-slab Repairs at Cracks (300 m x 0.3 m)	sq m	90	$350	N/A	$31,500	$31,500
Slab Replacement	sq m	3,000	$330	$990,000	N/A	N/A
Total Initial Capital Cost				**$1,065,000**	**$421,500**	**$375,000**

In each case it is important to consider not only the initial cost but the ongoing maintenance and repair costs associated with the repairs. Owners should specifically request such costs as part of the consultant's assessment of the alternatives for repair work. Operating costs for each of the alternatives follow.

Table G-3 Operating Cost Summary

Operating Costs

Description	Unit	Quantity	Unit Cost	Alternative 1 Slab Replacement	Alternative 2 Repair and Waterproof	Alternative 3 Repair and Seal
Replace Sealer at 3 Year Intervals	sq m	3,000	$8	N/A	N/A	$24,000
Seal New Cracks at 3 Year Intervals (Estimated at 30 m Each 3 Years)	m	30	$15	N/A	N/A	$450
Repair Waterproofing at 5 Year Intervals Assuming 5% of Surface Area is Replaced	sq m	150	$25	$3,750	$3,750	N/A
Top Surface Delamination Repair at 5% of Surface Area Every 5 Years (Alternative 2), Every 3 Years (Alternative 3)	sq m	150	$175	N/A	$26,250	$26,250
Underside Repairs at 5% of Surface Area Every 5 Years (Alternative 2), Every 3 Years (Alternative 3)	sq m	150	$350	N/A	$52,500	$52,500
Remove waterproofing at 20 years(Alternative 2)	sq m	3,000	$20	$60,000	$60,000	N/A
Replace waterproofing at 20 years (Alternative 2)	sq m	3,000	$25	$75,000	$75,000	N/A
Through-slab Repairs at Cracks (Alternative 3)	sq m	4.50	$350	N/A	N/A	$1,575

As stated earlier in this report, the selection of the most favourable of these three alternatives also involves individual owner's financial considerations. To illustrate how these considerations can influence an owner's selection of repair strategies four different owners are considered in this case study.

USE OF PRESENT PRINCIPAL CALCULATIONS

Owner A This owner is a 200 unit condominium corporation that can finance the work out of cash reserves that stand at $1,500,000. The condominium board of directors consider that the building is a long term investment and wish to optimize the value of the money spent on repairs. They also wish to eliminate the amount of ongoing maintenance and the disruption required to maintain the garage in the future.

Owner A has a relatively simple financial situation. Since all funds are immediately available, the owner's financial preference in this decision can be based on the present value costs given by the consultant. Since the building is a long term investment, the planning period considered for our purposes will be 40 years.

Table G-4 Owner A - Present Principal Summary

	Year Work Done	Alternative 1	Alternative 2	Alternative 3
Capital Cost	1	$1,065,000	$421,500	$375,000
Operating Cost for Alternative 1 and 2	5, 10, 15, [20]*, 25, 30, 35	$22,500	$573,750	
Operating Cost for Alternative 3	3, 6, 9, 12, 15 18, 21, 24, 27 30, 33, 36			$1,257,300
Recurring Capital Cost to Rewaterproof	20	$135,000	$135,000	
Present Principal	**P**	**$1,222,500**	**$1,130,250**	**$1,632,300**

* Does not include repair of waterproofing in year 20.

In this case, Alternative 3 offers Owner A the least initial cost at $375,000; however, the anticipated ongoing operating costs for Alternative 3 of $104,775 every three years over 40 years results in a total life-cycle cost of $1,632,300 over the 40 year period. Alternative 3 would also result in repair activities in the garage every three years which is contrary to the objectives of this owner. In this case, Alternative 3 is not a good choice for repair even though the initial costs are the least of the three alternatives.

Since the difference between the life-cycle costs for Alternative 1, (involving replacement of the slab and addition of upgraded protection systems against deterioration), and Alternative 2, (involving repair and waterproofing of the slab) is about $92,000 (about 8 percent) and, since this owner has the capital in the reserve fund to perform the work of Alternative 1, that alternative may be the best choice for them. Other than repair of the waterproofing membrane, Alternative 1 will also avoid disruption of the use of the garage to complete ongoing maintenance and repair work.

USE OF FUTURE PRINCIPAL CALCULATIONS

Owner B This owner has the same 200 unit condominium building as Owner A; however, Owner B has only $400,000 in the reserve fund. That is enough to perform only the work of Alternative 3 leaving very little in the fund for other work. The owner considers that the building is a long term investment and wishes to optimize the value of the money spent on repairs. Therefore, Owner B would prefer to complete Alternative 2 as it has the lowest life-cycle cost. Their consultant advises Owner B that repairs must be performed within five years.

Owner B needs to know how much must be contributed to the reserve fund over the next five years to perform the work of Alternative 2. The funds would earn interest at 6%/year and inflation is 4%/year. The equations used to calculate F are:

$$F = P \times (1 + e_1)^n \text{ and } \quad F = P \times [(1 + e_1) \times (1+e_2)]^n$$

Note that the capital costs for application of the sealer or the waterproofing increase only in response to inflation (e_1), whereas, the capital cost for delamination repair (top, soffit and though-slab) and crack sealing increases as a result of inflation and escalation (e_2) in deterioration. In this example, we must assume an appropriate escalation in the amount of deterioration that would happen over the five year period that the repairs are deferred. In this case, the consultant has predicted that the escalation in deterioration will occur at a rate of 10 percent per year. The cost in five years to do the repair work would be as shown in Table G-5.

Table G-5 Owner B - Future Principal Summary

Capital Costs

Description	Unit	Quantity	Unit Cost	Alternative 2 (P)	Alternative 2 (F)
Sealer	sq m	3,000	$8	N/A	N/A
Waterproofing	sq m	3,000	$25	$75,000	$91,249
Seal Cracks	m	300	$15	N/A	N/A
Top Surface Delamination Repair at 40% of Surface Area.	sq m	1,200	$175	$210,000	$411,481
Underside Repairs at 10% of Surface Area.	sq m	300	$350	$105,000	$205,740
Through-slab Repairs at Cracks (300 m x 0.3 m)	sq m	90	$350	$31,500	$61,722
Capital Cost (P)				**$421,500**	
Capital Cost (F) in 5 Years					**$770,192**

Annual Initial Payment (A)

$$A = F \times \left[\frac{i - e}{(1+i)^n \times (1-[(1+e)/(1+i)]^n)} \right]$$

$$A = F \times 0.164510651$$

Annual Initial Payment A= **$126,705**

Once the future costs of the alternatives are known, Owner B may convert that cost to an annual contribution for the five year period using the initial payment calculation for A shown above.
To accommodate the costs for repair in five years, the monthly increase in the reserve fund contribution to perform the repairs of Alternative 2 must be \$126,705 ÷ (200 x 12) = \$52.79.

USE OF INITIAL PAYMENT CALCULATIONS

Owner C This owner is a private rental landlord who has no cash reserves and must finance the repairs out of rental income. He finds that his allowable rent increase is restricted to a one-time adjustment of \$40 per month (approximately 6 percent of the existing rent) for each of the 200 rental units in the building. The owner intends to sell the building within six years and the building must be in good condition at the time of the sale. For this owner cash flow is critical to the repair decision.

In this case, Owner C does not have the funds available to do the work and thus, must finance the repairs and recover the costs from an allowable fixed rent increase. The amount this owner can recover from rents is \$96,000/year (200 units x \$40/month x 12 months/year). The maximum annual payment (A) to cover the cost for the repairs is, therefore, \$96,000.

Since Owner C intends to sell the building in six years, the planning horizon for Owner C need not be more than six years; thus $n = 6$. For the purposes of these calculations use $i = 6\%$ and $e_1 = 4\%$. The owner will do all the necessary work now, so escalation of the deterioration (e_2) is not a cost concern. Calculations shown below in Table G-6 indicate the Present Principal based on the initial cost and ongoing operating expenses for the 6 year period being considered.

Table G-6 Owner C - Initial Payment Calculation

	Year	Alternative 1	Alternative 2	Alternative 3
Capital Cost	1	\$1,065,000	\$421,500	\$375,000
Operating Cost	5	\$3,750	\$82,500	
Operating Cost	3, 6			\$209,550
Recurring Capital Cost		N/A	N/A	N/A
Present Principal	**P**	**\$1,068,750**	**\$504,000**	**\$584,550**

$$A = P \times \left[\frac{0.06 - 0.04}{1 - (1.04/1.06)^6} \right]$$

$$A = P \times 0.185185149$$

Annual Expense	**A**	**\$197,917**	**\$93,333**	**\$108,250**

The table also shows the calculated value of A, the initial payment, for each of the alternatives. Based on these calculations and assumed interest and inflation rates, Owner C should implement Alternative 2, as it is the only alternative that results in costs within the allowable maximum annual payment of \$96,000 that can be recovered from rent under the repair strategy shown.

It should be noted, however, that, if Owner C is able to sell the property without having to perform delamination repairs or reapply the sealer in year 6, the Present Principal costs for Alternative 3 would reduce to $479,775. In this case, the annual expense for Alternative 3, would reduce to $88,847 which could be recovered from the allowable rent increase of $96,000. Therefore, the future disposition of the property and the future maintenance requirements can have a significant effect on short-term repair strategies.

LONG-TERM CASH FLOW SCENARIOS

Owner D This 'owner' is a 200 unit co-operative housing development that will be seeking a new mortgage to cover the costs for building rehabilitation. The mortgage must cover the cost for the initial capital expenditure. The mortgage company and the co-operative housing development consider that the building is a long term investment and have negotiated a new 20 year mortgage. As such, the life-cycle planning period is 20 years. For the purposes of this example, the amount of the mortgage money allotted to the repair of the garage is $425,000.

Funds put into the capital replacement reserve fund are fixed amount. The owner's contribution to the garage component of the reserve fund will be $22/month/unit ($52,800/year) and will remain fixed throughout the life of the building.

This owner has enough money to perform the work of either Alternative 2 or Alternative 3; however, once the repairs are performed, all future maintenance must be paid for out of the reserve fund allowance. Owner B needs to know which repair scheme will best fit the cash flow available from the reserve fund to cover the ongoing maintenance.

Over the 20 year period being considered, the reserve fund growth due to interest earned and the fixed monthly contributions, as well as payment for work and the escalation of costs due to ongoing deterioration and inflation, would be as shown in Tables G-7 and G-8 for Alternatives 2 and 3, respectively. These tables use the formula for the calculation of the future principal F for the respective repair costs and include the escalation due to increased deterioration for concrete repair and crack repair.

Table G-7 Owner D - 20 Year Cash Flow Considering Alternative 2 Repairs

Year	Annual Reserve Contribution	Interest Earned @ 6%/Year	Waterproofing Repair Cost	Top Surface Delamination Repair Cost	Underside Delamination Repair Cost	Total Expenditures	Reserve Fund Balance
1		$210					$3,500
2	$52,800	$3,168					$59,468
3	$52,800	$6,736					$119,004
4	$52,800	$10,308					$182,112
5	$52,800	$14,095					$249,007
6	$52,800	$18,108	$7,642	$53,492	$122,269	$183,403	$136,513
7	$52,800	$11,359					$200,671
8	$52,800	$15,208					$268,680
9	$52,800	$19,289					$340,769
10	$52,800	$23,614					$417,183
11	$52,800	$28,199	$9,297	$65,082	$148,758	$223,138	$275,044
12	$52,800	$19,671					$347,515
13	$52,800	$24,019					$424,334
14	$52,800	$28,628					$505,762
15	$52,800	$33,514					$592,075
16	$52,800	$38,693	$11,312	$79,182	$180,987	$271,481	$412,087
17	$52,800	$27,893					$492,780
18	$52,800	$32,735					$578,315
19	$52,800	$37,867					$668,982
20	$52,800	$43,307					$765,089
21	$52,800	$49,073	$495,447	$96,337	$220,199	$811,983	$54,979

Table G-8 Owner D - 20 Year Cash Flow Considering Alternative 3 Repairs

Year	Annual Reserve Contribution	Interest Earned @ 6%/Year	Resealing Cost	Crack Repair Cost	Top Surface Delamination Repair Cost	Underside Delamination Repair Cost	Through-Slab Delamination Repair Cost	Total Expenditures	Reserve Fund Balance
1		$3,000							$50,000
2	$52,800	$6,168							$108,968
3	$52,800	$9,706							$171,474
4	$52,800	$13,456	$28,077	$848	$49,457	$113,044	$2,967	$194,393	$43,338
5	$52,800	$5,768							$101,906
6	$52,800	$9,282							$163,988
7	$52,800	$13,007	$31,582	$954	$55,632	$127,159	$3,338	$218,665	$11,130
8	$52,800	$3,836							$67,766
9	$52,800	$7,234							$127,800
10	$52,800	$10,836	$35,526	$1,073	$62,579	$143,037	$3,755	$245,969	($54,533)
11	$52,800	($104)							($1,837)
12	$52,800	$3,058							$54,021
13	$52,800	$6,409	$39,962	$1,207	$70,392	$160,897	$4,224	$276,682	($163,451)
14	$52,800	($6,639)							($117,290)
15	$52,800	($3,869)							($68,360)
16	$52,800	($934)	$44,952	$1,357	$79,182	$180,987	$4,751	$311,229	($327,723)
17	$52,800	($16,495)							($291,418)
18	$52,800	($14,317)							($252,935)
19	$52,800	($12,008)	$50,564	$1,527	$89,069	$203,586	$5,344	$350,090	($562,234)
20	$52,800	($30,566)							($540,000)
21	$52,800	($29,232)							($516,432)

These tables show that repairs employing Alternative 3 would result in a deficit the third time that maintenance is performed. However, repairs employing Alternative 2 would result in a surplus in the fund after 20 years. The tendency of some owners in circumstances such as that of Owner D might be to implement Alternative 3 initially, to take advantage of the lower initial cost, and then not perform the required ongoing maintenance of the slab because of the lack of funds.

These life-cycle cost scenarios describe some of the ways that some of the simpler formulae given in Table G-1 at the beginning of this Appendix can be used by owners to determine the proper course of action for them. The more complex calculations involving the more complex formulae in Table G-1 should be undertaken only by those with actuarial skills.

EFFECT OF INCOME TAX ON LIFE-CYCLE COSTING

Thus far in this review of life-cycle costing the concept of asset depreciation has not been considered. *Depreciation,* for our purposes, is simply a way that "for-profit" corporations reduce the net worth of assets over time and, therefore, reduce their tax liability. Capital expenses, such as major repair work to the parking garage of a building can be considered as investments in the property that can be used, through the permitted depreciation allowance, to reduce taxable income from the building.

For instance, Owner C could consider the effects of the available depreciation allowance and the expensing of operating costs against earned income on the cash flow available to do the work.

The effect that reduction in taxable income and depreciation have on the economic consideration of alternatives may change the preferred choice or make the preferred alternative even more attractive. Since consideration of taxable income reduction through depreciation allowances can alter the selection of the best financial alternative, the owner should obtain advice from the appropriate financial specialist.

APPENDIX H CATHODIC PROTECTION

PROTECTION THEORY

Since corrosion is an electrochemical process involving the oxidation of reinforcing steel at anodes, it seems reasonable that if a reinforced concrete system could be transformed completely to a cathodic condition, corrosion would not occur. In theory, this is made possible by the supply of direct current (I_{cp}) which is made to flow through the concrete electrolyte to the reinforcing steel. The charge in the corrosion cell flows from anode to cathode. As the I_{cp} charge is applied, the electrical potential (E_p) at the concrete/reinforcing steel interface polarizes and corrosion current is reduced.

The typical method of supplying the direct current to reinforced concrete parking slabs on buildings employs a soffit applied conductive coating to distribute current supplied by fine surface mounted wires connected to the DC power supply. See Figure H-1. The soffit anode is connected to the DC supply and the DC supply is connected to the reinforcing steel mat. The impressed current I_{cp} is distributed through the conductive coating to the reinforcing steel via the concrete ideally as a uniform current density i_{cp}

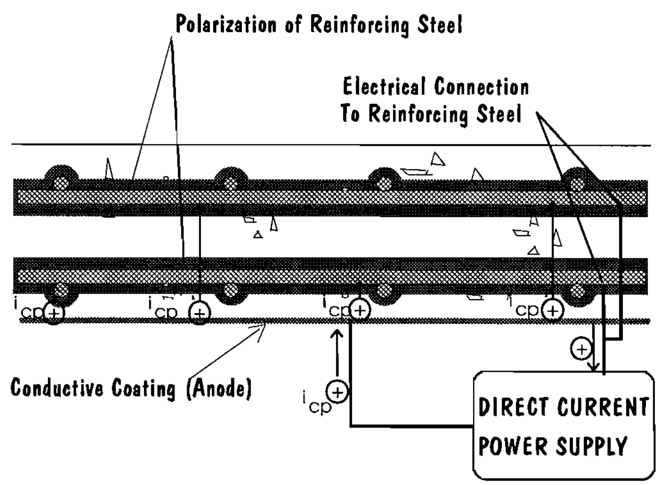

Figure H-1: Cathodic Protection of Garage Slabs

Cathodic protection systems used to protect concrete structures require electrical continuity of the reinforcing steel. Discontinuities may exist due to loose reinforcing steel bars or partial use of epoxy coated reinforcing steel. Those discontinuities must be eliminated for proper system performance.

One of the effects of the impressed current systems is the production of hydroxyl ions (OH) at the reinforcing steel. In addition, the positive soffit anode attracts the negatively charged chloride ions (Cl^-) moving them away from the reinforcing steel.

Prior to installation of a soffit mounted cathodic protection system, the soffit of the concrete slab must be tested for potential short circuits that may occur between the applied anode and the reinforcing steel mat. These usually occur in four ways:

- metal support chairs for the reinforcing steel,
- steel tie wires that touch the soffit of the slab,
- poorly placed reinforcing steel bars that are in close proximity to the soffit
- metal brackets and other soffit mounted metal fixtures

Since the cathodic protection system will not operate effectively if short circuits exist, all shorts must be located and isolated from the soffit anode.

Monitoring of the performance of cathodic protection systems is needed in order to make adjustments to the supply current, assess performance and make repairs as components of the system become damaged or deteriorate. There are differing views as to the appropriate methods for monitoring and criteria for assessing performance. The National Association of Corrosion Engineers (NACE) a well-established association holds a somewhat more conservative view than that presented by NRCCPA[18] a recently formed Canadian industry association. Some of these issues remain unresolved.

APPENDIX I TYPICAL COSTS FOR REPAIR AND PROTECTION OF CONCRETE

Based on the experience of the author, the following can be considered as appropriate unit costs to undertake concrete repair work. Unit prices will vary according to region. The following costs are based on 1994 values.

CLEANING

Cleaning concrete surfaces of dirt, stains, efflorescence and leach deposits is normally accomplished by pressure washing with water. Depending on the extent of cleaning required, unit prices can range between \$0.50 to \$2/m^2. This should not be confused with cleaning of repair areas prior to application of repair concrete, the costs for which are included in the unit prices for that item as given below.

SURFACE/SOFFIT PATCH - NORMAL PORTLAND CEMENT

Concrete repairs using normal Portland cement are typically appropriate if relatively large volumes of repair are required. Surface patches on slabs require removal of deteriorated concrete to a depth normally exceeding 25 mm and often up to 75 mm. In most cases, structural concrete will be removed to expose the reinforcing steel. Surface patches on horizontal surfaces, such as the top surface of parking slabs, will generally range in cost from \$130 to \$180/m^2 for depths up to 75 mm. This includes removal, surface preparation, and new concrete.

Costs to repair the edges of balcony slabs that are spalling due to corrosion of the embedded metals typically costs \$160 to \$200/m of length for repairs in the order of 150 to 200 mm deep. If the repair does not require removal of the edge of the slab and only isolated, thin patches are needed, such as may be the case if the edge reinforcing steel was placed with too little concrete cover, the costs for repair would be in the order of \$275/m^2 of balcony slab face.

Surface patches on vertical and overhead surfaces, such as walls, columns and the underside of suspended slabs, normally require form work. These repairs are, therefore, somewhat more labour intensive having costs which range from \$275 to \$450/m^2. On some occasions shotcrete is used. Shotcrete is sand, cement and water applied at a fairly low water-cement ratio under pressure through a nozzle. It is sprayed or 'shot' into its final position. This eliminates the need for forms; however, the costs remain relatively high at \$325/m^2 due to the nature of the application.

LATEX MODIFIED PATCHES

Latex modified patches less than 25 mm deep are most often used for small, patches in concrete surfaces for horizontal, vertical and overhead applications. Patching using latex modified concrete on the top surface of slabs is usually in the range of \$100/m^2. Vertical and overhead patching will range in cost from \$375 to \$475/m^2. Form work is often not necessary due to the small, shallow nature of the patch and the consistency of the latex mortar.

The relatively high unit costs are directly related to the cost for the latex additive; as such, latex mortars are used sparingly.

ROUT AND SEAL CRACKS

Routing and sealing the cracks in suspended slabs or slabs on grade will cost in the order of \$15 to \$20/m.

EPOXY INJECT CRACKS

Epoxy injection of structural cracks in concrete will usually cost in the range of \$75/m.

CHEMICAL INJECTION OF CRACKS

Chemical injection of cracks in concrete to stop water leakage will cost in the order of \$100/m for the chemical gel products. The gel products may require preparation of the crack to be injected. Plastic ports (occasionally metal) are inserted at 150 to 200 mm spacing along the length of the crack to be injected. Water may be injected first to clean the crack and to promote the chemical reaction.

These products are becoming more commonly used than the chemical crystalline products that have typically been in the order of \$50/m. The difference in cost is due largely to the cost of the products. The crystal growth products require that surface of the crack be routed out. The product is then applied as a paste to fill the crack.

SURFACE SEALERS

The application of a surface sealer to either a slab-on-grade or a suspended slab requires surface preparation of the concrete to remove dirt and laitance. This is usually accomplished by shot-blast (small, reused steel spheres shot against the surface of the concrete within an enclosed housing). Including surface preparation, the application of a sealer will be in the order of \$8/m^2.

SURFACE WATERPROOFING

The cost of waterproofing of a parking garage floor is dependent on the type of waterproofing system chosen. Asphalt-based waterproofing systems typically cost in the order of \$16 to \$25/m^2. Elastomer-based waterproofing systems will cost in the order of \$18 to \$27/m^2.

Removal of existing waterproofing may be necessary if it is deemed to have failed These costs do not include the removal of any previously applied waterproofing system which can be in the order of $16/m^2 to remove thin elastomer systems and $38/m^2 to remove asphaltic systems with a thick wearing course.

The cost for waterproofing a parking garage roof slab that is covered by landscaping, paving and other overburden will require removal of the overburden to expose the waterproofing. Generally, the cost to re-waterproof a roof slab will be in the order of $20/m^2. The overall cost including excavation and replacement of the overburden will be in the order of $100//m^2. Landscaping and plantings, brick and stone pavers and architectural features that may be removed and require renewal are not uncommon when re-waterproofing of a garage roof slab is required. Costs for such items must also be considered on a building-by-building basis.

The cost for addition of waterproofing to a below-grade foundation wall including excavation, waterproofing, and backfilling will be in the order of $400/m of wall length. Should a shored excavation be required these costs can increase to $1,000 to $1,500/m of wall length.

CATHODIC PROTECTION

The most widely used system of cathodic protection applied to buildings employs the conductive coating on the soffit of the slab. Costs for these systems vary from $40 to $50/m^2. Additional costs may be incurred to provide electrical continuity within the reinforcing steel mat and eliminate shorting of the system against exposed steel such as reinforcing steel support chairs.

REPLACE COMPONENT

On occasion the deterioration or damage may be so severe as to make repair impractical. It will cost in the order of $325 to $450/m^2 to remove and reconstruct a structural slab, such as a floor slab of a parking garage. This cost would include the shoring of the structure and form work for the new slab.

Repair of the delamination on the soffit of a parking garage floor slab, most often at areas coincident with top surface delamination, is frequently conducted as a "through-slab" repair involving replacement of concrete for the full depth of the slab in the affected areas. Such repairs cost in the range of $250 to $350/m^2.

Replacement of balcony slabs, particularly those constructed in the 1950s and earlier that employed an extension of the top chord of the steel floor joist, can cost in the order of $650/m^2 of slab area.

Replacement of a column or wall is unusual. Unit costs would be unique to the particular circumstances.

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