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Failure, distress and repair of concrete structures

Edited by Norbert Delatte

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Failure, distress and repair of concrete structures

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Introduction

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I am pleased to introduce you to this book that has been assembled by an international panel of experts. Worldwide, the investment in concrete infrastructure is vast. Mindess *et al*. (2003) observe that concrete is the predominant construction material. Concrete is said to be the most used material after water.

Concrete is often selected for its durability and long life, together with its low projected maintenance costs. However, poor design decisions, improper materials, or inadequate construction practices can considerably reduce durability, leading to the need for repairs. Alternatively, well-designed and built structures and facilities may be subjected to extraordinary loads and impacts or other unplanned distress and damage. This also triggers the need for repairs.

As our concrete infrastructure ages, the need for repairs will increase, and concrete repair will represent an expanding fraction of civil engineering practice. Two decades ago, the article 'Concrete durability a multibillion-dollar opportunity' (1988)* noted:

> A problem exists with respect to inadequate durability of concrete in service. The reasons for the problem are both technical and institutional. The manifestation of the problem is premature distress and degradation of concrete in structures. Many different physical and chemical processes, including corrosion of steel reinforcement, sulfate attack, freezing and thawing, and alkali-aggregate reactions, interact to bring about premature deterioration.

This article further noted that the investment in concrete infrastructure in the USA alone, at that time, was roughly $6 trillion. The need for rehabilitation was growing, and rehabilitation should be carried out using state-of-the-art technology.

The article described a number of disturbing trends that have unfortunately become worse instead of better in the intervening two decades. These include:

- Lack of attention to concrete repair in undergraduate civil engineering curricula, and even a decrease in the content of materials science and concrete technology taught.
- In parallel, lack of investment in training by the construction industry.
- Fragmentation of the concrete industry, with fewer individuals able and willing to address the overall picture. This includes separation of design and construction functions.
- Focus on short-term financial returns as opposed to long-term performance.

So, the situation is getting worse – or, perhaps, better for the repair contractors. Timely, well-executed repairs have an important place in the concrete durability opportunity. One useful concept, discussed on page 28 of *Durability of Materials and Structures in Building and Civil Engineering* (Yu and Bull, 2006), is the 'Law of Fives.' This states that, as a rough estimate, '$1 spent on design and construction is equivalent to $5 spent as damage initiates and before it propagates, $25 once deterioration has begun to propagate, and $125 after extensive damage has occurred.' In that context, this book focuses on the $5 preservation efforts and the $25 repairs before we are faced with the $125 problems.

I would also caution that poorly designed and executed repairs may, in fact, exacerbate problems. One example would be filling a working crack with a stiff crack sealing material. The movement of a working crack indicates that it is functioning as an unplanned stress relief joint. Filling the crack, then, would simply cause the stress to build up again, requiring relief again. One would therefore expect to quickly find a new working crack, parallel to the old one. In this case, often the best thing to do would be to do nothing and simply observe the crack, if the function of the structure or facility is not impaired.

Another example would be the inappropriate use of an impermeable sealant or overlay. These have the useful function of keeping water out of the concrete. However, they also keep moisture in and, if trapped moisture is in fact the problem, the repair will accelerate any moisture-related degradation.

Before initiating a repair, then, it is vital to know what is going on. Thus, Part I of this book addresses the various failure and deterioration mechanisms of concrete. The need for repair may be initiated by deterioration, defects, or damage. Cracks in concrete have a variety of causes. Some can be safely ignored, and some cannot, and it is important to be able to recognize the difference. There are also a number of tools and non-destructive testing technologies available for condition assessment of concrete structures.

Once the cause of the problem has been determined, it is then possible to design and execute a proper repair. This is the subject of Part II of this book. Some international standards are available or under development for repair. Methods are available for repairing cracks in concrete. Selection of appropriate repair materials is very important for performance. Sometimes conventional concrete can be used, and sometimes unusual or proprietary products may be necessary. Specifying and selecting materials requires careful consideration – whereas conventional concrete is often specified primarily on the basis of compressive strength, compressive strength of a repair material may not be among the top ten properties of interest. In fact, excessive strength or stiffness may lead to concentration of stress and to the failure of the repair.

The repair techniques discussed in detail in the various chapters of Part II include bonded concrete overlays, fiber-reinforced polymers, protection systems, and patching. Each technique has appropriate and inappropriate applications, as well as particular considerations for success. Some specific applications and case studies are also provided. The durability of repairs is also addressed, because it is particularly disheartening to have to repair a repair.

In a work of this nature, there are inevitable overlaps and perhaps a few contradictions. Sometimes, even the experts disagree on solutions to complex problems. However, I believe that this will be a very useful work for those charged with the difficult task of protecting and preserving our investment in concrete infrastructure.

*Concrete durability a multibillion-dollar opportunity, (1988), *Concrete International*, American Concrete Institute, **10**(1), 33–35.

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1

Causes and mechanisms of deterioration in reinforced concrete

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Abstract: The physically and chemically induced deterioration causes and mechanisms of reinforced concrete structures are discussed. Physically - induced deterioration is caused by freeze–thaw loads, non-uniform volume changes, temperature gradients, abrasion, erosion, or cavitation. Chemically-induced deterioration consists of carbonation, corrosion of steel reinforcement, sulfate and acid attacks, or alkali–aggregate reactions. The role of moisture and microstructure of different concrete types is also considered.

Key words: deterioration of concrete, microstructure, moisture, reinforcement, physical attacks, chemical attacks.

1.1 Introduction

The overall durability of concrete is relatively good; otherwise it could not have become the most abundantly used building material in the world. Concrete is also a building material whose durability properties can be improved quite easily and in a versatile manner simply by selecting its constituents appropriately or by using proper admixtures. However, these possibilities have not been exercised sufficiently because too often the cheapest possible concrete type has been selected for the structure, despite the demanding environmental conditions to which it will be exposed. Not only have economical considerations during the building phase governed the choice of concrete type, but also lack of knowledge of different deterioration mechanisms and of the ways to improve the durability properties of concrete have been the reasons why the service lifespan of many contemporary concrete structures has been unexpectedly short.

At the global level, the cost of renovating deteriorating concrete structures is huge. It is usually the case that relatively inexpensive measures taken at the design and erection stage could have increased the service lifespan of these structures by a factor of two or even three.

In this chapter, the causes and deterioration mechanisms will be reviewed. The emphasis has been on the most important deterioration features which affect large volumes of concrete structures.

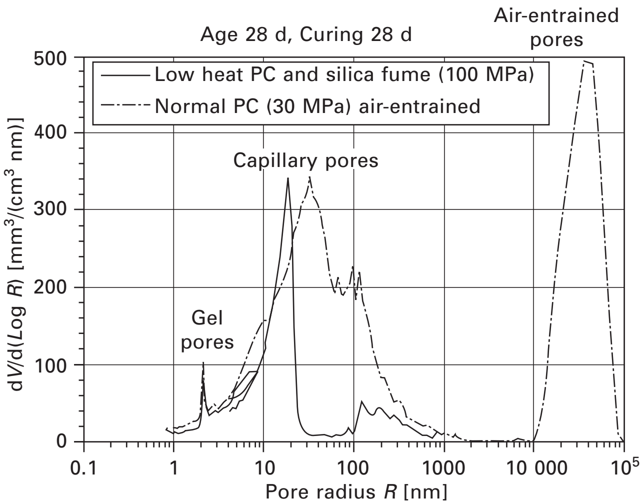
1.2 Microstructure and role of moisture in concrete deterioration

Concrete can be considered as one of the most non-homogeneous and demanding engineering materials used by mankind. This man-made artificial stone consists of aggregates having a wide dimensional range from sub-micron sized particles to several centimeters. Aggregate particles are usually surrounded by a highly porous transition zone, differing from the less porous bulk matrix of hydrated and partially hydrated cement paste. Void dimensions range from nanometer sized gel pores of calcium silicate hydrates to micrometer sized capillary pores and larger pores of several millimeters covering a dimensional range of over six orders of magnitude (Fig. 1.1). The pore system is partially filled with water.

The glue phase of calcium silicate hydrates is also very complicated, comprising amorphous and crystalline phases. One of the general features of concrete is the large number of cracks that are induced into it in normal climatic conditions. These defects are partly due to shrinkage tensions caused by the contraction of matrix gel and by moisture and temperature gradients between different parts of the concrete structure.

1.2.1 Effects of water–binder ratio and different binders

Almost all the durability problems of concrete are related to the pore size distribution of binder paste and to the degree of filling of pores by liquid.



1.1 Pore size distribution of binder pastes. Gel pores are measured by nitrogen adsorption, capillary pores by mercury intrusion porosimeter, and air-entrained pores from thin section data.

These features affect the pore water transport and subsequent pressures generated into the concrete matrix (concrete skeleton). How the deleterious substances can penetrate into concrete is mainly governed by the dimensions of the entrances between larger capillary pores and by the air-filled portion in the pore system. Cracks on the outer surface of the concrete structure also increase the penetration of water and deleterious substances into concrete. When the intruding deleterious substance is in the gaseous phase, diffusion takes place much faster if a considerable part of the pore volume is air-filled.

Depending on the deterioration mechanism, there is usually a relative humidity range measured in the pore system, during which the deterioration mechanism proceeds at an increased rate. For example, the carbonation rate of concrete is highest at 50–70% relative humidity in ambient air.

When the penetrating deleterious substances are in ionic form, the penetration rate depends on the concentration differences in pore water between the surface and interior concrete. Also, pressure difference between the different parts of the structure can act as the driving force which governs the penetration rate of the deleterious substances. In this type of transport mechanism, the dimensions of the entrances between much larger capillary pore voids and the size of the intruding molecules govern the permeability of concrete.

In the case of the diffusion mechanism, Fick's first law is not adequate to model the transport mechanism of concrete, because there is also a counter-diffusion of ions having opposite charge which strongly affects the process. Similarly, in the case of liquid penetration, Darcy's law has been proven to be too simple. The effect of entrance dimensions between capillary pores and the air volume in them cannot be modeled with proper accuracy simply by using the material properties such as water/cement ratio or air content.

Drying and wetting of concrete pore structure has a significant effect on the penetration rate, and this complicates the transport mechanism. Therefore, simple first-order diffusion and transport formulae cannot be applied in analyzing the deterioration rates in concrete. Additionally, the introduction of large air-filled protective pores (dimensions 0.02–0.1mm) by air-entraining admixtures successfully hinders the capillary flow of pore water and, therefore, air-entrainment affects all types of corrosion mechanisms in which deleterious substances intrude inside concrete from the surface of the structure.

If the binder is pure Portland cement, capillary pore volume can be calculated quite accurately as a function of water/cement ratio, hydration degree, and air content of fresh concrete. However, the effects of the entrance dimensions between larger capillary pore voids are not known sufficiently well to calculate the diffusion or permeability coefficients. The effect of additional binders such as condensed silica fume on total capillary porosity can be calculated but, again, knowledge of how it affects the entrance dimensions is not adequate. The effects of other secondary cementitiuous materials such as fly ashes are known to an even lesser degree.

Hydration of ground granulated blast furnace slag (GGBS) produces in concrete a microstructure which differs remarkably from that when Portland cement alone is used. Hydration of GGBS resembles a surface reaction of the binder particles. If proper curing measures are applied, the permeability of slag concrete is much smaller compared to Portland cement-based concrete when both concrete types are produced with the same water/binder ratio. However, carbonation of slag concretes coarsens the pore structure and, during later ages, permeability of slag concretes is increased (Matala, 1995). Carbonation also increases the permeability of concretes in which secondary cementitious materials have been used. Such pozzolanic materials are condensed silica fume and fly ash. Additional hydration of cracked concrete can sometimes heal the structure if there is moisture available. In regions subject to freeze–thaw deterioration in particular, this healing during summer times can increase the lifespan of the structure.

The variation in diffusion or permeability coefficients in different concretes produced by different cements and secondary cementitious materials is quite large. However, decreasing the water/cement or water/binder ratio and increasing the curing time makes all concretes more impermeable against penetration of deleterious substances and, thus, the estimated lifespan is increased. There exists also large deviation in the estimated lifespan due to different deterioration mechanisms. Therefore, no general rules can be given and different deterioration rates have to be assessed for different corrosion mechanisms.

1.2.2 Pore water transport and pore water tension-induced stress state

The pore system of concrete is seldom saturated by pore water. Therefore, there is a tensional stress state in pore water which causes compression in concrete. The tension in pore water can be calculated by applying Kelvin's equation:

$$p - p_0 = \frac{R \cdot T}{v_w} \cdot \ln \frac{p_v}{p_{v0}} \qquad [1.1]$$

in which R is ideal gas constant 8.314 J/mol/K, T is temperature in K, v_w is specific volume of water in m^3/mol, p_v is vapor pressure of pore water, p_{v0} is the saturation vapor pressure, and p_0 is atmospheric pressure 0.1 MPa.

When concrete begins to dry from the surface, the pore water tension generated causes compression on the surface concrete layer which decreases the permeability of the surface. However, during cyclic moisture loads, this compression is alleviated and cyclic moisture loads increase moisture transport through the surface layer. Then, the moisture flow is significantly increased compared to the situation when concrete dries without wetting and drying cycles.

During freeze–thaw loads, the transport of water causes considerable pressure differences in the concrete cross-section which can exceed the tensional capacity of concrete. Similarly, cyclic freeze–thaw loads can pump large quantities of external water into concrete, causing increased freeze–thaw stresses.

The inside of concrete also dries due to hydration causing self-desiccation, which generates large pore water tensions inside the structure. Together with shrinkage stresses, these generate cracks into concrete which can increase the transport of deleterious substances into it.

1.3 Physically induced deterioration

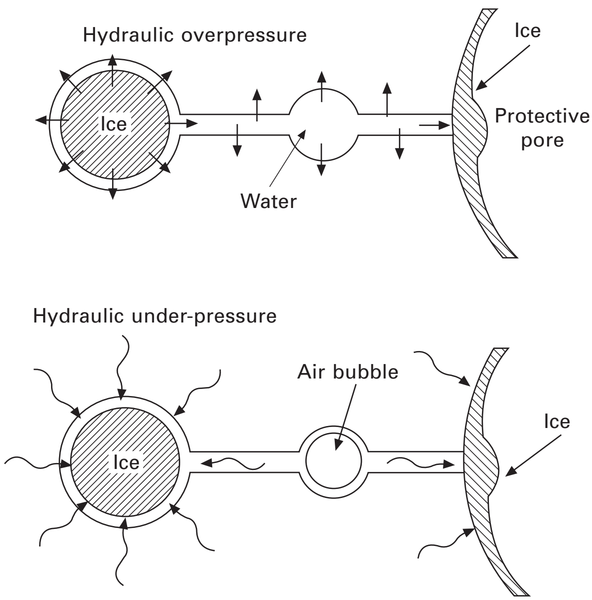
Corrosion loads and mechanisms of concrete and reinforcement can be divided into those of external and those of internal origin. External origin mechanisms usually include freeze–thaw deterioration, chloride ingress, carbonation, sulfate attacks, and acid attacks. Effects of alkalis on certain aggregates or deteriorations due to delayed ettringite reaction are included in internal origin mechanism. However, in this chapter the deterioration mechanisms are divided into physically and chemically induced mechanisms according to the main deterioration feature. The initiation of the deterioration mechanism can be caused by a single cause but, thereafter, the deterioration can continue as a combination of several other mechanisms. It can be quite difficult to assess which is the original cause of the initial corrosion.

1.3.1 Freeze–thaw deterioration

Two types of deterioration mechanisms are apparent in concrete structures exposed to freeze–thaw loads. In surface scaling, flakes of the outer surface disintegrate and the other mechanism, internal damage, is associated with cracking on the surface or interior of the structure. The causes of these mechanisms are different.

Large surface scaling is observed in concrete structures produced by a high water/cement ratio and low air content when the moisture content in the surface layer is high. The moisture content in the surface layer is high in situations when water is in direct contact with the surface or where there is heavy rain falling on the concrete structure. In these situations, cyclic freeze–thaw loads cause a pumping effect which increases the moisture content and degree of filling of the pore system at the surface to a large degree. If the air-filled pore volume at the surface is small and the air-filled pores are situated far apart (> 0.4mm), the expansion of about 9%, when water initially freezes, causes hydraulic overpressure in the pore water (Fig. 1.2). This pressure causes disintegration at the surface of the structure, a phenomenon which is related to the concept of critical degree of saturation (Fagerlund, 1977). If the degree of filling of the pore system exceeds the critical degree of saturation, even one freeze–thaw cycle causes deterioration in concrete.

In concrete structures it is usually the case that water content and degree of filling of the pore system are smaller inside the concrete. Therefore, the hydraulic pressure during the initial freezing temperature is smaller and no deterioration takes place. In fact, when there is ice present in the pores of the surface layer, the unfrozen water in the interior of the structure migrates towards the surface layer. This causes a tensional stress state in pore water of the interior portion of the structure. The reason for the water migration is the lower chemical potential of the ice surface compared with the chemical potential of the unfrozen pore water situated in smaller pores inside the structure. This feature also causes micro-ice-lens formation in the pores of concrete, as presented by Setzer (1999, 2001).



1.2 Water pressure mechanisms in freezing concrete. Hydraulic overpressure of pore water at the surface layer of concrete is the main deterioration mechanism in surface scaling while differences of pore water tension (under-pressure) at different locations in the concrete cross-section are responsible for internal damage in freeze–thaw deterioration. (Penttala and Al-Neshawy, 1999)

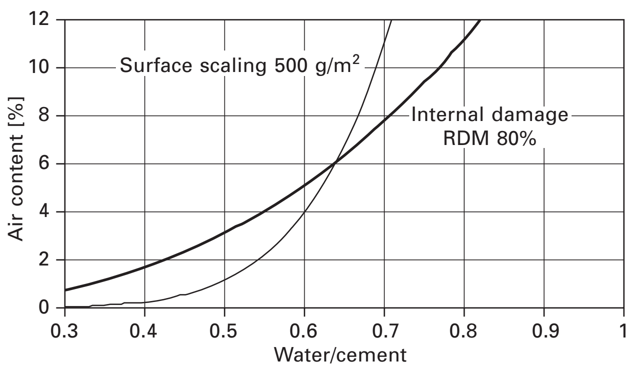
When concrete is produced by using a small water/cement ratio, the capillary pore volume is smaller and the permeability of the surface layer is also decreased, which decreases the moisture content in the surface layer. This reduces the need for air-filled protective pores into which pore water is squeezed by the hydraulic over-pressure, and the water can freeze there without causing surface scaling. Then, the internal damage mechanism becomes the decisive mechanism which governs the need for air-entrainment (Fig. 1.3).

When there is ice in the pore structure of concrete and no additional salts are present in the pore water, pore water tension can be calculated according to Equation [1.2] (Penttala, 1998):

$$p - p_0 = \frac{R \cdot T}{v_i} \ln\left(\frac{p_v}{p_{v0}}\right) + \frac{\Delta h_{wi}^0}{v_i \cdot T_0}(T - T_0) + \frac{1}{v_i} \cdot \int_{T_0}^{T} \int_{T_0}^{T} \frac{c_{pw}^0 - c_{pi}^0}{T} \, dT dT \qquad [1.2]$$

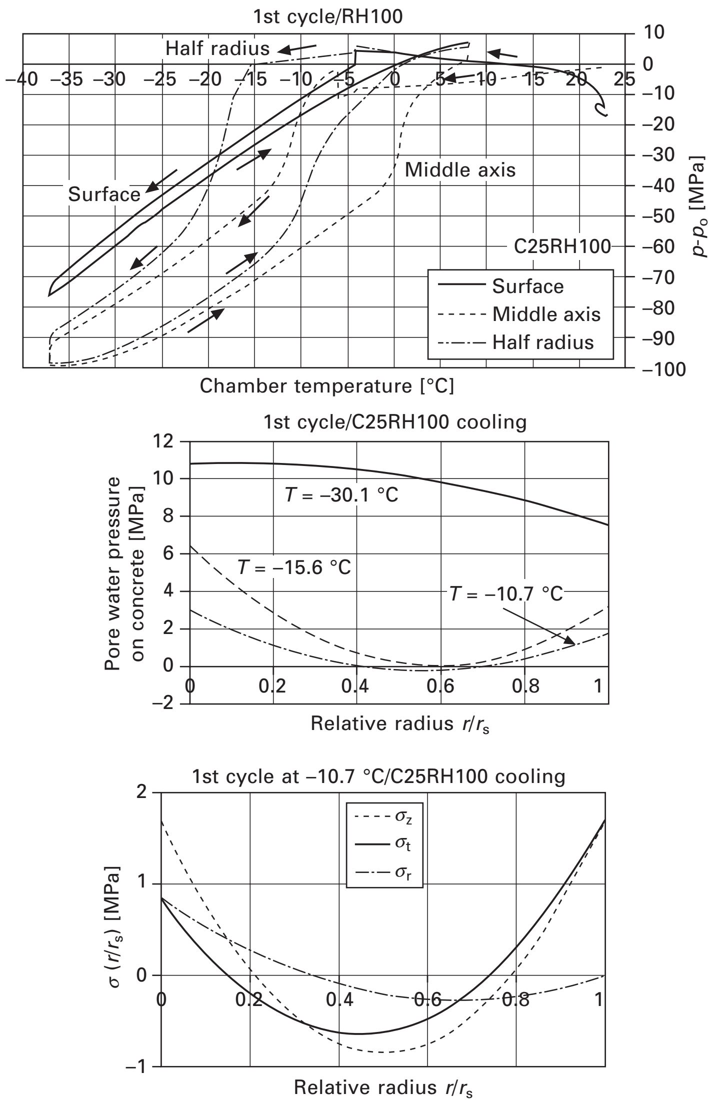
Heat of solidification is $\Delta h_{wi}^0 = 6.0 \times 10^3$ J/mol, v_i is specific volume of ice in m^3/mol, and c_{pw}^0 and c_{pi}^0 are specific heat capacities of water and ice, respectively. The reference temperature T_0 is 273.16 K and the reference pressure p_0 is 0.1 MPa. Pore water tension causes compression in concrete. Air-entrainment causes a higher three-dimensional compressive stress state in concrete compared to non-air-entrained concretes. This hinders crack formation and even closes cracks that have been generated (Penttala, 2007).

In the internal damage mechanism, the pore water tension differences and



1.3 The relation of the needed air-entrainment in the two deterioration mechanisms of concretes as a function of water/cement ratio. Tests are performed by the slab test method (Swedish standard SS13 72 44–1515, 1988) and the curves are calculated by statistical means (Penttala, 2006).

subsequent compression differences in concrete at different locations of the structure cause tensional stresses into concrete. If they exceed the tensional capacity of concrete, cracks are generated (Fig. 1.4).



1.4 Pore water tension as a function of chamber temperature (a). Pore water pressure distributions on concrete (b) and the stress state distributions (c) at the temperature of –10.7 °C as a function of the relative radius of a cylindrical test specimen. The non-air-entrained concrete test cylinder was cured under water and had a compressive strength of 25 MPa (Penttala, 2007).

Air entrainment

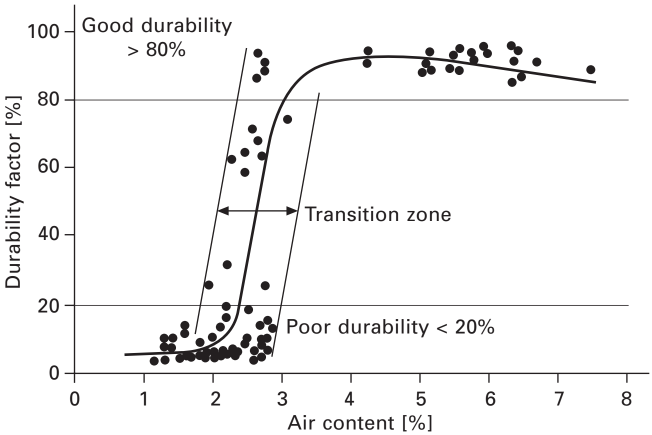
Adequate freeze–thaw durability of concrete can be assessed by increasing the air content in binder paste by air-entraining admixtures. The optimal dimension of these protective air pores ranges from 0.02–0.05 mm. With this size, they stay air-filled in normal moisture load conditions and effectively hinder the capillary flow of pore water in concrete. Also, the strength reduction caused by the pores is still reasonably small.

The air-entraining admixture should be compatible with the concrete composition and with the possible other admixtures used, so that the dispersion of the protective pores in the binder paste is sufficiently homogeneous and the distance from one air-filled pore to the nearest other air-filled pore is sufficiently short (in normal concretes smaller than 0.35–0.45 mm). The spacing factor and the pore size distribution can be assessed by optical means from a thin section test sample cut from hardened concrete, but continuous quality control is usually performed by total air content measurements from fresh concrete by the pressure method.

Normal aggregates are usually durable against freeze–thaw distress and their porosity is rather small. Binder paste is the vulnerable portion of the concrete matrix, and it should possess adequate protective pore size distribution and total air content. Therefore, when freeze–thaw durability in continuous quality control is assessed by air content, the acceptance air content value depends on the relative volume of the binder paste in concrete. The relative binder paste volume is mainly a function of the binder content and maximum aggregate dimension. The water/cement ratio and maximum aggregate dimension are the variables normally used in determining the total air content required in concrete to ensure freeze–thaw durability. If there are no additional salts in the pore water, the air content measured from fresh concrete should exceed 4.5% when the maximum aggregate dimension is 32mm (Fig. 1.5). A sufficient average value can then be 5.5% in which the distribution of the spacing factors of the protective pores has been taken into consideration.

In the estimated service lifespan assessment presented later in the chapter, the sufficient air content value is presented in Table 1.2 as a function of water cement ratio and maximum aggregate size.

The air-entraining admixture dosage required depends on several variables. These include, for example, the coarseness of the cement, quality and type of the secondary cementitious binders, amount of blast furnace slag, fineness and amount of fillers, temperature of concrete, transport distance of fresh concrete, water/cement ratio of the concrete, consistency of the mix, possible pumping of concrete, and other admixtures used in the mix. Depending on the concrete constituents, the air-entraining admixture dosage can be varied and there are national restrictions in the use of some ingredient materials.



1.5 Effect of air-entrainment on the freeze–thaw durability of concrete when the freezing liquid is pure water (Cordon, 1967).

For example, if the carbon content in fly ash is high (> 7%) or changes much between different delivery lots, it is not advisable to use it in the production of freeze–thaw durable concrete.

Effect of salts

If there are salts on the surface of concrete during freeze–thaw loads or if additional salts have intruded into the pore water, the deterioration is significantly more severe compared with normal concretes. Salts increase the surface scaling of the structure, and the deterioration can proceed very rapidly after only few freeze–thaw cycles. This seems to indicate that the main deterioration mechanism in salt-freezing is of physical origin. There are several research results that seem to indicate that a 2–4% salt concentration in the surface water layer causes severe deterioration and, if the salt concentration is higher, the deterioration decreases (Verbeck and Klieger, 1957). However, some salts, especially calcium chloride, in large concentrations and after long exposure, can also cause chemical corrosion in concrete.

When there are salts dissolved into the pore water of concrete, the concentration difference between pore water and external salt-free water or moisture draws more water into the pore system. This increases the freezable water content in concrete considerably and the degree of filling of the pores is increased. Similarly, salts increase the pore water tension in the internal damage mechanism of concrete. The additional dissolved ions in pore water decrease the freezing point, but the degree of filling of the pores of salted concretes is much higher and larger pores are filled with water compared with unsalted comparison concretes preserved in similar environments. Therefore, the initial freezing temperature of normal concretes and salt-freezing concretes are about the same (Penttala, 1999). Due to super cooling at the initial freezing temperature of the pore water, the freezing ice amount in salted concretes can be several times larger compared with unsalted comparison concrete and, therefore, the hydraulic pressure generated in the surface layer is also much larger. This explains the more severe surface scaling in the salted concretes.

During ice formation, the ice structure is nearly free of salts and the salt concentration increases in the unfrozen pore water. If the initial salt concentration in the pore water is large, the salt concentration in the unfrozen pore water situated in smaller pores can reach the eutectic temperature of the salt combination. When temperature decreases further, salts begin to precipitate, causing large crystallization pressures against the pore walls. This phenomenon causes internal damage in concrete. Similarly, the large concentration differences in unfrozen pore water between the surface layer of the structure and interior concrete into which salts have not been able to intrude cause osmotic pressures in pore water and pore water transport which generate internal stresses in concrete. These mechanisms explain the more severe deterioration of concretes exposed to salts.

At the initial freezing temperature of the pore water, the hydraulic pressure in the surface layer is much higher compared with unsalted concretes. Therefore, the protective pore volume has to be increased and the distance between these air-filled pores has to be decreased. Then the pressured pore water can flow into the protective pores and freeze there without causing damage in the concrete matrix. Similarly, in the salted concretes air-entrainment causes a higher compressive, three-dimensional stress state in concrete compared with non-air-entrained concretes. This hinders crack formation and even closes cracks that have been generated in concrete. The protective pores also alleviate the possible crystallization pressures of the salts.

In the salt-freezing situation, the air content measured from fresh concrete by the pressure method should now exceed about 5.5% when the maximum aggregate dimension is 32mm. The average air content should exceed 6.5%. Similarly, the water/cement ratio should be decreased so that it is below 0.45 in concrete structures exposed to salt freezing loads.

When concrete binder consists of a large amount of GGBS (> 60%) or condensed silica fume (~ 10%), carbonation of the surface layer coarsens the pore structure of these concretes. Due to the coarser pore size distribution, the freezable pore water volume is increased. At salt-freezing loads, the surface layer of these concretes can disintegrate very quickly, after 10–20 freeze-thaw cycles (Matala, 1995).

Estimated service lifespan assessment

In the 2004 Finnish concrete code (CAF, 2004), a calculation method for estimating service lifespan with regard to frost exposure and carbonation is presented. The expected service lifespan can be estimated by equation:

$$t_L = t_{Lr} \cdot A \cdot B \cdot C \cdot D \cdot E \cdot F \cdot G \qquad [1.3]$$

where t_L is the estimated service lifespan, t_{Lr} is the reference service lifespan (50 years), and, *A*–*G* are lifespan coefficients reflecting various factors.

Coefficients *A*–*G* in Table 1.1 are different for frost resistance and for carbonation. For example, in frost resistance, coefficient *A* takes into consideration air content, water/cement ratio, and maximum aggregate size. Coefficient *B* depends on massiveness (volume/surface area ratio) of the structure and possible coating of the structure. Coefficient *C* takes into consideration the curing measures. Coefficient *E* depends on the geographical direction and geographical location of the structure and coefficient *G* gives the impact of inspection and maintenance frequency. In carbonation, the effects of concrete strength class, cement type, and air content are taken into consideration in coefficient *A*. Coefficient *B* depends on concrete cover thickness over reinforcement and possible coatings on the concrete surface. Coefficient *C* depends on the curing measures and coefficient E_1 takes into consideration the effects of exposure class, coefficients E_2–E_4 take into consideration geographical direction, geographical location, and frost exposure. Coefficient *G* again depends on inspection and maintenance frequency.

As an example, the estimated service lifespans of concretes situated in environment class XF3 (frost action, horizontal structure, no salt exposure) can be assessed according to Table 1.2. Then only the material-related parameters are taken into consideration and all other coefficients have a value of 1. The estimated lifespan can be calculated by multiplying coefficient *A* by 50 years according to Equation [1.3]. The deterioration formulae by which the coefficient values have been calculated are also presented in the concrete code.

Table 1.1 Factors influencing frost resistance in Finnish concrete code (CAF, 2004)

Coefficient	Factor	Design parameters
A	Materials, porosity	Air content and water/cement ratio
B	Design, structural details	Structure type, coating
C	Performance of work	Curing time
D	Interior climate	–
E	Exterior exposure to weather	Frost exposure class, geographical direction
F	Working load	–
G	Maintenance measures	Inspection and maintenance frequency

Table 1.2 Values of coefficient A in exposure class XF3. Shaded values can be used only in studying the effects of existing structures in the case of poor quality (CAF, 2004)

Air content [%] Max. aggregate size [mm]			Coefficient *A* Effective water/cement ratio								
8	12	> 16	0,3	0,35	0,4	0,45	0,5	0,55	0,6	0,65	0,7
2,5	2,0	1,5	1,04	0,69	0,52	0,43	0,36	0,32	0,29	0,26	0,24
3,0	2,5	2,0	3,12	1,30	0,84	0,63	0,52	0,44	0,38	0,34	0,31
3,5	3,0	2,5	4,00	2,51	1,26	0,86	0,66	0,55	0,47	0,41	0,37
4,0	3,5	3,0	4,00	4,00	1,91	1,14	0,83	0,66	0,55	0,47	0,42
4,5	4,0	3,5	4,00	4,00	3,08	1,50	1,01	0,77	0,63	0,54	0,47
5,0	4,5	4,0	4,00	4,00	4,00	2,00	1,23	0,90	0,72	0,60	0,52
5,5	5,0	4,5	4,00	4,00	4,00	2,77	1,50	1,04	0,81	0,67	0,57
6,0	5,5	5,0	4,00	4,00	4,00	4,00	1,84	1,21	0,91	0,74	0,62
6,5	6,0	5,5	4,00	4,00	4,00	4,00	2,28	1,39	1,02	0,81	0,67
7,0	6,5	6,0	4,00	4,00	4,00	4,00	2,91	1,61	1,13	0,88	0,73
7,5	7,0	6,5	4,00	4,00	4,00	4,00	3,85	1,88	1,26	0,96	0,78
8,0	7,5	7,0	4,00	4,00	4,00	4,00	4,00	2,21	1,41	1,05	0,84
8,5	8,0	7,5	4,00	4,00	4,00	4,00	4,00	2,62	1,58	1,14	0,90
9,0	8,5	8,0	4,00	4,00	4,00	4,00	4,00	3,16	1,77	1,24	0,97
9,5	9,0	8,5	4,00	4,00	4,00	4,00	4,00	3,91	1,99	1,35	1,03
10,0	9,5	9,0	4,00	4,00	4,00	4,00	4,00	4,00	2,25	1,47	1,11

1.3.2 Non-uniform volume changes and temperature gradients

This section studies the effects of phenomena which cause cracking in concrete. Cracks on the concrete surface and around large aggregate particles can drastically increase the permeability of concrete. Cracks form a route for deleterious substances to penetrate into concrete, and this decreases the service lifespan of the structure. Cracks caused due to inadequate reinforcement against mechanical loads and cracking caused by fire loads are excluded from the discussion.

Cracks can be generated in concrete at early ages, during the first days or weeks, due to hydration or production technology-related reasons. At later ages, the non-uniform volume changes are mainly caused by chemically induced reasons such as reinforcement corrosion, carbonation, and alkali-aggregate reactions which will be dealt with in Section 1.4.

Concrete in the fresh state

Depending on the consistency of concrete, segregation of concrete constituents can take place. Heavier aggregate particles segregate to the bottom part of the mold or form and water rises up to the surface of the structure. Water bleeding can cause water pockets below larger aggregate particles and under reinforcement. When the water content in the pore structure has decreased due to hydration and drying of the structure, these water pockets leave air-filled cavities under aggregates and reinforcement. Permeability of concrete is increased. Similarly, the density of concrete increases towards the bottom of the structure and shrinkage cracks are generated at the weak upper surface of the structure. This plastic settlement of the concrete ingredients can cause large cracks over the reinforcement bars at the upper surface, because reinforcement does not settle with the concrete consolidation.

When the bleeding surface water has evaporated, if very fine supplementary cementitious binders such as condensed silica fume have been used, the tensional forces generated on the surface pore water can cause severe plastic cracking at the upper surface. The capillary tension in pore water can be calculated by Kelvin's equation Equation [1.1] by using the relative water vapor pressure of the ambient environment. Pore water tension compresses concrete at the surface layer in the beginning. If the evaporation rate exceeds the bleeding rate, cracks form intruding deep into concrete and generating severe plastic cracks. This can be avoided by using a stiff enough concrete consistency and by using proper curing measures so that these cracks do not impair the durability properties of the concrete surface.

Shrinkage

During hydration of the binders and subsequent drying of concrete, large volumetric changes occur in different phases of the hardening concrete. These time-dependent shrinkage deformations cannot take place freely, and they are restrained by aggregate particles, reinforcement, or structural constraints. Tensional stresses are generated into the structure, and they can exceed the tensional capacity of the binder paste, initiating shrinkage cracks. If the crack widths are large, the penetration rate of water and deleterious substances into concrete is increased.

The cementitious reactions during hydration cause chemical shrinkage in the binder paste. Chemical shrinkage of cement is about 25% of the volume of chemically bound water, which is about 7% of the volume of the constituents participating in hydration reactions. This represents a linear deformation of about 0.1% in normal-strength concrete.

The free water consumption during hydration and the volumetric decrease in the binder paste due to chemical shrinkage cause self-desiccation in the pore structure of concrete. The curved water menisci in the capillary pores generate tension in the pore water, which causes a compressive stress state in the binder matrix and aggregates. Pore water tension can be calculated by Kelvin's equation Equation [1.1], if the relative pore water vapor pressure is measured inside concrete. This autogeneous deformation caused by pore water tension is quite large for high-strength concrete in which the water/binder ratio is below 0.35. In these concretes, the relative water vapor pressure p_v/p_{v0} can be below 0.8. In high-strength concretes, autogeneous shrinkage cracks decrease the otherwise excellent durability properties. If the water/cement ratio is very low (0.17), autogeneous shrinkage can have a value of 0.7 %.

Evaporation of water from the surface of the concrete structure eventually dries the pore system. The volume changes are called drying shrinkage, and this is the dominating shrinkage mechanism in normal-strength concretes. The highly porous colloidal cement paste has very high surface area, and evaporation of pore water generates large shrinkage deformations. The final value of drying shrinkage can usually range from 0.3–0.6% in normal-strength concretes. During drying, the pore water tension differences between surface and interior concrete cause compression in the surface layer and tension inside the concrete.

The small amount of carbon dioxide in air reacts with calcium hydroxide in the pore water and eventually also other constituents of binder paste possessing CaO. This phenomenon is discussed in more detail in Section 1.4.1. Carbonation changes the pore structure of concrete and also generates carbonation shrinkage. Carbonation shrinkage can be as much as 30–50% of the total shrinkage of the surface layer of concrete. Part of the carbonation shrinkage is caused by the dissolution of $Ca(OH)_2$ crystals so that the compression caused in them by drying shrinkage is relieved. The forming calcium carbonate does not achieve any more similar compression.

Shrinkage takes place in the binder paste and, as aggregates do not shrink in a similar manner, differential shrinkage tensions generate cracks mainly in the interfacial transition zone (ITZ) between larger aggregates and bulk binder paste. The ITZ is more porous and weaker compared to bulk binder paste or aggregates and, therefore, cracks are generated into this zone. The ITZ is about 15–50 μm thick in normal-strength concretes.

Temperature gradients

During the hydration of massive concrete structures (dimensions over 0.7–1 m), hydration heat increases the temperature in the middle part of the structure when the concrete is 0.5–2 days old. At the same time, the strength and Young's modulus of concrete are increasing. If the ambient temperature of the environment is low, there can exist over 25 °C temperature differences at different parts of the structure. This generates compression at the middle part of the structure and tension at the surface which can cause cracking on the surface. Usually the crack widths are small (0.01–0.1 mm) and the crack depth into the surface layer of concrete is less than 50 mm.

However, the durability properties of the surface layers can be significantly compromised.

This crack formation can be hindered by selecting low-heat cements or by replacing a large portion of the cement by GGBS. Granulated blast furnace slag possesses much smaller hydration heat evolution. Also, replacing part of the cement by secondary cementitious binders such as fly ash can alleviate the tensional stresses at the surface. In very massive structures, heat insulation of the outer surface or even cooling of the inside of concrete by water pipe systems can be necessary.

During the cooling phase of the structure, concrete is shrinking, and this cooling shrinkage can be 0.2–0.4%. Comparing this with the other shrinkage deformations, it is of the same magnitude and can be an additional feature in shrinkage crack formation.

Heat treatment is a common way to increase the strength development rate especially in precast unit production. If the temperature rise or decrease rate in the process is large, this generates cracks into the structure. The coefficients of temperature expansion in water vapor, water, and binders and aggregates differ from each other by one and two orders of magnitude. The expansion differences in the different concrete phases are the reason for crack formation.

In older concrete structures, the temperature variations of the environment seldom cause cracking, if the structure is free to expand and the edge constraints do not hinder the deformations. However, in the design of certain bridge structures, for example, long-span box girders, temperature deformations have to be taken into consideration.

1.3.3 Mechanical abrasion, erosion, and cavitation

A common feature in this group of durability properties is that the deterioration mechanism is quite complicated and no simple tests that address all aspects of the phenomena are available. Mechanical abrasion of concrete surfaces can be caused in very different manners. The reason can be sliding of different materials on the surface, rolling of steel wheels, scraping motion of a machine, or percussion generated, for example, by studded tires of vehicles. In dams, off-shore structures, or industrial processes the action of abrasive materials carried by water, some other liquid, or ice leads to erosion of the concrete surface. Cavitation deterioration is caused by flowing water when the pressure in water changes abruptly. This damage mechanism has obtained its name because air cavities or bubbles collapse in flowing water generating a repeating, spike-like hitting pulse on the concrete surface.

Good-quality concrete surfaces that resist the durability loads belonging to this deterioration mechanism group have some common properties. Concrete compressive strength should exceed 35–40 MPa, and an increase in compressive strength usually improves the wear resistance. However, the cement content should not exceed 350 kg/m^3 because normal good-quality aggregates have much better abrasion and erosion resistance compared with binder paste. The concrete mix should be proportioned so that risk of segregation is as small as possible. Extended curing always has a beneficial effect on the durability properties of these concrete structures. High-strength concretes or high-performance concretes are especially suitable concrete types for this group of structures.

In abrasion and erosion resistant concretes, the properties of aggregates are of vital importance. Strong and hard aggregates are preferred, and the gap-graded sieve curve used in proportioning the aggregates causes a situation in the surface layer of concrete in which a greater amount of large aggregates is at the surface. As aggregates have better abrasion resistance compared with cement paste, this improves the abrasion and erosion resistance of the concrete surface. Steel fibers and polymer concretes can be advantageous in structures prone to cavitation.

1.4 Chemically induced deterioration

Carbonation, corrosion of steel reinforcement, effects of acids and sulfates, delayed ettringite formation, and alkali–aggregate reactions are the chemical reactions causing deterioration of concrete structures which will be discussed in this section. Carbonation of the surface layer of concrete does not damage concrete as such. It is important because it reduces the pH value in the pore water. This is a prerequisite for corrosion of steel reinforcement in situations when there are no chlorides in concrete pore water. Similarly, the ingress of chlorides into concrete can be caused by physical means, but chlorides substantially enhance the corrosion of steel reinforcement and, therefore, their effects will be considered together with reinforcement corrosion.

1.4.1 Carbonation

Air contains a small amount of carbon dioxide (0.03% in rural areas and 0.3% in large cities) and it dissolves into the pore water of concrete producing carbonic acid. Carbonic acid reacts readily with calcium hydroxide situated in pore water in dissolved and crystalline form. Reaction products are water and $CaCO_3$ or its polymorphs aragonite and vaterite. Calcium hydroxide is mainly responsible for the high alkalinity of pore water in concrete (pH-value 12.4–13.5) while $CaCO_3$ is nearly neutral (pH ~ 7). In this way, carbonation decreases alkalinity in pore water. Carbonation reaction needs a suitable amount of water to proceed. The highest rate of carbonation occurs at a relative humidity between 50 and 70% in the ambient air. When concrete is very dry or nearly saturated, carbonation rate is minimal.

The reaction product of carbonation, $CaCO_3$, is larger than $Ca(OH)_2$ and, thus, the pore volume in concrete decreases when calcium carbonate precipitates on the pore walls of concrete. This decreases permeability of concrete. However, cracking caused by carbonation shrinkage mentioned in Section 1.3.2 impairs the decreased permeability, and it can be assumed to remain nearly unchanged with respect to carbonation.

When most of the available $Ca(OH)_2$ in the pore water has been consumed, C–S–H gel will also begin to disintegrate. Eventually, all concrete constituents possessing CaO in their structure will carbonate after a long time. The compressive strength of carbonated, normal-strength concrete remains nearly unchanged because $CaCO_3$ and its polymorphs also carry loads in a manner similar to that of the virgin concrete.

Carbonation of reinforced concrete structures poses no durability problems if the structure is situated in a dry environment, for example, indoors. However, if concrete is wet, carbonation of the reinforcement cover concrete can shorten the expected service lifespan significantly.

Alkalinity of pore water forms a thin passivity layer of oxide on the reinforcement steel surface. This passivity layer completely protects the reaction of steel with oxygen and water, and reinforcement bars in non-carbonated concrete do not rust. When carbonation has decreased the alkalinity of pore water near the surface of the steel bars to a pH value of approximately 9, the protective passivity layer on the steel surface is broken and, if oxygen and water are present, rusting of steel reinforcement begins (Fig. 1.6).

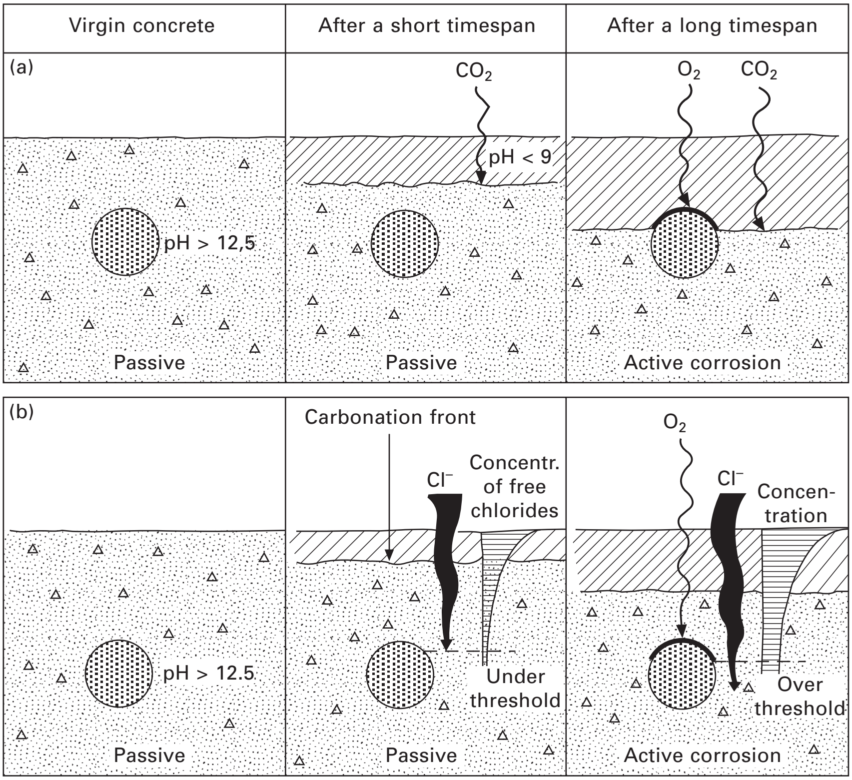
According to test results, the depth of the carbonation layer from the surface of the concrete structure increases in proportion to the square root of time if the environment of the structure is under steady hygrometric conditions:

$$x = k \cdot \sqrt{t} \tag{1.4}$$

where x is the carbonation depth [mm], k is the carbonation coefficient [mm/year$^{0.5}$], and t is the exposure time [years]. This equation becomes effective after a couple of years carbonation.

The carbonation coefficient depends on the permeability of the surface layer of concrete and the relative humidity of the environment. When concrete is produced by using a low water/cement ratio, the carbonation rate decreases compared with low-strength concretes. If the surface is exposed to occasional rain so that the pore structure of concrete is nearly saturated part of the time, the carbonation rate is decreased compared with steady hygrometric conditions.

The form of the carbonation depth equation is such that the rate of the penetration depth of the carbonated layer decreases with increasing time. An increase in the thickness of the reinforcement cover layer effectively increases the estimated service lifespan of the reinforced concrete structure by increasing the reinforcement corrosion initiation time.



1.6 Corrosion initiation of concrete reinforcement caused by carbonation or chlorides when concrete is wet: (a) carbonation-initiated corrosion and (b) chloride-initiated corrosion (Fagerlund, 1987).

Four features which affect the carbonation rate have been examined above: carbon dioxide volume in the ambient air, permeability of the cover concrete over reinforcement, pore filling degree (moisture content) of the cover concrete, and the thickness of the cover concrete. The amount of CaO in concrete which can carbonate is obviously an important variable. The higher this CaO content is, the slower the carbonation rate. There is a big difference in CaO content which can carbonate in concretes produced by different blended cements. If the secondary cementitious binders are pozzolans such as fly ash or silica fume, they react with $Ca(OH)_2$ and form C–S–H, and the calcium hydroxide content in the concrete decreases. Therefore, concretes produced by blended cements are prone to carbonate faster than concretes produced by Portland cements. Similarly, if the binder contains a large amount of GGBS, there is a much smaller amount of CaO available for carbonation, and the carbonation rate is higher.

However, the permeability of these concretes produced by blended cements can be substantially reduced by increasing the hydration degree of the surface layer with prolonged and adequate wet curing. If the amount of these secondary cementitious binders is relatively small (fly ash < 30% or GGBS < 50% of the total binder amount) and adequate curing is applied, carbonation rate does not differ much from those concretes produced by pure Portland cements.

Sulfate-resisting cement leads to about 50% greater depth of carbonation compared to the situation when other Portland cements are used. Therefore, increased cover to reinforcement may be required.

1.4.2 Corrosion of steel reinforcement

Corrosion of steel reinforcement inside concrete can be initiated by two different mechanisms (Fig. 1.6). If there are no chlorides in the pore water, corrosion can be initiated when the carbonation front in the concrete cover over reinforcement has reached the steel bars. When the alkalinity in the pore water near reinforcement has decreased to the value of pH 9, the protective passivity oxide layer of $\gamma\ Fe_2O_3$ on the surface of the steel bars is broken and corrosion begins if water and oxygen are available on the surface of the reinforcement. Rusting of the reinforcement is a chemical reaction and, therefore, also a temperature-related phenomenon. In low sub-zero temperatures, the corrosion rate is considerably slower compared with the situation when the temperature is high (+20–40 °C).

The passivity protective oxide layer can also be broken, if the chloride content in pore water in the vicinity of the steel bars exceeds a certain threshold value which is dependent on the OH^--ion concentration of the pore water. Also, in chloride-initiated corrosion, water and oxygen must be available near the reinforcement surface.

After the initiation period, corrosion is an electrochemical process in which electrons and OH^--ions are transported between anode and cathode parts of the reinforcement and an electric circuit is formed. At the anode, positive metal ions Fe^{2+} are dissolved into the pore water and electrons move to the cathode via reinforcement. At the cathode, a chemical reaction takes place between electrons, oxygen, and water to form hydroxyl ions which move to the anode through pore water. At the anode, hydroxyl ions react with iron ions and $Fe(OH)_2$ or rust forms. There has to exist a difference in electrical potential between the anode and cathode as a driving force to sustain the reaction. Corrosion takes place only at the anode and, if the reaction can proceed freely, ferric hydroxide $Fe(OH)_3$ will form as the end product. The volume of the corrosion products can increase by a factor of over five which causes tensile stresses around the reinforcement bar. Eventually, in low-strength concretes, this can cause cracking, spalling, or even delamination of the concrete cover over the reinforcement.

Cracks in the concrete cover have only a small influence on the service lifespan of the structure if there are no chlorides present and if the cracks are generated in a direction perpendicular to the reinforcement bars. This holds even if the crack width is relatively large. Corrosion products and re-alkalization in the crack over the reinforcement effectively hinder the advancement of corrosion. Concrete produced by a low water/cement ratio is, of course, more advantageous in this respect compared with concretes in which the water/cement ratio is high. If occasional mechanical loads are so severe that there is change in the crack width or if flowing water rinses the cracked surface, cracks will decrease the service lifespan of the structure.

When there is a chloride concentration exceeding a threshold value in the pore water, chloride ions break the protective oxide layer over the steel to form an anode, while the unbroken surface forms the cathode. During the chemical reaction, ferrous chloride is formed at the intermediate stage of the reaction, but, as ferrous hydroxide contains no chloride, Cl^- is regenerated by formation of HCl. In reality, the process of chloride-induced reinforcement corrosion is more complicated because drying of concrete and changes in moisture content affect the corrosion rate in a complicated manner (Raupach, 1996, 2006).

Only free chlorides in the pore water are effective in initiating the chloride-induced reinforcement corrosion. A part of the chlorides has reacted with the aluminates in the binder paste and is bound into the binder matrix. Hausmann (1967) has derived the relation between chloride content and OH^--concentration by studying reinforcement corrosion of steel in chloride/water solutions. Corrosion is initiated only if the concentration relation between chloride ions and hydroxyl ions is fulfilled, Equation [1.5]:

$$\frac{C_{\mathrm{Cl^-}}\ (\mathrm{mol/l})}{C_{\mathrm{OH^-}}\ (\mathrm{equiv./l})} \geq 0.6 \qquad [1.5]$$

OH^--concentration can be calculated by a formula derived by Tuutti (1982):

$$C_{\mathrm{OH^-}} = \frac{c}{p} \cdot \left(\frac{\mathrm{Na}}{23} + \frac{\mathrm{K}}{39}\right) \cdot 100 \qquad [1.6]$$

in which c is cement content in [kg/m^3 concrete] and Na and K are wt% of soluble sodium and potassium, respectively, in the binder. Pore volume percentage in concrete is represented by p. Not only does this threshold value initiate reinforcement corrosion, but the moisture content in the concrete must be sufficiently high. Moreover, the permeability of concrete cover with respect to oxygen penetration has an effect on the beginning of the corrosion.

In chloride-induced reinforcement corrosion, cracking of the concrete cover layer substantially decreases the service lifespan of the concrete structure because then re-alkalization in the crack does not take place.

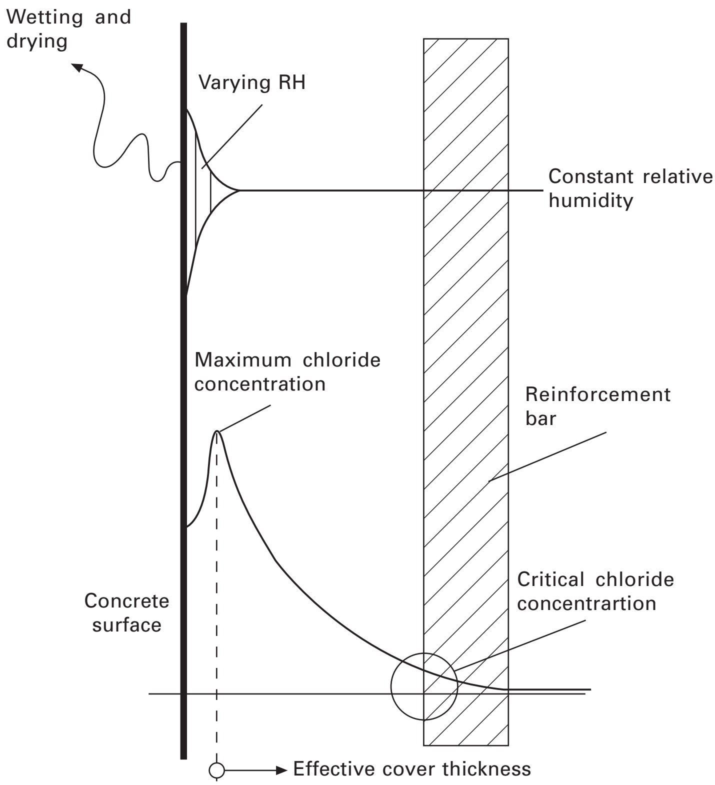
Penetration of chlorides into concrete

Reinforcement corrosion is one of the major deterioration mechanisms of reinforced concrete structures worldwide. The presence of chlorides increases the severity of the corrosion attack considerably. Chlorides can penetrate into concrete which is in contact with de-icing salts or seawater. Typical structures that are damaged by chloride-initiated reinforcement corrosion include bridges, car parking structures, and off-shore structures such as piers, dams, docks, and harbor structures.

Critical amounts of chlorides may also be present in the fresh concrete mix even though at present very few admixtures used in concrete contain chlorides. If seawater is used in producing concrete, chlorides are introduced into the mix.

When chlorides penetrate into concrete from outside, they are usually in a water solution. In moist concrete, the main transport mechanism is diffusion, but the capillary transport mechanism is also possible if concrete is exposed to drying and wetting cycles. Cyclic freeze–thaw loads can effectively increase the chloride content in concrete pore water. It is common practice to measure the maximum concentration of the chloride front penetrating from the surface of the concrete cover over reinforcement some centimeters inside the exposed surface (Fig. 1.7). Even though drying of concrete complicates the theoretical modeling of the phenomenon, Fick's second law is commonly applied in mathematical modeling of chloride intrusion into concrete.

The three most important variables that govern the chloride intrusion into concrete and the corrosion of the reinforcement are concentration of chlorides at the surface, concentration threshold value which initiates corrosion of steel, and the transport rate of chloride ions in the concrete cover layer. Without coating the surface of concrete, there are usually very limited means to decrease the concentration at the surface. If this chloride concentration is high, it is nearly impossible to hinder the penetration of chlorides to the reinforcement during long exposure times (50–100 years). At normal chloride exposure concentrations (seawater or de-icing agents), by selecting binders that cause a high OH^--concentration into pore water, the chloride threshold value that initiates corrosion of steel can be increased. Similarly, some binders react with chlorides and this decreases the free chloride concentration in pore water solution. These binders contain large amounts of C_3A or GGBS. This is only a temporary relief, because during carbonation large volumes of these bound chlorides dissolve back into pore water.



1.7 Chloride distributions in the surface layer of concrete (adapted from Sandberg, 1993).

The transport rate of chloride ions can be decreased by producing a more impermeable concrete cover by using lower water/cement ratio and by applying longer wet curing. The rate of chloride-induced corrosion is reduced considerably in structures situated in environments where relative humidity is less than 80%.

After initiation of chloride-induced reinforcement corrosion, it usually takes less than 10 years for the concrete cover surface to deteriorate to such an extent that repair measures have to be applied.

1.4.3 Reactions in binder paste and aggregates

The chemically induced deterioration mechanisms to be examined in this section are sulfate attack, acid attack, and alkali–aggregate deterioration reactions.

Sulfate attack

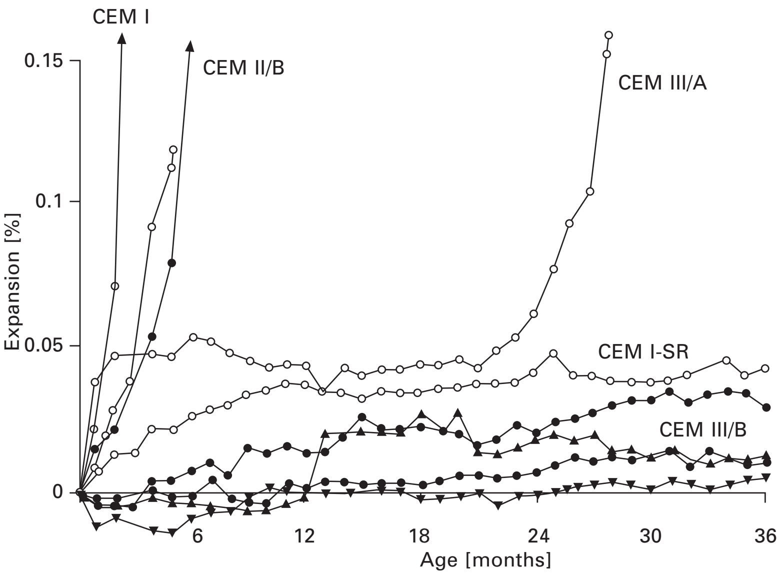
Sulfate attack is initiated when water-soluble sulfates (SO_4^{2-}), originating from ground or from seawater, penetrate into concrete pore water and react with aluminates or calcium hydroxide in cement paste. Reaction products expand remarkably which causes crack propagation and decreases the strength properties of concrete.

Four reaction mechanisms are responsible for sulfate damage in concrete. Sulfate ions can react with calcium hydroxide forming gypsum ($CaSO_4 \cdot H_2O$). Aluminates from cement or sometimes from aggregates can react with sulfates forming trisulfate (ettringite $3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 31H_2O$). The increase in volume of the solid phases in these reactions is 124 and 227%, respectively.

The third sulfate deterioration mechanism is attributed to sulfate absorption into silicates or to a reaction with C–S–H. In these instances thaumasite ($CaSiO_3 \cdot CaCO_3 \cdot CaSO_4 \cdot 15H_2O$) is produced. This reaction takes place at low temperatures (Schneider *et al.*, 2003; Mielich and Öttl, 2004). The fourth mechanism does not need an outside source of sulfates to cause expansion and cracking into concrete. The deterioration mechanism can be termed inner sulfate attack caused by excessive heat treatment in concretes produced by Portland cement. When the temperature rises to 70–100 °C during hydration, ettringite transforms into monosulfate ($3CaO \cdot Al_2O_3 \cdot CaSO_4 \cdot 12H_2O$) and sulfate. At lower temperatures, monosulfate becomes metastable and, if there is sufficient water available in hardened concrete or if the water content in the concrete subsequently increases, ettringite can again be formed. This reaction is accompanied by expansion in the concrete structure and subsequent cracking. This reaction can happen after a period of a couple years and, therefore, it is sometimes called delayed ettringite formation (Heinz, 1989; Stark *et al.*, 1992). This deterioration mechanism has been observed in façade precast units, concrete railway sleepers, and basement slabs.

The severity of sulfate corrosion expansion caused by outside sulfate attack is different depending on the salt composition. The severity increases in the order calcium sulfate, sodium sulfate, and magnesium sulfate. The severity of the attack increases also when the moisture content in concrete increases.

Sulfate attack can be mitigated by minimizing the C_3A content of the cement by applying sulfate resisting cements (Fig. 1.8). Sulfate resisting cements have a C_3A content below 3% or blast furnace slag content in the binder exceeds 70%. The other mitigation method is to reduce the $Ca(OH)_2$ content in concrete by applying blended cements in which the pozzolanic reaction decreases the calcium hydroxide amount.



1.8 Sulfate expansions of test mortars produced by different binders. The water : cement ratio of the mortars is 0.6, mortars have been immersed in sodium sulfate solution in which SO_4^{-2} content is 30 g/l (Frearson, 1986). The author has introduced contemporary cement-type notations into the figure.

Acid attack

Due to the high alkalinity of pore water (pH 12.5–14) in concrete, all binder paste constituents are stable in this environment. All strong acids (pH < 4.5) and many weak acids (5.5 < pH < 6.5) effectively decrease pore water alkalinity and attack $Ca(OH)_2$ and C–S–H gel of the binder paste. Most aggregates endure acid attack much better compared with the binder paste.

Acid attack is a surface phenomenon similar to carbonation and, therefore, the penetration depth is a function of the square root of time. The decomposition of hydration products forms new compounds which, if they are soluble, may be leached out from the structure. Some compounds can be disruptive as such, for example, H_2SO_4 is a combination of acid attack and sulfate attack.

In the case of weak acids, the severity of the attack cannot be evaluated by pH value only. Then one has to rely on a corrosion list of substances which attack concrete to varying degrees (Biczok, 1972; ACI 515.1R, 1985). No standard test procedures are available but, if comparative tests are performed, they should be carried out with the same concentration and composition of the acids that occur in the environment of the planned structure. Similarly, the flow rate of the solution and temperature should be comparable with the real planned structure. Accelerated tests are not recommended.

Pollution-induced acid rain consists mainly of nitric acid and sulfuric acid and is usually so dilute that it does not cause deterioration of concrete surface. However, if the pH value of acid rain is below 4.5, it can cause weathering of exposed concrete surfaces.

If sewage pipelines are not ventilated properly, bacterial growth can further a situation in which sulfuric acid is formed. The deteriorating portion of the concrete pipe structure occurs above the level of flow of the sewage.

Flowing pure water or ground water containing a small amount of CO_2 dissolves calcium hydroxide and causes surface erosion of concrete. Aggressivity of this water increases with the decreasing hardness (Ca^{2+}content/l) of the water (Gérard *et al*., 2002).

Alkali–aggregate reactions

There are two reaction types causing deleterious swelling of concrete in moist environment due to reactions between alkalis (Na_2O and K_2O) and certain aggregates. In alkali–silica reaction, the reactive forms of silica are opal, chalcedony, and tridymite, which occur in opaline or chalcedonic cherts, siliceous limestones, and some volcanic rocks as rhyolites. Alkali–carbonate reaction is caused between some dolomitic limestone aggregates and the alkalis of the cement (Neville, 1995).

In both deleterious aggregate reactions, not all aspects of the mechanisms involved are known. Reactive siliceous minerals in the aggregate react with alkaline hydroxides originating usually from cement. Alkali–silicate gel is formed in the voids and cracks of the aggregate or on the surface of the aggregate. The gel absorbs water and swells in large volume (5–20%) if water is available in concrete and the environment. Internal pressures are generated into the concrete and eventually cracking can destroy the concrete structure totally. On the surface, cracks form a map-like pattern and sometimes pop-outs can be observed.

A combination of mix design features and moisture condition have to be fulfilled for the deleterious swelling of the gel to occur. The severity of the swelling of the gel depends on the amount of reactive material and on its particle size, alkali content in the pore water, and the moisture content in the concrete. For different reactive aggregates, particle size fraction, and cement type, a different pessimum combination can be found. If the reactive aggregate material amount in concrete is very small or very large and the moisture content is below a threshold value, the expansion caused by the swelling gel can be insignificant.

To hinder the alkali–silica reaction, the maximum relative humidity in the interior of concrete should not exceed 80–85%. The cement type should have as low a content of alkali oxide as possible. The equivalent Na_2O content in the cement should not exceed 0.60 (eqv. $Na_2O = Na_2O + 0.659 \cdot K_2O$ by weight). Also, application of pozzolanic secondary cementitious binders has been shown to diminish the deleterious expansion caused by the alkali–silicate reaction.

The imperfectly understood damage mechanism in the alkali–carbonate aggregate reaction involves the de-dolomitization of the dolomite structure. When dolomite $CaMg(CO_3)_2$ structure is changed into $CaCO_3$ and $Mg(OH)_2$, it becomes more open and other minerals such as clay in the dolomite aggregate begin to expand due to moisture. Pozzolanic secondary binders are not effective in controlling the alkali–carbonate expansion which is contrary to alkali–silicate reaction. Fortunately, alkali–carbonate reaction is quite rare.

1.5 Summary

This chapter of the book deals with deterioration mechanisms in concrete and reinforced concrete. It can give the impression that a large number of concrete structures are subject to deterioration. However, most indoor concrete structures usually have no corrosion problems and their service lifespan can be measured in hundreds of years.

Outdoor concrete structures constitute about one-third of the total volume of concrete structures, and the major deterioration mechanisms that affect them are reinforcement corrosion and freeze–thaw deterioration. A large majority of the outdoor structures are not exposed to salts, and it is not difficult to design and build concrete structures that possess an estimated service lifespan of 200 years. The technology and design knowledge already exist to extend the estimated lifespan of outdoor structures exposed to chlorides to 100 years. All other deterioration mechanisms apply to concrete structures that comprise below 5% of the total concrete volume. Even in these structures, durability properties of concrete usually exceed those of the competing materials.

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2
Types of damage in concrete structures

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Abstract: Different types and sources of damage in concrete structures are dealt with and the classification of damage into different categories is discussed. The simple and convenient classification of damage by sources, which includes chemical attack, fire, overloading by static and dynamic (impact and earthquakes) loads, and, finally, malicious damage, is suggested for the practicing engineer. Examples of different kinds of damage are given and discussed. Finally, two case studies, which help the practicing engineer in making the damage investigation and developing recommendations for rehabilitation of structures and prevention of damage in future, are reported.

Key words: reinforced concrete, damage, chemical attack, fire, overloading, impact, earthquakes, malicious damage.

2.1 Introduction

When one starts investigating the damage or failure of a given concrete structure, the reasons for such damage are not always clear. The documentation on the structure and the history of its manufacture and maintenance are not always available. At the same time, the visual inspection and appearance of the structure can provide much valuable information about the type of loading that resulted in the damage and help to understand what happened.

Within the same type, each loading situation is unique. Like a detective trying to gather an overall picture of the crime by analyzing fingerprints of persons involved and other evidence, the experienced engineer can recreate and tell the story of the given damage/failure with a certain degree of probability.

The present chapter deals with different types and sources of damage in concrete structures. The classification of damage into different categories is discussed. The simple and convenient classification of damage by sources, which includes chemical attack, fire, overloading by static and dynamic (impact and earthquakes) loads and, finally, malicious damage, is suggested for the practicing engineer. Examples of different kinds of damage are given and discussed. Finally, two case studies, which help the practicing engineer in carrying out the damage investigation and developing recommendations for rehabilitation of structures and preventing damage in future, are reported.

2.2 Types of concrete damage

Damage to concrete structures can be categorized in different ways. The classification can be made in terms of damage types, causes, mechanisms of attack, frequency of defects, kinds of deficient structures, financial loss due to different defects, amount and extent of repair measures, etc. Let us consider first the type of concrete damage, and then its causes and mechanisms.

Concrete damage can be of the following main types:

1. scaling;
2. spalling;
3. curling;
4. cracking.

This classification is based on the simple visual appearance of concrete defects. At the same time, different types of damage described shortly hereafter correspond to causes or mechanisms of concrete damage, and can therefore can be considered as their ‘fingerprints.’ For example, scaling and spalling of concrete are characteristic of freezing–thawing cycles and fires, respectively.

Scaling is one of the leading complaints of many homeowners and probably the easiest defect of a concrete surface to avoid. When concrete scales, for example, as a result of freezing and thawing cycles, the finished surface flakes or peels off. Generally it starts as localized small patches, which later may merge and extend to expose large areas. Light scaling does not expose the coarse aggregate. Moderate scaling exposes the aggregate and may involve loss of up to 2.5–10mm of the surface mortar. In severe scaling, more surface material is lost and the aggregate is clearly exposed and stands out. Scaling is primarily found in outside concrete flat work such as sidewalks, patios, and driveways. In most cases it is blamed on de-icing salts used on the concrete during the winter months. Despite these assertions, the truth is that properly specified, placed, and cured concrete should be able to endure the effects of typical de-icing agents.

Spalling can be described as the breaking of layers or pieces of concrete from the surface of a structural element. Spalling occurs often when concrete is exposed to the high and rapidly rising temperatures experienced in fires; however, it can be also a result of other mechanisms, such as steel corrosion or improper design/technology of pretensioning steel for prestressed concrete members.

There are four main types of concrete spalling:

- **surface spalling** – affects aggregate on the concrete surface, whereby concrete fragments typically up to 20mm in diameter become detached;
- **corner break-off or sloughing off** – this tends to occur in the later

stages of a fire and affects more vulnerable concrete on wall corners where it is heated on two planes;

- **explosive spalling** – early rapid heat rise forcibly separates pieces of concrete at high pressure, with an 'explosive' effect; this form of spalling is considered as the most dangerous one;
- **corrosion spalling** – when reinforcing bars rust, their volume increases and steel cover becomes detached.

Curling is the upward or downward bending of the edges of a concrete element (usually slab or beam), giving the concrete member a cupped shape. The curled edges are unsupported by the base, making them susceptible to cracking under heavy loads.

Cracking is a path of the (local) separation of a structural element or material into two, or more, pieces under the action of stress. Cracking, like corrosion of reinforcing steel, is not commonly a cause of damage to concrete. Instead, cracking is a symptom of damage created by some other cause. For example, cracking can be the result of one or a combination of factors, such as drying shrinkage, thermal contraction, restraint (external or internal) to shortening, subgrade settlement, and applied loads. While cracks may develop in concrete for a variety of causes, the underlying principle is the relatively low tensile strength of concrete.

Cracking can be considered as a separate type of damage, but it also accompanies the rest of the damage types listed before: scaling, spalling, and curling. A comprehensive state-of-the-art report[1] reviews the causes of cracking, discusses various tests that can be performed to assess the susceptibility of a material to cracking, and provides several case studies. In particular, the following classification of cracks in concrete is suggested (Table 2.1).

Cracks in concrete are discussed in detail in the next chapter of the book.

Another classification of damage types of concrete is available in the work by Al-Mandil *et al.*,[2] which is based on detailed *in-situ* and laboratory investigations of girder-slab and slab-type decks. This paper distinguished between two main types of damage of concrete:

1. structural damage, such as that resulting from overloading of the bridges; and
2. material damage, such as that resulting from lack of quality control and poor construction materials in an environment conductive to corrosion.

Cracking of concrete due to stresses induced by thermal gradients, especially in thick slab-type decks supported on minimally functional bearings, which do not allow free movement at the ends, was listed as an additional causal factor contributing to damage in the arid environment. This classification

Table 2.1 Classification of cracking types[1]

Type of cracking	Form of crack	Primary cause	Time of appearance
Plastic settlement	Over and aligned with reinforcement, subsidence under reinforcing bars	Poor mixture design leading to excessive bleeding, excessive vibrations	10 min to 3 h
Plastic shrinkage	Diagonal or random	Excessive early evaporation	30 min to 6 h
Thermal expansion and contraction	Transverse	Excessive heat generation, excessive temperature gradients	1 day to 2–3 weeks
Drying shrinkage	Transverse, pattern or map cracking	Excessive mixture water, inefficient joints, large joint spacings	Weeks to months
Freezing and thawing	Parallel to the surface of concrete	Lack of proper air void system, non-durable coarse aggregate	After one or more winters
Corrosion of reinforcement	Over reinforcement	Inadequate cover, ingress of sufficient chloride	More than 2 years
Alkali–aggregate reaction material	Pattern and longitudinal cracks parallel to the least restrained side	Reactive aggregate plus alkali hydroxides plus moisture	Typically more than 5 years, but weeks with a highly reactive material
Sulfate attack	Pattern	Internal or external sulfates promoting the formation of ettringite	1–5 years

is based on the scale of the analysis, in which either structural or material engineering approaches are used. However, it is still uncertain how to distinguish accurately between structural and materials aspects of the analysis, and therefore this classification is not strict enough.

2.3 Causes and mechanisms of concrete damage

Another possible way to classify concrete damage would be to divide it by following four main factors causing the damage: physical, chemical, biological, and mechanical factors.

The physical factors include heat, changes in temperature, moisture, wind, etc. The physical causes of concrete damage can be grouped into two main categories:[3] (i) surface wear or loss of mass due to abrasion, erosion, and cavitation; (ii) cracking due to normal temperature and humidity gradients, crystallization of salts in pores, structural loading, and exposure to temperature extremes such as freezing or fire.

The chemical factors include acids, leaching of salts, organic substances, etc. The chemical causes of concrete damage can be grouped into three main categories:[3] (i) hydrolysis of the cement paste components by soft water; (ii) cation-exchange reactions between aggressive fluids and the cement paste; and (iii) reactions leading to formation of expansive products, such as in the case of sulfate attack, alkali–aggregate reaction, and corrosion of reinforcing steel in concrete.

The distinction between the physical and chemical causes of deterioration is purely arbitrary; in practice, the two are frequently superimposed on each other.[4] For example, loss of mass by surface wear and cracking increases the permeability of concrete, which then becomes the primary cause of one or more processes of chemical deterioration. Similarly, the detrimental effects of the chemical phenomena are physical; for instance, leaching of the components of hardened cement paste by soft water or acidic fluids would increase the porosity of concrete, thus making the material more vulnerable to abrasion and erosion.

The biological factors include micro-organisms, fungi, algae, moss, etc. Finally, the mechanical factors include overloading (both by static and dynamic loads), construction faults, etc.

A comprehensive guide to concrete repair[5] prepared for the Bureau of Reclamation of the United States Department of the Interior addresses the following common causes of damage to concrete:

1. Excess of concrete mix water
2. Faulty design
3. Construction defects
4. Sulfate deterioration
5. Alkali–aggregate reaction
6. Deterioration caused by cyclic freezing and thawing
7. Abrasion–erosion damage
8. Cavitation damage
9. Corrosion of reinforcing steel
10. Acid exposure
11. Cracking
12. Structural overloads
13. Multiple causes

The excess of concrete mix water is considered by this guide as ‘the single most common cause of damage to concrete'. Excessive water increases porosity, reduces strength, increases shrinkage (except autogenous shrinkage of low water to cement concrete, which is controlled by completely different mechanisms, such as chemical shrinkage and self-desiccation, both are out of the scope of this chapter), increases creep and reduces the abrasion resistance of concrete. The guide[5] notes that damage due to excessive mix water is sometimes difficult to diagnose, because it is masked by damage from other causes listed above, such as freezing–thawing cycles, abrasion, or drying shrinkage cracking. We agree completely with this statement, but this merely proves that it is difficult to develop an ideally accurate classification of the damage causes, in which the causes are completely independent.

A similar case is faulty design, which can create many different types of concrete damage. For example, insufficient concrete cover is often responsible for accelerated reinforcement corrosion. In other words, the reinforcement corrosion can be a consequence of the faulty design, and the decision on the primary cause of the damage depends very much on the specific project.

The present chapter uses another classification of concrete damage based on the causes and mechanisms, which are observed the most frequently by the eyes of the practicing civil engineer, and we will follow this classification in further sections:

1. Chemical attack
2. Fire
3. Static overloading
4. Impact
5. Earthquakes
6. Malicious damage

It has to be emphasized that in real life it is often difficult to distinguish clearly between different reasons and mechanisms of damage. We also realize that in reality the reasons and mechanisms listed above can overlap, or may be primary and secondary. For example, a mistake on the part of a crane operator can result in an extreme impact load applied to the structure. Depending on the type of structure, the damage caused by human mistakes or wrecking can be similar to that observed in earthquake or impact loading.

The classification introduced here cannot be considered as an ideal one, but in our opinion it serves well for grouping the types of damage by their frequency and practical importance in everyday life.

2.4 Chemical attack

Damage due to chemical attack can be classified by following main categories:

1. damage caused by acidic reactions;
2. damage caused by aggregate reactions – for example, by alkali–silica reaction (ASR);
3. damage due to chloride attack.

We will consider here only the first type of damage (caused by acidic reactions), because the second and third types are described in detail in other chapters of this book.

Acids react with calcium hydroxide, which is formed in the course of cement hydration. The products of acidic reactions are highly soluble and can be easily removed by leaching.

Acidic reactions occur in the cement paste matrix, but they affect the aggregate integrity, the process leading to the damage of concrete. Depending on the type of coarse aggregate, two different mechanisms of aggregate damage can be distinguished:

- **Granite aggregate** – gradual dissolution of the cement paste matrix results in disintegration of the aggregate particles.
- **Dolomite or limestone aggregate** – the acidic reaction in the cement paste matrix uncovers the aggregate surface, dissolves the aggregate material – the process significantly increases the contact of the acid with concrete, and therefore the rate of the dissolution of concrete in structures increases by many times. This is illustrated further in the case studies. In cases when concentrated acids leak, the process of dissolution occurs very rapidly – acids simply dissolve structures. Figure 2.1 illustrates the dissolution of the concrete structure of the port chemical terminal as a result of the acidic leakage.

2.5 Fire

Fires can be caused by accident, energy sources, or natural means, but the majority of fires in buildings are caused by human factors (e.g. discarded cigarettes). Once a fire starts and the contents and/or materials in a building are burning, then the fire spreads via radiation, convection, or conduction with flames reaching temperatures of between 600 and 1200 °C.

During the fire concrete undergoes severe microstructural changes, which are caused by complex physical and chemical mechanisms (Table 2.2); these lead to irreversible structural damage. The effect of fire on concrete and concrete structures is described in more detail in the work by Khoury.[6] Rehydration of the calcium oxide on cooling of the structure causes expansion, which can cause damage to material which withstood fire without falling apart. Concrete in buildings that experienced a fire and were left standing for several years shows an extensive degree of carbonation.

The main harm in fire is caused by a combination of the effects of smoke

2.1 Dissolution of the concrete structure of a chemical wharf in the sea port as a result of the acidic leakage.

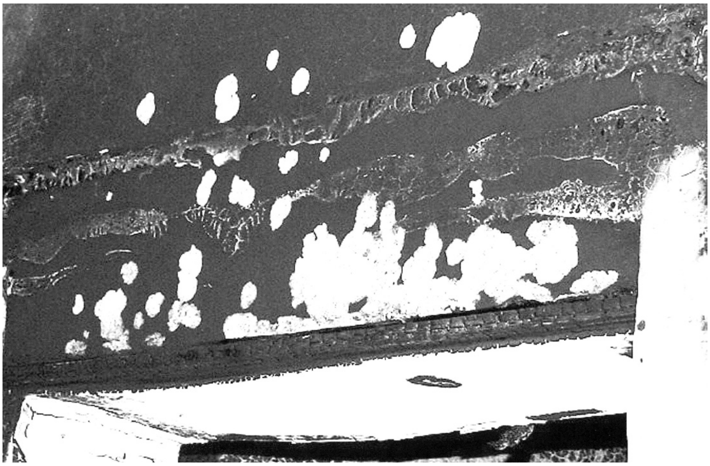
and gases, which are emitted from burning materials, and the effects of flames and high air temperatures. Fortunately, concrete does not burn and does not emit any toxic fumes when affected by fire. It will also not produce smoke or drip molten particles, unlike some plastics and metals, so it does not add to the fire load. For these reasons concrete is considered to be a fire resistant material. This excellent performance is mainly due to a relatively poor thermal conductivity, which is crucially important for fire safety design. It is this slow rate of heat transfer (conductivity) that enables concrete to act as an effective fire shield not only between adjacent spaces, but also to protect itself from fire damage. The only potential risk to life from concrete in fire occurs in the form of concrete spalling (Fig. 2.2), which principally affects high-strength concrete. However, even here, effective measures can be taken to reduce the probability of spalling.

The most serious form of damage to concrete under fire is explosive spalling, which occurs usually during the first 30 minutes after fire starts. The conventional explanation of explosive spalling is that it is caused by

Table 2.2 Changes in concrete during fire

Temperature	What happens with concrete
1200 °C	Concrete melting starts.
900 °C	Air temperatures in fires rarely exceed this level, but flame temperatures can rise to 1200 °C and beyond.
800 °C	Total loss of chemically bound water from the hydrated products in cement paste matrix. Ceramic binding starts.
700 °C	Calcium carbonate decomposes at about 700 °C–800 °C.
600 °C	At 573 °C, quartz undergoes rapid expansion due to phase change from α to β form. Above 600 °C concrete is not functioning at its full structural capacity.
500 °C	Up to about 500 °C, the major structural changes are carbonation and coarsening of pores. At this temperature significant loss of strength occurs and the elasticity modulus of concrete is significantly reduced. Above this temperature cementitious materials experience considerable creep and lose their capacity to bear loads. Hot rolled steel is likely to recover its full yield strength even when heated to 600 °C, but beyond this some strength losses will occur.
400 °C	Prestressing will be down to 50% of its strength above 400 °C and cold worked steel will be affected above 450 °C. At 450 °C calcium hydroxide decomposes, yielding calcium oxide. Concrete is weakened and some loss of modulus of elasticity occurs.
300 °C	Up to about 300 °C, the concrete undergoes normal thermal expansion, and no appreciable damage occurs. Pink, white or buff color indicates that the temperature was at least 300 °C. Above this temperature shrinkage occurs due to dehydration of cement paste constituents. At the same time, the aggregate continues expanding, which causes internal stresses. Strength loss starts, but in reality only the first few centimeters of concrete exposed to a fire will get any hotter than this temperature and internal temperature is well below this.
200 °C	Some spalling may take place, with pieces of concrete breaking away from the surface. Some kinds of aggregate (e.g. flint) undergo dehydration. The aggregate particles start to expand beyond 100 °C, generating differential strains. However, extensive microcracking of the cement paste, which can result in disintegration of the concrete, is not usually observed. The process of ‘transient creep’ (or load-induced thermal strain), occurring when concrete is first heated, is generally credited with relieving the differential thermal strains between the paste and aggregate, and preventing disintegration.
100 °C	Hydrothermal reactions and loss of chemically bound water start.

the build-up of water vapor pressure in concrete during fire and thermal stresses. High-strength concrete is not permeable, therefore water vapor formed within it during heating will not be able to dissipate and pressure is formed. When that pressure exceeds the tensile strength of the concrete,



2.2 Spalling above a doorway.

explosive spalling will occur. As far as normal strength concrete is concerned, spalling is seldom encountered, and it is generally not a safety risk.

It is generally accepted that it is the rapid heating of concrete under fire which is a more significant cause of spalling than the exposure of concrete to high temperatures over time. Similarly, there is consensus that concrete made with limestone, lightweight and/or air-dried aggregate is less susceptible to spalling than that made with siliceous aggregate.

There are four alternative ways of lowering the spalling risk:[7]

1. The addition of polymer (for example, polypropylene) fibers into the concrete mix. This approach works on the basis that, as the concrete is heated by fire, the polymer fibers melt, creating passageways along which water vapor dissipates, so avoiding a build-up of pressure.
2. The spray coating of finished concrete with a substance that slows down the rate of heat transfer from fire. It is the rate of temperature change in the concrete that has been proven to be at least as important a cause of spalling as the on-going exposure to high temperature itself.
3. Placing a preformed thermal barrier over the concrete surface, a method sometimes used in tunnel construction.
4. The fourth, and relatively new, concept to counter the spalling threat is to provide vents in the concrete to alleviate pore pressure.

Finally, spalling can be caused by improper methods of fire suppression. For example, spalling can also occur when concrete, exposed to the heat of a fire, is hit by a cold water stream from a fire fighting hose.

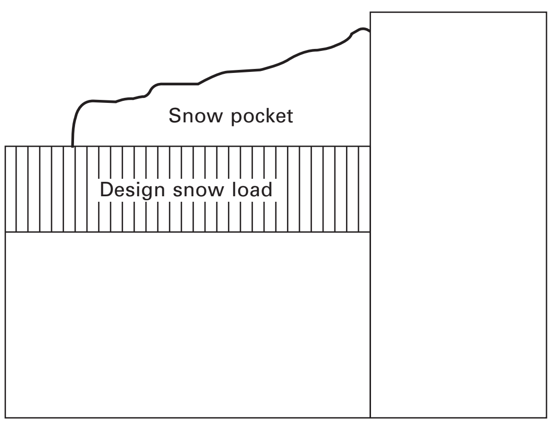
2.6 Static overloading

The type and extent of the damage due to overloading depend on the static scheme of the structure. In any case, the result of overloading is the development of high stresses exceeding the stress limits. For example, overloading of the beam or slab over the span results in appearance of 'bending' cracks in the areas of extreme bending moments. Overloading by the shear load in the areas adjacent to the supporting parts of the columns results in diagonal cracking in the vicinity of slab–column (beam–column) joints and separation of the column.

Even in the case when overloading results in the formation of small cracks, their appearance (or ignoring the need to repair them) can promote the accumulation of chlorides or other harmful species on the crack surfaces and in their vicinity when concrete is exposed to aggressive environments. The further stages of the damage will be dictated by inevitable corrosion of the reinforcement.

Mistakes in choosing the structural calculation scheme or in calculating the design loads result in overloading, and this often causes to damage to the structure or a part of it. The design and calculation of the structure on a regular uniform snow load, when the configuration of the structure provokes a formation of snow pockets (and, as a consequence, can cause overloading by several times), can serve here as an example (Fig. 2.3).

Another, also frequent, situation is when the structure is subjected to unexpected loads during the construction period, when the construction loads are not taken into account at design. The cracks and other defects developed trigger the corrosion process in the aggressive environment with consequent



2.3 Example of the under-estimation of design load resulting in overloading of the structure (an extra load from the snow pocket is not taken into account).

deterioration. For example, a contractor could decide to use equipment heavier than the design loads, or to load the multi-span concrete floor structure before concreting the joints, and then the concrete slabs designed for a multi-span floor structure work as single-span elements.

2.7 Impact

Dynamic overloading is another cause of damage to concrete and concrete structures. This section and the next sections deal, respectively, with impact and seismic loads as the main types of dynamic overloading.

The character of the impact damage in concrete structure depends, to a large extent, on the modulus of elasticity of concrete. In general, the higher the modulus of elasticity, the more brittle the material. This relation is known for high-strength concrete, which, in general, is more prone to brittle failure than normal-strength or low-strength concrete. During an earthquake, the structures absorb some energy, while the energy absorbed depends on the load and displacement. The small local displacement of high-strength concrete, for example, leads to significant growth of the load applied and, in turn to drastic growth of the local stresses and severe deterioration of the concrete, while in normal- or low-strength concrete the same energy is absorbed by local crushing or small spalling.

Let us compare different photos illustrating the effect of impact loads on structures made of high-strength concrete (Fig. 2.4), regular normal-strength concrete (Fig. 2.5) and fiber-reinforced concrete (Fig. 2.6). Figure 2.4 shows typical damage to an electricity cable trench on a wharf structure 850 m long made of high-strength concrete containing silica fume. It should be noted that, during three years of maintenance of this mooring line, about 30% of the cable trench slabs cast with silica fume concrete has been damaged. In other words, using a relatively brittle material (albeit having high strength) in cases where impact loads are expected can lead to severe damage and must be avoided.

Figure 2.5 illustrates damage observed in the concrete foundation of the bridge of electric connectors for refrigerator containers. This damage was caused by the crane operator, who put the container directly on the concrete foundation by mistake. The local limited spalling of concrete can be observed.

Figure 2.6 shows damage to the top course of a crane beam made of structural synthetic fiber-reinforced concrete. It can be seen that the typical angle of concrete spalling (~ 15°) is significantly less than the typical shear angle of 45°, which is usually observed in plain regular concrete. This fact seems to be a result of the energy absorbed by the fibers after the initial cracking occurred.

2.4 Typical impact damage of high-strength concrete containing silica fume.

2.5 Typical impact damage of regular concrete.

2.6 Typical impact damage of structural synthetic fiber-reinforced concrete.

2.8 Earthquakes

The main difference between earthquake action on the structure and other cases of overloading is that the seismic load is not a local effect, but an overall load applied to the whole structure. The seismic loads are cyclic sign-changing loads. Basically, an earthquake is a special case of the dynamic overload closely related to the frequency of the structure itself. When the structure's frequency coincides with the earthquake frequency, resonance occurs in the structure or in its parts. Various consequences of earthquake – such as settlements, ground breaks, and movements – lead to unpredicted static overloading of structures and, as a result, to their failure.

It would be useful to apply the general classification of structural damage due to earthquakes to all kinds of structures: concrete, soil, steel, etc. This approach is a part of performance-based design. It is used in the design of port structures and, in our opinion, can be successfully applied for design of concrete structures in other applications and not only for seismic loads. According to this approach, the International Navigation Association (PIANC) distinguishes among four acceptable damage levels in performance-based design: (i) serviceable; (ii) repairable; (iii) near collapse and (iv) collapse (Table 2.3).[8]

The acceptable level of damage is specified according to the specific needs of the users/owners of the facilities and may be defined on the basis of the acceptable level of structural and operational damage given in Table 2.3. The structural damage category in this table is directly related to the amount of work needed to restore the full functional capacity of the structure and is often referred to as direct loss due to earthquakes. The operational damage category is related to the amount of work needed to restore full or partial serviceability. Economic losses associated with the loss of serviceability are often referred to as indirect losses. In addition to the fundamental functions of servicing sea transport, the functions of port structures may include protection of human life and property, functioning as an emergency base for transportation, and acting as protection from spilling hazardous materials. If applicable, the effects on these issues should be considered in defining the acceptable level of damage in addition to those shown in Table 2.3.

Once the design earthquake levels and acceptable damage levels have been properly defined, the required performance of a structure may be specified by the appropriate performance grade S, A, B, or C defined in Table 2.4. In performance-based design, a structure is designed to meet these performance grades.

The performance-based seismic design approach described here makes frequent use of the terms serviceability and repairability. In view of this, it has to be emphasized that in concrete structures designed according to the performance-based approach under modern codes and standards the building serviceability and repairability should be guaranteed, although earthquakes of level 1 can cause local spalling and cracking.

Seismic load, especially its horizontal component, is the decisive load which defines the design of buildings in seismic areas. The horizontal component of a seismic load usually causes very high shear stresses in columns; liquefaction of sand provides overloading of submerged retaining structures.

Table 2.3 Acceptable level of damage in performance-based design

Operational	Structural	Acceptable level of damage
Little or no loss of serviceability	Minor or no damage	Degree I: Serviceable
Short-term loss of serviceability[b]	Controlled damage[a]	Degree II: Repairable
Long-term or complete loss of serviceability	Extensive damage in near collapse	Degree III: Near collapse
Complete loss of serviceability	Complete loss of structure	Degree IV: Collapse[c]

[a] With limited inelastic response and/or residual deformation.
[b] Structure out of service for short to moderate time for repairs.
[c] Without significant effects on surroundings.

Table 2.4 Performance grades S, A, B, and C

Design earthquake		Performance grade
Level 2	Level 1	
Degree I: Serviceable	Degree I: Serviceable	Grade S
Degree II: Repairable	Degree I: Serviceable	Grade A
Degree III: Near collapse	Degree I: Serviceable	Grade B
Degree IV: collapse	Degree IV: Repairable	Grade C

2.7 Crushing failure at bottom of column.[8]

Buildings constructed according to old codes and standards, which did not take seismic actions into account, have, as a rule, neither sufficient reinforcement to resist shear, nor proper connections, which causes local failures in columns (Fig. 2.7) and beams (Fig. 2.8) and sometimes leads to a collapse of whole buildings, as shown in Fig. 2.9.

The column failures in the lowest floor are especially characteristic of buildings constructed in Mediterranean countries on columns with an open ground floor. In this case, the lowest floor is often used for commercial purposes and lacks the stiffness provided by the infill at the upper floors (see Fig. 2.9).

Steel confinement reinforcement in concrete columns plays an important role in helping to resist seismic loads. The detailed view of the column with spiral confinement steel, which helped keep the concrete core intact, is shown in Fig. 2.10. It can be seen that the longitudinal reinforcement is tied to the exterior of the spiral; this detail leaves the longitudinal steel vulnerable.

The beam damage patterns include (i) typical 45° shear cracking generating

2.8 Close-up of characteristic X-shaped cracks and failure of coupling girders or short spandrel girders in the apartment building, Anchorage, Alaska. These girders were not properly designed for the shear demands.

from beam ends because of insufficient stirrups (see Fig. 2.10), (ii) concrete crushing at the column face, (iii) end beam pull-out/separation from the last column in the frame due to large drifts and inadequate bar hooks and development, and (iv) 90° hook pop-outs, which cause spalling and undermine the strength of the concrete core.[9]

2.9 Malicious damage

Faulty maintenance, human mistakes or wrecking/sabotage are considered here as cases of malicious damage. Both faulty maintenance and unintended human mistakes, such as smashing of vehicles, are a very frequent cause of the damage in the majority of industrial and transportation concrete structures. The consequences are different: from local spalling to the overall collapse of the structure. Figures 2.11 and 2.12 show damage to a storehouse

2.9 A weak-story mechanism developed at the first floor of a reinforced concrete building.

wall structure caused by a truck that moved in reverse gear and hit the gate portal by mistake. It can be seen that the absence of minimum reinforcement caused the brittle failure.

A very common cause of damage is smashing by vehicles exceeding the maximum allowable height (vehicle plus carriage) into transportation structures, such as gate frames, tunnels, and bridges (Fig. 2.13). As stated in the introduction, this kind of damage can be similar to that of earthquake or impact loading, depending on the type of structure. A very effective method of preventing this damage is to hang a sign limiting the height of the vehicles passing.

2.10 The detailed view of the column with spiral confinement steel, which helped keep the concrete core intact.[8]

2.10 Case studies

In this section two interesting case studies describing severe damage to concrete are reported. These studies can serve as good lessons to civil engineers involved in design, construction, and maintenance of concrete structures.

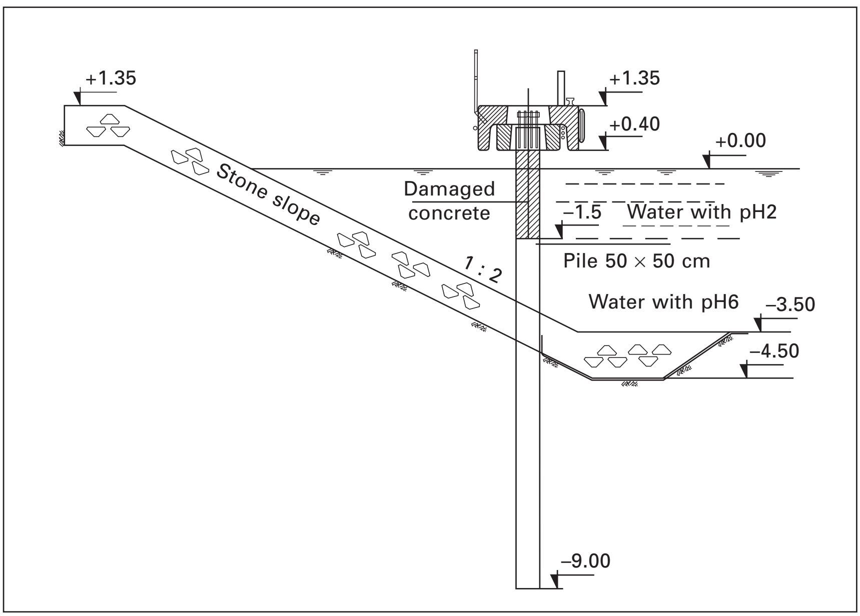
2.10.1 Case study 1

Kishon Fishery Harbor is a small marina and fishery port in the lower part of the Kishon river, Haifa Bay, Mediterranean Sea. Large chemical plants are located above on the watercourse. In 1999, new piers 350m long were erected.

2.11 Damage to a concrete column caused by human error.

2.12 Zoom at the column cross-section demonstrating insufficient reinforcement.

2.13 Damage of the gate frame by high vehicle.



2.14 Typical section of concrete quay in Kishon Fishery Harbor.

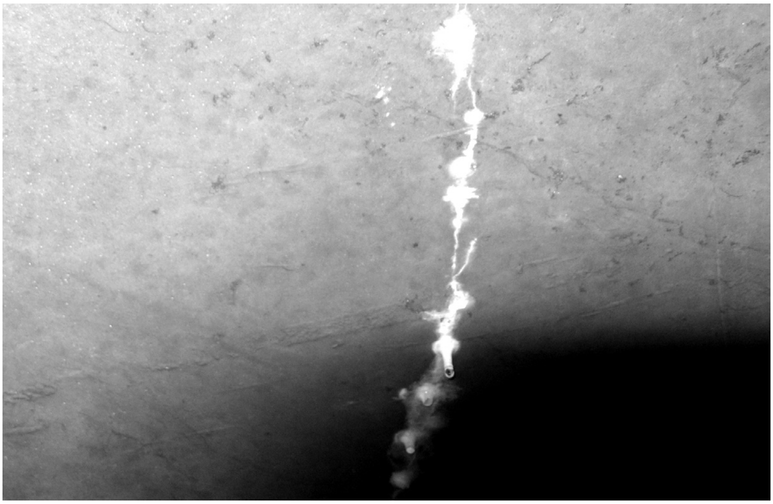
The pier structure represented u-slabs on single piles. The typical section of the quay structure is shown in Fig. 2.14. In 10 months after finishing the construction, it was observed that the piles lost more than 20 mm of concrete cover. It was surprising that severe damage was found only at depths of 1.5 m and above, and no damage at all was found below this level.

The investigation of the damage causes included numerous samples of water. It was discovered that the upper layer of river water (at depths of 1.5 m and above) consisted of a solution of water from the river and acidic waste solution, a by-product of the chemical plants. This solution has a pH of 2 and a density of 1.03, while the lower water layer represented seawater with a pH of 6 and a density of 1.05. The two water layers were almost not mixed with each other. As a result of acidic action, the external layer of concrete was significantly damaged (Fig. 2.15). The structure was finally rehabilitated using wrap jackets. This case and the judicial claim following the investigation were effective in exerting pressure on the chemical plants, which were finally obliged to stop discharging highly acidic waste water solution into the river.

2.10.2 Case study 2

The semi-open quay in this case study was built in 2003–2005. The precast concrete slabs were inspected prior to the installation, and no defects were observed. In 2007, the superstructure was tested and cracks (Fig. 2.16) were found and mapped (Fig. 2.17). The typical cross-section of the wharf is shown in Fig. 2.18. When the cracks were drawn on the structural scheme, it was discovered that all of them were located on the same line of the slabs,

2.15 The appearance of the pile made of concrete with dolomite aggregate after 10 months being submerged in acidic waste water solution with pH = 2.

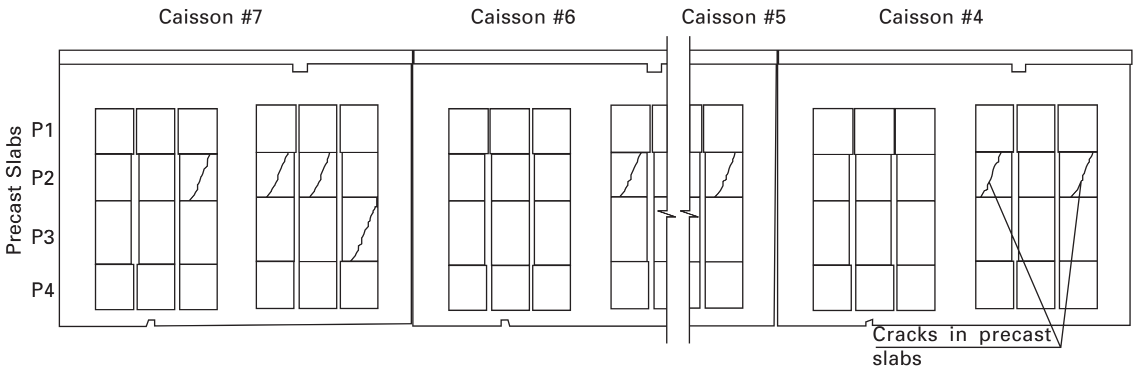


2.16 Caisson Wharf in Ashdod Port – typical overloading crack in precast slab.

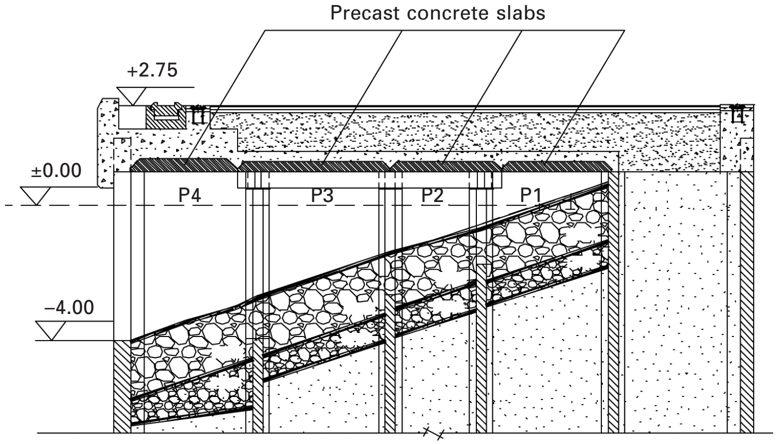
while no cracks were found in other areas. Because the reinforcement in all the slabs was the same, and no load was applied during the maintenance period, the conclusion was drawn that the cracking was developed due to overloading during the installation of the slabs.

2.11 Conclusions

Different types and sources of damage in concrete structures have been reviewed. A simple and convenient classification of damage by sources, which includes chemical attack, fire, overloading by static and dynamic (impact and earthquakes) loads and malicious damage (which covers faulty maintenance, human mistakes and wrecking), is suggested for the practicing engineer. Examples of different kinds of damage are given and discussed. Two case studies are reported, which help the practicing engineer in undertaking damage investigation and developing recommendations for rehabilitation of structures and preventing damage in future. Visual inspection and assessment of the appearance of concrete structures, in combination with advanced instrumental control and different physical and chemical methods of analyzing the materials (concrete and its constituents – cement matrix and aggregates, and also the reinforcement), as well as the environment, provide valuable information about the type and history of the loading which resulted in the damage, and helps in investigation of structural failures.



2.17 Caisson Wharf in Ashdod Port – partly precast slabs plan.



2.18 Caisson Wharf in Ashdod Port – typical cross-section.

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3
Types and causes of cracking in concrete structures

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Abstract: Distress mechanisms that can initiate or contribute to concrete cracking may be divided into those that occur in the plastic state and those that occur in the hardened state, and may also be differentiated by whether they arise from internal factors or from external factors, such as thermal expansion, overloading, restraint, or chemical reactions. The cracks themselves may range from minute internal microcracks to very large cracks caused by external factors of nature. A successful repair or mitigation of concrete distress requires a careful determination of the root cause of the cracking by methods that include visual observation, coring, and non-destructive testing.

Key words: microcracking, alkali–silica reaction (ASR), delayed ettringite formation (DEF), plastic shrinkage, freeze–thaw.

3.1 Introduction

It is a generally accepted notion among structural engineers that concrete will crack. Concrete, by its nature and composition, is a brittle material. As such, concrete has relatively low tensile capacity and is also susceptible to volume instability.

Often, an engineer is asked to diagnose a cracked structure in order to develop an appropriate repair. The first step in development of repair for a cracked concrete structure is to determine the cause of the cracking. Cracking may be caused by internal and external mechanisms, such as thermal expansion, overloading, restraint, and chemical reactions. The cracks themselves range from very small, internal microcracks to very large cracks caused by external, that is, environmental factors. Cracking may occur shortly after casting while the concrete is in a plastic state or when it is in the hardened state. In either case, cracking can be generated by internal or external factors. Often cracking is accompanied by volume change and the loss of moisture from either fresh or hardened concrete. In general, the term 'plastic shrinkage' is used for fresh concrete, while 'drying shrinkage' is used for hardened concrete.

3.2 Cracking in a plastic state

Significant movement of plastic concrete at early age may cause cracking. This movement may be due to volumetric reduction or to differential settlement within the concrete matrix.

3.2.1 Plastic shrinkage cracking

In the plastic state, cracking associated with volumetric change is often referred to as 'plastic shrinkage' cracking. Plastic shrinkage generally occurs due to a rapid loss of water from fresh concrete. The most common situation is the evaporation of surface water from the surface of the fresh concrete or from suction of the sub-base or formwork directly beneath the concrete.

Causes of plastic shrinkage cracking

The mechanism for the volumetric change that leads to plastic shrinkage cracking begins with the removal of water from the concrete paste by evaporation. Subsequently a complex series of menisci develops. The formation of the menisci generates negative capillary pressures that cause the paste volume to contract. The paste volume continues to contract until the water within the paste is no longer evenly distributed and separate zones of water are formed. The differential volume changes induce tensile stresses that are relieved by cracking (Mindess and Young, 1981). This cracking occurs when the capillary pressures are no longer evenly dispersed through the paste and they spontaneously equalize by rearranging. Prior to this rearrangement the pore pressures reach a maximum point referred to as the 'breakthrough' pressure.

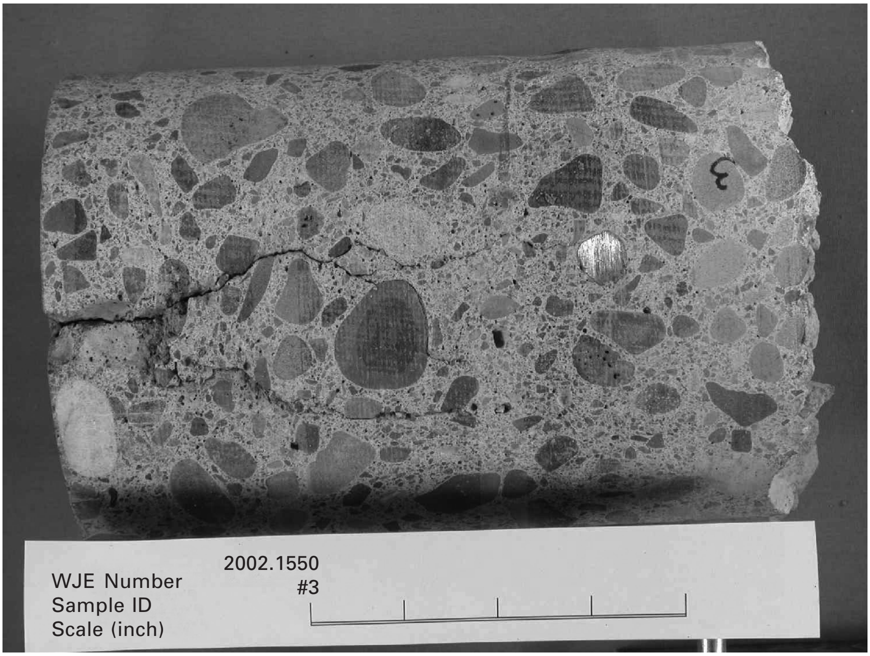
Plastic shrinkage cracks are identified by their short length, typically a few inches to a few feet in length. The cracks appear in an irregular pattern from a few inches to a few feet apart. A concrete slab exhibiting plastic shrinkage is shown in Fig. 3.1. Note that typical plastic shrinkage is a paste-related phenomenon. Figure 3.2 displays a core taken from the cracked area of the sample shown in Fig. 3.1. Note the crack propagation around the aggregate.

The primary factors that contribute to this change are external environmental conditions including:

- high air and concrete temperature;
- low humidity;
- high winds.

Evaporation factors may be prevalent in casting concrete with large surface areas such as paving and decking. To control plastic shrinkage of the

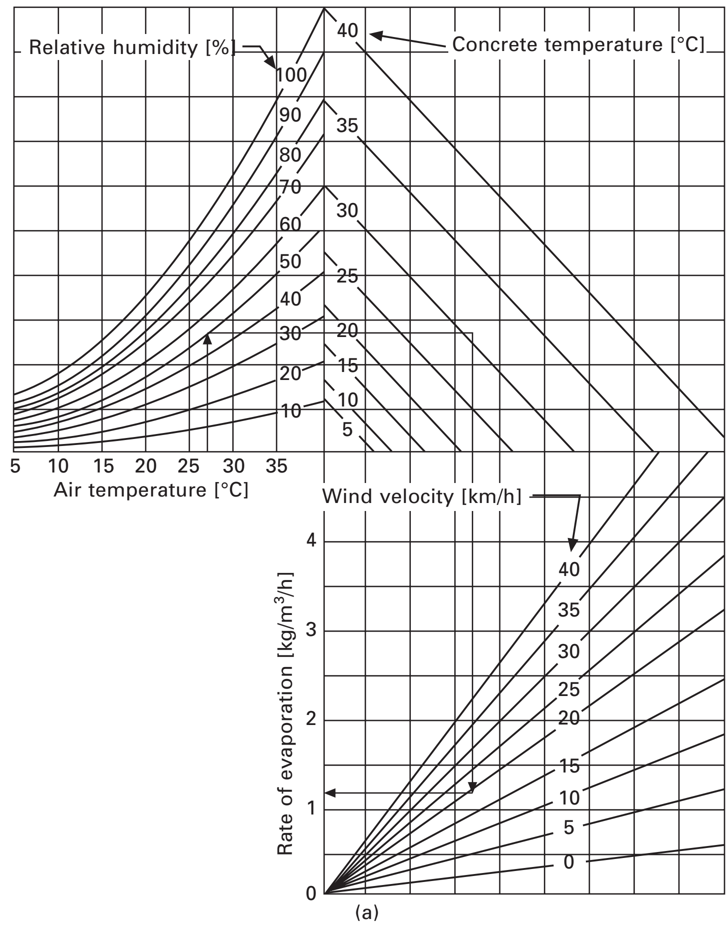
3.1 Plastic shrinkage in a bridge deck.



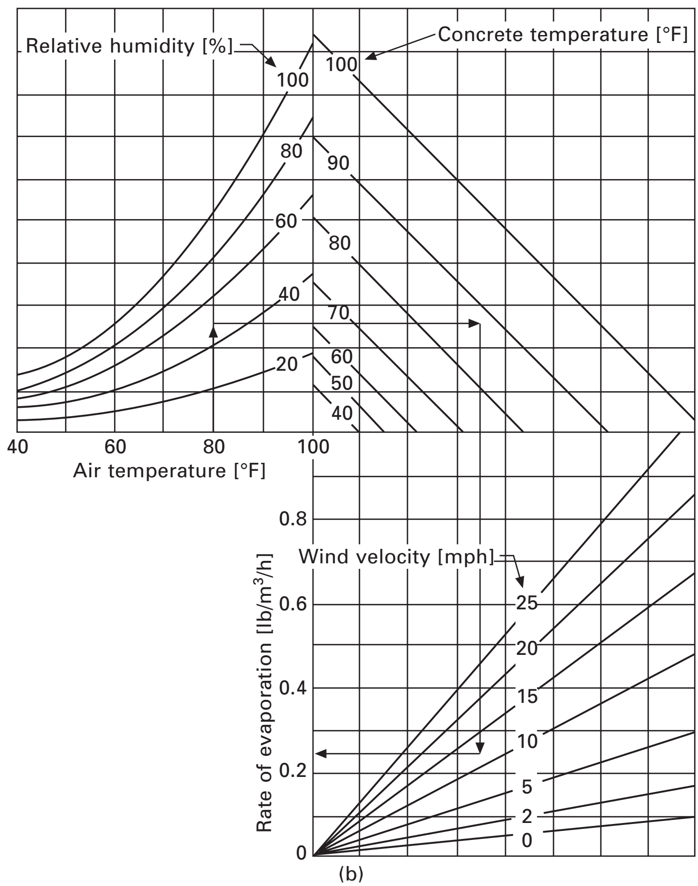
3.2 Core taken from a bridge deck with plastic shrinkage cracking. Note the crack pattern through the paste and around the hard aggregate. (Courtesy of Wiss, Janney, Elstner & Associates) concrete, the rate of water evaporation must be controlled. Figure 3.3 shows a commonly used nomograph for estimating water evaporation.

Mitigation of plastic shrinkage cracking

Precautions should be taken to control or prevent plastic shrinkage. Plastic shrinkage generally occurs during hot weather concrete casting. General steps to mitigate plastic shrinkage include the following:



3.3 Nomograph for estimating the rate of evaporation of water from a concrete surface. The arrows demonstrate an example for a day where the ambient air temperature is 80 °F (27 °C), the relative humidity is 50%, the concrete temperature is 87 °F (31 °C), and the wind velocity is 12 mph (19 km/h). The resulting rate of evaporation is about 0.25 lb/ft^2 /h (1.2 kg/m^2/h). (a) SI units; (b) Imperial units. (Courtesy of Portland Cement Association)



3.3 Cont'd

- moisten sub-grade, forms, or sub-surface;
- moisten the concrete aggregates that may be dry or absorptive;
- erect windbreaks, temporary or permanent, to reduce wind velocity over the concrete's surface;
- erect sunshades to reduce the concrete surface temperature;
- keep freshly mixed concrete cool;
- protect the surface of the concrete – protection includes fog spraying and appropriate curing;
- reduce the time between placing and curing;
- protect concrete immediately after final finishing.

3.2.2 Plastic settlement cracking

Early age movement of plastic concrete may lead to permanent distress in the form of plastic settlement cracking. This type of distress may be caused by multiple factors; however, the concrete typically undergoes a differential movement at a point following stiffening of the concrete, but prior to initial set.

Causes of plastic settlement cracking

The causes of plastic settlement cracking may be external, or it may be due to a relative difference in support or when elements have varying section depths. As these mixtures settle due to the force of gravity, coarse aggregate particles may be obstructed by reinforcing steel located near the concrete surface. Concrete mixtures exhibiting excessive bleeding or lack of cohesiveness have been reported to exhibit plastic settlement cracking. Early age movement or external vibration of concrete has been shown to yield limited or negligible effects on cement hydration and strength development, provided that these occur prior to concrete setting. However, if deformations exceed the early age strain capacity of the plastic concrete, cracking may occur.

Mitigation of plastic settlement cracking

Plastic settlement cracking may be minimized with appropriate revibration of the concrete; however, care should be taken not to vibrate during setting when concrete/steel bonding has begun. Actions that may reduce the occurrence of plastic settlement cracking include the following:

- using lower slump concrete mixes;
- using more cohesive mixes with higher fines content, possibly with air entrainment;
- reducing the setting time of concrete and bleeding quantity;
- increasing the cover of reinforcement;
- adjusting placement method for variable section depths of slabs and beams.

3.3 Cracking after hardening

Cracking of concrete after hardening can be caused by several factors including freeze–thaw cycling, creep, autogenous deformation, drying shrinkage, chemical effects, load-induced cracking, both external and internal, and thermal cracking. Each of these factors will be discussed in detail in the following sections.

3.3.1 Freeze–thaw damage

Weathering processes can initiate cracking in concrete structures. Two distinct crack progressions promoted by weather are thermally induced cracking from cyclic heating and freezing and scaling because of the interaction between the concrete surfaces and de-icing chemicals. Thawing and freezing is the most common weather-related physical distress. Distress arising from hydraulic forces occurs because of the concrete element being damaged by water freezing in the paste, aggregate, or both. Osmotic distress or scaling develops as a result of chemical diffusion and the movement of moisture near the surface of the concrete. The resulting distress is exhibited as a roughened and pitted surface. The following sections discuss each of the freeze–thaw-related distress mechanisms, and how each mechanism can be mitigated.

Freezing of cement paste

Freeze–thaw distress is generated by repeated hydraulic pressures that cause an expansion of up to 9% from water freezing in the paste volume of the concrete. As water in concrete pore spaces transforms from a liquid to solid state, the resultant expansive pressures lead to deterioration of the concrete paste. This is only one part of the pressure created by freezing processes. An additional pressure occurs as residual water is forced out of the concrete pores. This pressure, known as hydraulic pressure, increases as the pore space is consumed by the formation of ice, forcing residual water out of the pores. If this pressure is not dissipated, the tensile capacity of the paste is exceeded and the concrete suffers a localized rupture. In each successive freeze–thaw cycle, the cumulative effect causes expanding deterioration of the concrete. The deterioration is visible in the form of cracking, scaling, and general degradation of the surface paste.

Development of an appropriate air void system by means of addition of air-entraining agents added to the paste during batching may provide relief from these freeze–thaw and scaling progressions. Air entrainment of this type will only be useful to mitigate freeze–thaw if the entrained air pores are uniformly distributed and in close proximity to the residual water. The inclusion of these pores, if spaced sufficiently close together, supplies a relief zone for displaced residual water as hydraulic pressures build within the paste matrix. Since these relief zones need to be in close proximity to the residual water, well-distributed spacing of the entrained air pores is critical.

In the mechanism described above, resistance to freezing and thawing is a function of the degree of saturation by water, the rate of freezing and the average distance to a free surface (including pores) where ice can form safely. Thus low-permeable concrete with a w/c (water-to-cement ratio) of 0.35 or less, or partially dry concrete, will not experience freeze–thaw distress. In general, the critical degree of saturation has been found to be close to 0.80 (Mindess and Young, 1981).

Freezing of unsound aggregates (D-cracking)

Aggregates in concrete may also be susceptible to freeze–thaw effects. In pavements, a common aggregate-related freeze–thaw distress is D-cracking, which is a progressive structural deterioration of concrete beginning in certain types of susceptible coarse aggregates, caused by repeated freeze–thaw cycles when the aggregate has absorbed moisture.

D-cracking occurs over a long period and is generally initiated at the base of the slab. The base of the slab is often damp as the slab itself inhibits drying out. Moisture penetrates the pores of certain coarse aggregates and, as the moisture freezes, it expands to form ice. Once a crack has initiated from this frozen expansion, further freeze–thaw cycles cause the residual moisture to propagate deeper into the concrete section, initiating new cracks during the next freeze. It is important to note that traffic itself will not cause D-cracking. However, high traffic can accelerate the process as cracking initiated by freeze–thaw action is exacerbated by the fatigue action of the traffic-induced loading.

Early detection of D-cracking can be difficult. Typically, D-cracking occurs in areas of moisture concentration. In paving, longitudinal and transverse joints are often susceptible to D-cracking. Certain types of coarse aggregates are more susceptible to D-cracking than others. These materials, as often quarried, contain pore structures from 0.4–2.0 micrometers in diameter that can be deleterious in concrete. Studies have determined that such pore structures are conducive to freeze–thaw deterioration and D-cracking in particular.

Aside from aggregate type, the size of aggregate particles can also be a contributing factor to D-cracking. Larger particles have intrinsically longer water paths. As a result, the larger aggregates of a deleterious type will form more cracks, and smaller aggregates, typically of 13mm (1/2 inch) or smaller diameter, will typically perform better than larger aggregates.

D-cracking is most prevalent in geographical regions that have a larger number of freeze–thaw cycles. Repeated cycles generally keep the base material damp and permit moisture migration into the coarse aggregate. In the USA, D-cracking is often observed in Illinois, Indiana, Michigan, Ohio, Kansas, Missouri, and Iowa. In regions where the base material remains frozen, D-cracking seldom occurs, as the moisture remains frozen and is not made available for further migration by subsequent thaw processes. However, with the recent emergence of de-icing materials, these pavements may become more susceptible since some de-icing chemicals thaw the frozen concrete and adjacent base material.

D-cracking can best be controlled by the use of sound and durable coarse aggregate. Aggregates such as quality dolomites and igneous varieties are not prone to D-cracking, whereas aggregates with high portions of chert and limestone can be more susceptible. Where D-cracking is likely to be a concern, it is recommended that specific aggregates be tested to determine their propensity for promoting this form of distress.

Two test procedures are generally accepted by the industry to determine the likelihood of D-cracking in cast concrete. The primary method is to test the coarse aggregate in concrete specimens. These freezing and thawing tests are conducted over several hundred cycles. Between freeze–thaw cycles, the rate and extent of the expansion of the concrete specimens are measured. Since each freeze–thaw cycle can take a day or more to conduct, these tests will be time-consuming. ASTM C666, '*Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing*' (ASTM, 2008a), was issued to assess freeze–thaw resistance of aggregates, as well as the effectiveness of the in-place air void system. It requires either 300 cycles or enough cycles for the dynamic modulus to reach 60% of its initial value, whichever occurs first. The relative durability factor (*DF*) is calculated from this test, where the durability is expressed as the percentage of the initial dynamic modulus (*P*) times the number of cycles (*N*) divided by 300:

$$DF = \frac{PN}{300} \qquad [3.1]$$

where

$$P = DM^2/DM^2_{\text{initial}} \qquad [3.2]$$

Generally, a durability factor of less than 40 suggests that the aggregate is unsatisfactory, while a value over 60 is indicative of an aggregate that will not exhibit cracking in pavements.

Alternatively, the Iowa Pore Index Test (Myers and Dubberke, 1980) may be used for a shorter turnaround of results. This test measures the amount of water that can be injected into an oven-dried aggregate between one and 15 minutes after application of an air pressure of 240 kPa (35 psi). This test is generally effective in identifying aggregates with pore diameters in the range of 0.4–2.0 micrometers, and correlates well for homogeneous materials such as quarried stone. However, notable problems have been reported with this test as it applies to siliceous gravels.

Salt scaling

In addition to physical distress from freeze–thaw cycles in harsh and cold climates, concrete paste may also be chemically and physically attacked by the application of de-icing chemicals. Salt scaling may be confused with initial freeze–thaw distress on a large surface area like a paving surface. The concrete surface that experiences salt scaling becomes roughened as the surface mortar is lost and the coarse aggregate is exposed at the surface.

Scaling generally occurs in a damp environment that is exposed to several freeze–thaw cycles. In addition to freezing of the moisture in the capillary structure of the paste, internal pressures can build up in the paste from differential concentrations of alkali solution. These pressures are termed osmotic pressures. The level of osmotic pressures is generally greater than normal hydraulic pressures that occur when water freezes. The exact cause of surface scaling is still unknown; some attribute it to the sudden thermal change at the near-surface region as de-icing chemicals change the surface temperature. In addition to near-surface distress in the form of scaling, some de-icers, when used at high application rates, may attack the cementitious hydration products, leading to progressive decomposition of the paste.

In general, sound concreting practices will eliminate most scaling concerns. An air-entrainment admixture should be used and coupled with mix designs of low w/cm (water-to-cementitious materials ratio) of 0.35–0.40. While scaling may occur in concrete exposed to de-icers even with air-entrainment, the level of distress is typically greater in mixes lacking proper air void systems. In addition to adjustment of the mix design, care should be taken not to over-finish the surface. Over-troweling may weaken the surface paste and allow the concrete to become more susceptible to freeze–thaw action. Moist curing has been shown to reduce the potential for scaling by reducing the amount of microcracking and surface irregularities as the curing process controls the rate of hydration and evaporation, especially near the surface.

3.3.2 The role of creep

Concrete creep plays an important role in relating the stability of concrete to the internal and external stress conditions. Creep of concrete is an complex subject in itself, and thus only a cursory review as it relates to the potential for cracking is included here. The magnitudes of potential creep and shrinkage strains are generally comparable, and each process can undergo considerable non-elastic deformation. In addition, both creep and shrinkage of hardened concrete occur within the paste fraction. To understand the effects of drying shrinkage, one must understand the role of the interplay between creep and drying shrinkage. Creep generally decreases the potential for cracking of concrete due to stress relaxation effects. The level of allowable creep should be considered when designing and repairing structures for minimal movement or deflection of slabs and beams. The common factors shared between the two phenomena are quality of the concrete, conditions of joints and spacing, curing conditions, and ambient temperature. Practitioners should understand creep phenomena, the factors that contribute to or mitigate creep, as well as the role that creep plays in cracking of concrete.

Definition

When concrete is loaded, the load-induced deformation may be divided into two parts, the immediate deformation as a result of the load, and the long-term time-dependent deformation that begins when the concrete is loaded and continues at a decreasing rate for an indefinite time as long as the concrete remains loaded. This long-term deformation is the phenomenon referred to as creep. In normal stress ranges, creep is proportional to stress up to approximately 50% of the ultimate strength. In this range, creep is essentially:

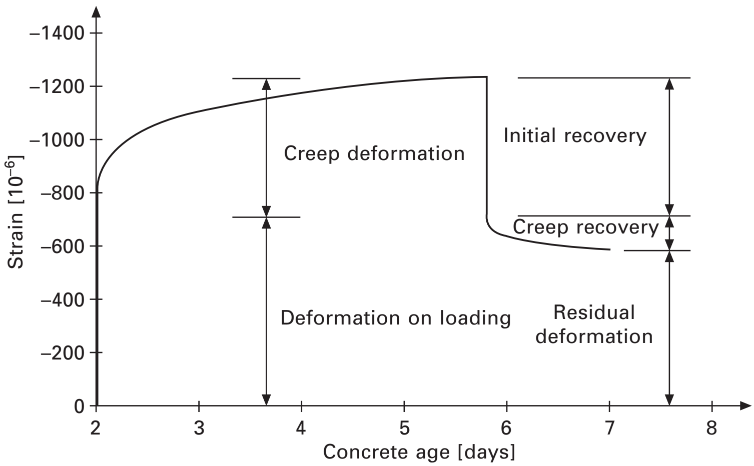
$$\text{specific creep } (\Phi) = \frac{\varepsilon_{cr}}{\sigma} \qquad [3.3]$$

where ε_{cr} is the total creep strain and σ is the applied stress. Figure 3.4 demonstrates how strain varies as a function of time after loading.

Factors affecting behavior

In order to mitigate and control the phenomenon of creep, the factors affecting creep must be understood. Accordingly, factors that affect creep and drying shrinkage cracking include w/cm, curing, temperature, moisture, cement composition, and chemical admixtures.

While it has been studied extensively, the relation between creep and w/cm is still not well understood. Changes in the w/cm ratio naturally lead to changes in the cement content and strength of the concrete. In addition, changes in the paste volume lead to changes in Φ. As the ratio of w/cm increases, so generally does Φ but not always in a predictable manner.



3.4 Typical creep curve for plain concrete (after Mindess, 1981).

In addition to the role of the w/cm ratio, the extent of creep can also be influenced by curing conditions. Specifically, the time of curing (at loading) affects the magnitude of creep. Traditional moist curing is not as effective in reducing creep as is steam curing. The combination of available moisture with elevated temperatures during curing allows for the reduction of creep stress. This reduction is more pronounced at elevated steam-curing temperatures.

Temperature can play an important role in limiting creep. For internal concrete temperatures up to 80 °C (175 °F), creep is essentially proportional to temperature. However, beyond this temperature threshold, creep is reduced as the temperature is increased. The amount of reduction may vary, but even at short durations of exposure to high temperatures, creep is reduced. This can be rationalized as early 'aging.'

Obviously, moisture conditions will influence the hydration process of concrete and, as hydration (and the consumption of water) occurs, resulting stresses accumulate inside the paste matrix. A component of these stresses when the concrete is subjected to loading is creep stress. As noted above, the w/cm ratio can provide additional water and promote hydration of the paste. Therefore, as less water is available in the paste matrix, less creep stress will be present. The greatest decrease occurs on drying in a 40% relative ambient humidity.

Both sides of the w/cm ratio can have a pronounced effect on creep. As a general rule, increasing C_3A or decreasing C_3S can increase or decrease-respectively, creep stress. Type I cements will generally lead to greater creep stresses in the concrete than Type III cements. This is due to the amount of C_3A in Type I cements versus that in Type III systems. Levels of gypsum content are often adjusted to mitigate creep. There is an optimal sulfate content for creep mitigation; however, this amount tends to be higher than the amount required for minimizing drying shrinkage.

A wide variety of chemical admixtures is used in today's high-performance concretes (HPCs). Among these are super plasticizers, water-reducing admixtures, and set-retarding agents. As stated previously, creep and drying shrinkage are closely related stresses. Admixtures that increase drying shrinkage may also increase creep. Admixtures that diminish the water demand reduce the w/cm ratio and, as a result, creep effects can be partially offset. However, the actual performance of the specific admixture and components should be considered and tested to verify reduction in shrinkage potential. Generally, concrete placed at a higher slump, even when aided with chemical admixtures, will exhibit higher shrinkage potential.

Autogenous shrinkage

The autogenous deformation of concrete is defined as the unrestrained, bulk deformation that occurs when concrete is kept sealed and at a constant temperature. As we move toward HPCs, the need to address the root causes of autogenous shrinkage cracking is more critical.

Autogenous deformation occurs when no additional water is added after mixing and the concrete begins to dry out. This drying out can occur even without moisture being lost to the atmosphere. The consumption of water during hydration is known as self-desiccation and leads to autogenous shrinkage (Mindess and Young, 1981). Concretes with w/cm ratios of less than 0.42 may be susceptible to autogenous shrinkage.

Autogenous deformation may be caused by different mechanisms. For example, the growth of certain salt crystals, such as ettringite, may cause expansion during hydration. More commonly, the deformation results from the products of cement hydration occupying less space than the reacting water and cement. This phenomenon, chemical shrinkage, causes a successive emptying of the pore structure and leads to tensile stresses in the pore water through the formation of menisci, similar to plastic shrinkage. Menisci formation causes the relative humidity to drop, and self-desiccation occurs in the cement paste. The concrete may crack globally (macrocracking, if restrained), or locally at the surface of non-shrinking aggregates (microcracking).

Strategies to mitigate autogenous shrinkage cracking may be of either an autogenous or a non-autogenous nature. Autogenous strategies include modifications to the concrete mixture proportions, such as adjustments in the w/cm ratio. Strategies to offset the dimensional changes associated with autogenous shrinkage include changes to the cement composition and/or fineness, or supplementation with cementitious additives like fly ash or ground granulated blast furnace slag. Changes to the mix proportioning are also often considered. In essence, shrinkage or self-desiccation can be mitigated by increasing the strain capacity of the concrete or by reducing the shrinkage of the material.

Drying shrinkage

Drying shrinkage is similar to creep in many ways. In common with creep, drying shrinkage is initiated by volume changes that result from the consumption and rearrangement of moisture within restrained concrete. The strain generated is of the order of 600×10^{-6}mm/mm (in/in). Drying shrinkage is developed as the mixing water is consumed during hydration of the concrete. This water is often consumed from external environmental conditions. For drying shrinkage cracking to occur, restraint must be present. If there were no restraint, concrete would not crack. Internal restraint can be generated on the surface of the aggregate particles contained in the concrete. External restraint can be generated from subgrades, or as can be the case with larger sections, from tensile stresses developed at the interior of the paste/aggregate interface. As the tensile capacity of the concrete is exceeded by the stress of the restraint, the concrete relieves the stress in the form of cracks.

The magnitude of the tensile stresses created by the concrete's moisture-induced volume changes will be influenced by a combination of factors. These factors include the amount and rate of shrinkage, the degree of restraint, the amount of creep, and the modulus of elasticity. The shrinkage experienced is primarily shrinkage of the cement paste. Therefore, the amount, size, and type of aggregate can influence the amount of drying shrinkage, simply by substituting for some of the paste volume: if the paste volume is reduced, the amount of shrinkage is reduced. The quality of the aggregate will have a significant effect on drying shrinkage as well. Because the aggregate provides internal restraint, it makes sense that the stronger and sounder the aggregate, the more cracking from shrinkage is mitigated.

Mitigating drying shrinkage

Shrinkage cracking can be controlled and mitigated by a combination of design considerations. In slabs, proper joint spacing and joint design is required to permit controlled cracking. The 'control' joints create a periodic plane of weakness in the slab section that permits the concrete to crack and relieve the tensile stresses accumulated from drying shrinkage. These control joints must be carefully designed and detailed to ensure that the concrete is not restrained at these locations. In addition, the control joint needs to be deep enough to ensure an adequate plane of weakness is created in the section.

In addition to control joints, careful consideration should be given to the concrete mix design, including reduction of paste volume and w/cm ratio, as well as aggregate amount, gradation, and quality, and, finally, the possibility of supplemental admixtures and polymeric fibers. Since drying shrinkage occurs in the concrete's paste portion, simply reducing the proportion of paste to aggregate volume can improve the concrete's resistance to drying shrinkage. The coarse aggregate size and type will influence drying shrinkage as well. The larger coarse aggregates will increase stresses at the aggregate paste interface. In addition, lightweight aggregates are stable, but their low modulus of elasticity will generate higher shrinkages than in normal weight concrete. Finally, supplements such as water-reducing admixtures will change the w/cm ratio and improve the concrete's resistance to drying shrinkage. Small supplemental polymeric fibers have recently been used in mix designs to absorb the internal tensile stresses generated during hydration. These fibers have been shown to reduce drying shrinkage cracking.

In addition to the effects of the mix design, environmental factors such as relative humidity, rate of drying, and time of drying can also influence drying shrinkage. Moist curing has been effective at controlling the rate and time of drying and hydration, thereby reducing shrinkage stresses. Temperature and relative humidity should be considered in developing an adequate curing program prior to placement.

3.3.3 Chemical distress cracking

A typical concrete matrix may contain several diverse components that can be chemically incompatible internally or with the environment and ultimately result in chemical-related distress. Paramount among these chemically induced distress mechanisms are alkali–silica reactivity (ASR), sulfate attack, and delayed ettringite formation (DEF). Each of these conditions can lead to additional durability concerns with reinforced concrete. Identifying this distress often requires a detailed evaluation at a microscopic level. Experienced petrographers (Fig. 3.5) can distinguish ASR from DEF, for example. These distress conditions are otherwise difficult to distinguish in the field. What is more, these types of concrete distress can often be very difficult to repair. Care should be taken to mitigate these chemical reactions prior to casting, as repair of these distress types can be very costly.

Leaching and efflorescence

Leaching and efflorescence are often noted in the field where water has migrated through the concrete structure (Figure 3.6). This water migration may pose an

3.5 Petrographer at work. (Courtesy of WJE)

3.6 Efflorescence visible below a conventional reinforced concrete deck. (Courtesy of WJE)

additional durability concern. Efflorescence can be misdiagnosed as chloride contamination. Efflorescence is actually deposited salts that crystallize after migration of a solution through the concrete, followed by either evaporation of water or interaction with carbon dioxide in the atmosphere at the concrete surface. These minerals are generally sulfates and carbonates of sodium, potassium, or calcium with the major component being calcium carbonate. Since any efflorescence is a product of the migration of water through the concrete section, its presence may be a symptom of water infiltration-related distress, which could further other on-going distress mechanisms. Often, the causes of the efflorescence and water infiltration need to be addressed. Common remedies include providing a membrane or coating on the surface of the concrete.

Sulfate reaction

Sulfate attack can affect concrete through either internal or external reactions. General sulfate-related distress observed on the surface of concrete is referred to as external sulfate attack (or simply sulfate attack). External sulfate attack occurs when soil and groundwater contain sufficient sulfate levels, in the forms of sodium, potassium, magnesium, or calcium sulfate, which react with the cementitious paste of the concrete. Generally, deterioration starts at the contact zone between the concrete and the sulfate-containing environment, forming a propagating front. While concrete ahead of this front is essentially normal, concrete behind this front is completely changed in composition and texture, with its integrity significantly lost. Initially, sulfate reacts with the monosulfur aluminate phase of the paste from the cement hydration, forming ettringite, $C_3A{\cdot}3CS{\cdot}H_{32}$. These reactions are highly expansive, and will generate stresses exceeding the tensile strength of the concrete, resulting in cracking and disintegration of the concrete. In later stages, most of the cement paste may be replaced by ettringite and gypsum. The extent of sulfate attack, however, often depends on the quality of the concrete and the type of sulfate. More porous concrete (higher w/cm) may be more readily attacked than low-permeability concrete. Magnesium sulfate is generally more aggressive than other forms of sulfate because magnesium ions also participate in the reaction, replacing calcium and forming brucite (magnesium hydroxide). Calcium sulfate is usually the least reactive. An illustration of a more aggressive sulfate attack is shown in Fig. 3.7.

Sulfate attack can be progressive: as long as sulfate ions can penetrate the concrete surface, further reaction may occur throughout the concrete interior. This mechanism has been understood by engineers for many years and, as a

3.7 Sulfate attack at the base of a conventionally reinforced concrete column result, cements of Type II and Type V containing less than 8% and 5% C_3A, respectively are specified to mitigate sulfate attack. Limitations on concrete w/cm are often required, with the intent of lowering surface absorption of sulfate ions into the concrete. In addition, supplementation with silica fume, fly ash, and ground slag has been proven to provide resistance to sulfate attack, provided that the C_3A content of the cementitious material system is limited, and the impermeability of the concrete matrix is preserved.

Sulfates in the soil or groundwater should be measured if sulfate attack is suspected. Sulfate ions are commonly measured in parts per million (ppm) and the scale of severity, according to the ACI 318 Building Code for Concrete (ACI, 2008, Table 4.2.1) is:

- **Negligible** – When sulfate content is less than 150 ppm (mg/l) in water, no restriction on cement type is necessary.
- **Moderate** – When sulfate content is between 150 and 1500 ppm in water, Type II cement with a pozzolan additive should be used.
- **Severe** – When the sulfate content is between 1500 and 10 000 ppm in water, Type V cement should be used, along with a w/cm ratio below 0.45.
- **Very severe** – When the sulfate content exceeds 10 000 ppm in water, Type V cement should be used with a pozzolan and a w/cm of below 0.45. In addition, the ACI Building Code recommends a minimum f'_c of 29 MPa (4250 psi) (Mehta and Monteiro, 2006).

Alkali–silica reaction

Some types of siliceous aggregates can react with hydroxyls developed during the hydration of Portland cement in the presence of alkali metals, such as sodium and potassium. These siliceous aggregates are often amorphous, including primarily acidic volcanic rocks, opal, and chert. Some strained quartz may also be reactive. The reaction forms a hydrophilic amorphous gel in and around the aggregate particles, which is able to absorb large amounts of water, and thus generate significant volume increases in the concrete section. The reaction may eventually overcome the concrete tensile capacity and result in severe cracking. The outward signs of alkali silica reaction include surface aggregate pop-outs and map cracking.

The occurrence and extent of ASR depends on three factors: (i) alkali content of the concrete; (ii) type and concentration of the reactive aggregate; and (iii) readily available moisture. If one of the factors is eliminated or controlled, the deterioration due to ASR can be significantly reduced.

The phenomenon of ASR was first studied by T.E. Stanton in the early 1940s (Stanton, 1942). The alkalis usually come from the manufacture of Portland cement clinkers, which are measured as percentages of Na_2O

equivalent. Experimental data have shown that Portland cements with an equivalent Na_2O content greater than 0.6% can cause deleterious alkali–silica expansion. Accordingly, Portland cements containing less than 0.6 Na_2O are classified as low-alkali cements and are produced in most of the USA. However, using low-alkali cement alone is not a guarantee for mitigating ASR in concrete. More recently, it has been recognized that the total alkali load of the concrete must be controlled in order to reduce the risk of ASR when a reactive aggregate is used. A generally accepted limit is 4 lbs of Na_2O equivalent per cubic yard of concrete (2.4 kg per cubic metre).

Delayed ettringite formation

The most important form of internal sulfate attack is delayed ettringite formation (DEF). The sulfates normally originate in cementitious materials with high sulfate contents. DEF has been linked with elevated temperatures of fresh concrete, either due to an internal cause such as hydration of cement in mass concrete, or to an external cause such as steam-curing, a method often used in precasting. Ettringite is normally one of the earliest phases formed during cement hydration. However, when the concrete temperature is higher than approximately 65 °C (150 °F), ettringite is unstable and the formation of the primary ettringite is halted. The constituents of ettringite are dispersed as monosulfate and calcium silicate hydrates. These constituents form secondary ettringite in already hardened concrete when thermodynamic conditions become favorable, i.e., at relatively lower temperature with available moisture, to result in paste expansion followed by macro-cracking of the concrete. This cracking has an irregular appearance similar to map cracking caused by ASR. As a result, DEF and ASR may be confused with each other and misdiagnosed in the field. Petrographic analysis is generally required to distinguish theses two types of chemically induced cracking.

Microscopically, ASR-induced cracks propagate through reactive aggregate particles, and sometimes cracks can be observed to originate from aggregate particles. In addition, cracks are often filled with alkali silica gel. On the other hand, DEF-induced cracks occur as peripheral cracking, with cracks propagating around aggregate particles that are normally not cracked. Cracks become filled with secondary ettringite crystals.

Mitigation of internal chemical attack

The deleterious effects of these chemical attacks are an engineering concern, as they occur throughout the cement paste. Distress is often a function of available moisture and, as a result, is often observed in damp environments. Repair can be difficult and costly and, in some situations, full replacement may become necessary. Replacement of elements such as pretensioned girders can be especially costly, and conditions can even lead to bridge deck removal. The designer should take note of the importance of mitigation.

Alkali–silica reactions can be mitigated by limiting the total alkali load of the concrete, by controlling moisture levels, and by the use of sound, non-reactive aggregate. Mitigation of ASR can also be accomplished with the use of supplementary cementitious materials. Laboratory tests and field practice have demonstrated that partial replacement of Portland cement with adequate amounts of fly ash, especially Class F, ground granulated blast furnace slag, silica fume, and some natural pozzolans, or a combination thereof, can effectively mitigate ASR-induced cracking. More recently, some chemical admixtures, primarily lithium nitrate, have been developed to provide mitigation of ASR.

Many standard tests have been developed to assess the potential reactivity of aggregate and the effectiveness of supplementary cementitious materials in mitigating ASR. Among these test methods, ASTM C289, C1260, C1293, and C1567 are the most frequently used (ASTM, 2007a, b; 2008b,c). ASTM C289 is a chemical test that measures the reaction of the crushed aggregate with sodium hydroxide. This test, while yielding prompt results, has proven to be unreliable at times. ASTM C1260 is an accelerated mortar bar test that produces results at 16 days. Cured mortar bars are submerged in 1N NaOH solution at 80 °C and the change in length of the mortar bars is monitored, for 14 days. While the test conditions are harsh and often produce false positives, aggregates that pass the test, with an average expansion < 0.10% at 14 days, are virtually assured to be non-reactive. ASTM C1293 is an accelerated concrete prism test that can be used to evaluate the potential reactivity, as well as the effectiveness of mitigations that rely on the use of supplementary cementitious materials. Concrete prisms containing an elevated alkali content are exposed to 100% relative humidity at 38 °C for 12 months. At the end of 12 months, if the average expansion is less than 0.04%, the aggregate or aggregate–cementitious materials combination is considered innocuous. If the expansion is 0.04% or greater, the aggregate is considered reactive and the mitigation method is considered non-effective. This test is the most realistic standard test and is considered reliable by the industry. However, the 12-month testing period can make it impractical for meeting construction schedules. ASTM C1567 is an accelerated mortar bar test similar to ASTM C1260 that is designed to test the effectiveness of mitigation of ASR by use of supplementary cementitious materials.

The key to mitigation of DEF is the control of concrete temperature. This can be achieved by use of reduced proportions of cementitious materials, by substituting with supplementary cementitious materials, or through the use of physical methods that reduce the internal concrete temperature, or the curing temperature, in the case of steam curing. It has been reported that postponing the starting time for high-temperature curing may also reduce the risk of DEF. The composition of the Portland cement may also play an important role. Limiting the C_3A and sulfate contents of the cement can also help to prevent DEF. Similarly to the mitigation of ASR, replacing a portion of the Portland cement with supplementary cementitious materials will mitigate DEF. The mechanism in which these materials mitigate DEF is still debated, but it is generally accepted that supplementary cementitious materials help reduce the C_3A content of the total cementitious materials, and thus reduce the calcium hydroxide content in the paste. Since calcium hydroxide is required for the formation of secondary ettringite crystals, and also reduces the permeability of the concrete, the result is a reduction of the moisture exposure of the concrete.

3.3.4 Load-induced cracking

The most common forms of cracking recognized by engineers arise from external loads. These load-induced cracks can form from thermal forces, internal and external, and from overstressed conditions generated by externally applied loads. In each case, the external load-induced stresses exceed the tensile capacity of the concrete and lead to cracking. Often, the stresses can be reduced by providing additional restraint or strengthening. Both the ACI Building Code and Association of State Highway and Transportation Officials Bridge Specifications (AASHTO, 2002) require reinforcing steel to carry tensile stress and to promote adequate distribution of cracks and reasonable limits on crack width. Studies of crack width have been on-going for several years. However, the most widely used crack width equation was developed by Gergely and Lutz (1968). This equation is used to predict the most probable surface crack width from bending stress. The equation is as follows:

$$w = 0.076\,\beta f_s \sqrt[3]{d_c A} \times 10^{-3} \text{ (in. lb.)} \qquad [3.4]$$

where w = probable maximum crack width, in inches (1 inch = 25 mm), β = ratio of distance between the neutral axis and tension face to distance between neutral axis and centroid of reinforcing steel, f_s = reinforcing steel stress, in ksi (1 ksi = 6.89 MPa), d_c = thickness of cover from tension fiber to center of closest bar, in inches, and, A = area of concrete symmetric with reinforcing steel divided by the number of bars, in inches. The reader should reference ACI 318 (ACl, 2008) and AASHTO Bridge Specifications (AASHTO, 2002) for additional information on reinforcement spacing and cover for crack control.

Construction loads can be the largest loads experienced by a structure during its life. In addition, these loads typically occur when the concrete of the structure has not fully matured. Design professionals often overlook these loads in the development of a concrete design. In addition, precast and pretensioned elements are often transported to a project and subjected to stresses that can produce cracking during their transportation. Care should be taken to ensure that concrete components are correctly supported during transportation.

3.3.5 Thermal cracking

Thermal stresses in concrete can be internal or external. These stresses can and do develop cracks in the concrete matrix. Thermal stresses are prevalent in mass concrete, precast, prestressed and all structures that are exposed to large external temperature changes. The following sections provide a brief explanation of both internal and external thermal stresses, along with their effects, mitigation, and repair.

Internal thermal cracking

Thermal stresses can be developed in a number of ways, but for this discussion they will be categorized as internal or external. Internal stresses are developed from unequal heat of hydration-induced volume changes. These changes generally occur in mass concrete elements during casting. In these elements, the internal temperature is greater on the interior of the structure since the exterior will cool earlier in the hydration process. In large elements, unmitigated thermal stresses can reach temperatures in excess of 120 °C (250 °F). Temperature-induced cracking in mass concrete can be mitigated if proper measures are taken to reduce the amount and rate of the temperature change. Methods include the following:

- use of low cement content to control the heat of hydration;
- casting of smaller segments when possible;
- insulation of concrete surfaces to provide a uniform temperature change;
- provision of artificial cooling.

Artificially controlling the internal temperatures during placement can be accomplished by the use of cooling pipes placed in the interior, allowing chilled water to pass through the structure so as to regulate the internal temperature.

Thermal expansion

In general, concrete is a brittle material. However, unrestrained concrete will expand and contract without cracking. The coefficient of thermal expansion is defined as the change per unit length per degree of temperature change.

An average range of values for the coefficient of thermal expansion of concrete is approximately 5.4–12.6 × 10^{-6}/°C (3–7 × 10^{-6}/°F). Thermal expansion and contraction of concrete vary with factors such as aggregate type, cement content, w/cm, relative humidity, and concrete age. The amount of strain that a temperature change produces is directly proportional to the coefficient of thermal expansion. If restrained at the ends against contraction, the stress can be approximated as $f = \alpha E$. Given a normal strength concrete, f'_c = 21 MPa (3000 psi) and a coefficient of expansion of 10 × 10^{-6}/°C (5.5 × 10^{-6}/°F), it can be shown that a temperature change of 14 °C (25 °F) will crack the concrete section. Designers should consider restraint conditions to permit movement facilitated by the use of properly designed contraction, expansion, and isolation joints.

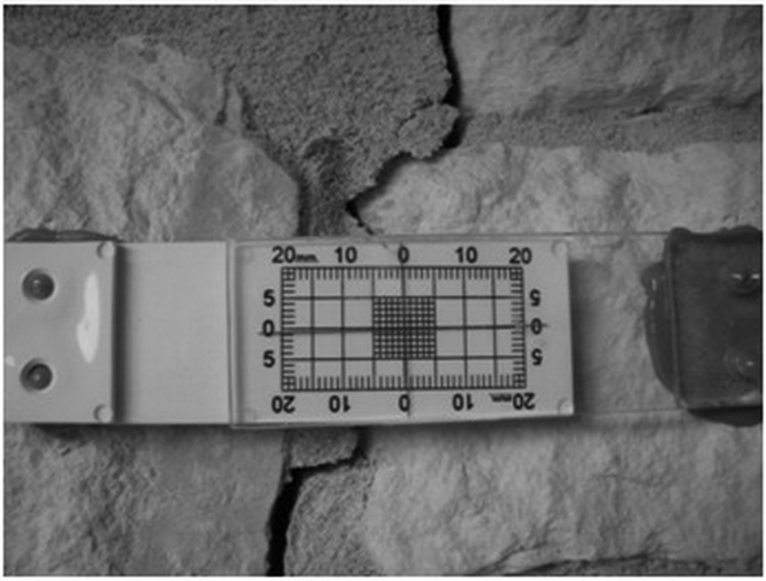
3.4 Assessment of distressed concrete

3.4.1 Assessment and evaluation of cracks

Successful concrete repair is governed by the ability of the design professional to identify the root cause of the cracking, in order to adequately counteract the forces that produced it. The process begins with the evaluation, assessment, and identification of the root cause of the cracking and distress. Many unsuccessful repairs have addressed the symptom of the distress while ignoring the root cause. To evaluate and assess the root cause, the practitioner has several tools available, including visual assessment, and both non-destructive and destructive forms of testing. Each approach has its advantages and disadvantages, and a successful evaluation may require the use of a variety of techniques in tandem. Cracks that compromise the strength, stiffness, or durability of an element should be repaired.

Visual observation

Prior to conducting the assessment, a thorough review of the existing construction documents, if available, should be conducted. Through the document review, the practitioner is able to gain an understanding of potential restraint conditions, a general sense of the intended behavior, as well as the apparent design capacity. Each crack should be located on an accurate sketch of the structure or element. Included on the sketch should be the location, length, and size of each crack. Cracks should be measured to within 0.025mm (0.001-inch) using a crack comparator. Crack movements can be documented over time to gain an understanding of the environmental factors that may be influencing the cracks. A simple crack monitor, such as the plastic device shown in Fig. 3.8, can be affixed with epoxy to either side of the crack, allowing the practitioner to measure propagation and rotation



3.8 Crack monitor adhered to stone. (Courtesy of WJE)

3.9 Potentiometer in use to measure the crack movement over a year. (Courtesy of WJE)

of the crack. Potentiometers wired to a data collector can be used for more sophisticated monitoring over a longer period of time. Figure 3.9 shows a potentiometer in use.

Non-destructive testing

Possibilities for non-destructive testing are highly dependent on the type and frequency of the crack or delamination. Often mechanical sounding over large areas (a bridge deck would be one example) can be executed with a chain drag. As the chains move over the surface, a very perceptible change in sound is emitted. Interior delaminations can also be detected by pulse velocity, ground-penetrating radar (GPR), and impact echo. Pulse velocity, as performed by ASTM C597 (ASTM, 2002) uses a mechanical pulse transmitted to one face of the concrete and received at the opposite face. The time taken for the pulse to pass through the member is measured electronically. If the distance is known, the pulse velocity can be calculated. Changes in the measured pulse velocity can occur if an internal discontinuity results in an increased path length.

Alternatively, flaws can be detected by impact-echo testing. This method involves introducing mechanical energy, in the form of a brief impact, to the structure. When the surface of the concrete is impacted, a transducer acoustically mounted on the same surface receives the wave energy reflections from boundaries or discontinuities within the member. With knowledge of the propagation velocity, the amplitude spectrum can be evaluated to compare relative velocities between test points and determine the location of boundaries or discontinuities within the concrete member using sound waves generated from an impact on the surface of the concrete. The sound wave reflects through the member, echoes from a defect or other surface, and is then received by a displacement transducer located near the impact point. The device provides the frequency content of the time domain waveform for analysis; the frequency associated with the resonance appears as the peak amplitude. This amplitude is associated with the thickness of the concrete section. An internal flaw can be detected by the flaw's amplitude peak correlated with the speed of the sound wave at the flaw.

Both radiography and radar can be employed to detect flaws. With radiography, both X-ray and gamma-ray equipment is used. Gamma-ray methods are the less expensive of the two. Both methods are better suited for cracks that are anticipated to be parallel to the direction of radiation. Care needs to be taken when using these methods at an operational facility, and the testing may need to be scheduled during times where the structure is unoccupied.

GPR is often used to detect existing reinforcement and discontinuities in reinforced concrete. GPR uses electromagnetic waves to measure these elements. This method can cover a large area to detect cracks, voids, and to measure thickness. However, interpretation of the data can be complex and should be undertaken by an experienced technician, as the interpretation of electronic frequencies and dielectric constants of the concrete can affect the results. As with visual assessment, non-destructive testing should be verified and correlated with physical observation.

Corrosion assessment testing

One of the more significant durability issues with existing reinforced concrete structures is corrosion of the reinforcement. Corrosion of the reinforcement creates a significant change in volume of the reinforcement that can result in cracking and delaminations within the concrete structure or element. While physical sounding may reveal some of the delaminations, half-cell potential testing can determine the potential for corrosion over a large area. The most commonly used half-cell is the copper–copper sulfate electrode (ASTM C876, 1999). Figure 3.10 illustrates the use of this technique.

Destructive testing (coring)

To confirm results from visual observations and non-destructive testing, the practitioner often selects representative cores from the distressed or cracked areas. A breadth of information can be obtained from the concrete core. Examination may yield the width, depth, and age of the crack. The natural carbonation of the exposed surface of the crack can be determined in the laboratory by application of the indicator phenolphthalein. When

3.10 Copper–copper sulfate half-cell in use; note the sponge to keep the concrete moist and complete the circuit. (Courtesy of WJE) exposed to phenolphthalein, uncarbonated concrete surfaces will turn purple. Experienced concrete petrographers can approximate the age of a crack from the measured depth of the color change from the crack surface. Petrographic examinations (ASTM C856, 2004) of cracked concrete can identify material causes of cracking and distresses such as D-cracking, shrinkage, freezing damage, DEF, ASR, fire-related distress, and corrosion. Petrography can also indentify related factors such as w/cm, paste volume, aggregate soundness, type, and gradation, cement type, air entrainment, and distress mechanisms that occur at a microscopic level.

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4
Condition assessment of concrete structures

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Abstract: An overview of the principles and practices of condition assessment of existing concrete structures is presented as well as the progression of assessment work prior to the implementation of repairs from a consulting engineer's perspective. Non-destructive testing techniques, materials engineering/testing techniques, selection of samples, and development of evaluation options are also considered.

Key words: condition assessment of concrete structures, non-destructive evaluation, damage assessment, load testing, concrete repairs.

4.1 Introduction

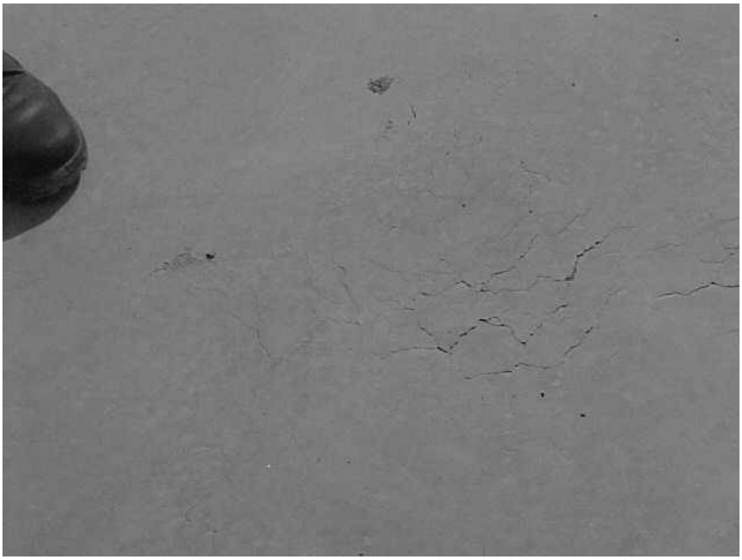
This chapter provides an overview of the principles and practices of condition assessment of existing concrete structures and the progression of assessment work prior to implementation of repairs from the consulting engineer's perspective. The pertinent non-destructive testing and materials engineering/testing techniques, selection of samples, development of evaluation options, as they apply to both existing structures and new construction problem resolution, will be discussed.

4.2 Evaluation of concrete structures – overview

Evaluation of concrete structures may be necessary under many circumstances. Figures 4.1–4.5 show various instances at which a concrete assessment may be necessary. Although individual circumstances may vary, evaluation of concrete structures is generally initiated related to instances such as:

- New construction/concrete placements
    1. problems related to fresh concrete during placement (high heat, flash/false setting, frost/wind exposure during placement, plastic shrinkage cracks, segregation);
    2. occurrence of construction defects (crazing, finished surface problems, delaminations, poor consolidation, slab curling);
    3. low strength results or batching errors;
    4. occurrence of a failure (upon removal of formwork or initiation of post-tensioning).

4.1 Delamination in a concrete core removed from a slab.



4.2 Crazing of the finished surface upon exposure of fresh concrete to wind.

- Structures in service
  1. exposure of concrete to a damaging occurrence (such as a fire, accident, spill or a blast);
  2. exposure of concrete to deteriorating mechanisms (such as ASR, de-icers, chloride induced corrosion, sulfate exposure);

4.3 Tendons displaced during post-tensioning operations.

4.4 Reinforcing steel exhibiting corrosion leading to dislodged cover.

3. poor performance or observed distress (such as cracking, deflections, or disintegration);
4. occurrence of a failure in service (partial failure or collapse);
5. modification in current service use or increase in service loads.

4.5 Impact and fire damage to precast concrete structure.

4.3 Methodologies to evaluate concrete structures

Methodologies in assessment of concrete generally vary with the type of damage mechanism or cause that is being investigated. Generally methodologies, whether *in-situ* or laboratory testing, aim to identify the cause of the observed distress or poor performance. The applied method also aims to identify the extent of the problem both in terms of location and spread as well as severity and structural implications.

A sampling plan needs to be developed, encompassing the potentially affected areas as well as an area from an unaffected portion of the structure, preferably of the same concrete placement or truck. The testing program can consist of non-destructive tests and/or *in-situ* minimally invasive tests as well as removal of core samples for laboratory testing from the identified areas.

The specifics of the testing program vary with the mechanism or cause in question. For example, in an assessment involving corrosion of reinforcing steel, the evaluation program would likely involve sounding or chain dragging for identification of regions with delaminations in the concrete cover, *in-situ* half-cell potential testing, and testing of concrete samples for pH/carbonation and water-soluble chloride content. A low-strength evaluation may involve *in-situ* Windsor probes or pulse velocity measurements and subsequent removal of cores for determination of strengths. A slab curling investigation, on the other hand, may require review of construction records in comparison with temperature/humidity records and a survey using a floor flatness device commonly used for ASTM E1155 Standard Test Method for Determining FF Floor Flatness and FL Floor Levelness Numbers.

Tables 4.1–4.3 are excerpts from ACI Committee 228.2R-98 and provide a list of techniques and applications. Tables 5-1 and 5-2 in ACI 364.1R-04

Table 4.1 Summary of non-destructive testing methods – Table 2.1 from ACI 228 2R-98

Section no.	Method and principle	Applications
2.1	**Visual inspection** – Observe, classify and document the appearance of distress on exposed surfaces of the structure.	Map patterns of distress such as cracking, spalling, scaling, erosion, or construction defects.
2.2.1	**Ultrasonic pulse velocity** – Measure the travel time of a pulse of ultrasonic waves over a known path length.	Determine the relative *condition* of concrete based on measured pulse velocity.
2.2.2	**Ultrasonic-echo** – Transducer emits short pulse of ultrasonic waves which is reflected by opposite side of member or an internal defect; arrival of reflected pulse is recorded by an adjacent receiver, and round-trip travel time is determined.	Locate delaminations and voids in relatively thin elements. Primarily a research tool.
2.2.3	**Impact-echo** – Receiver adjacent to impact point monitors arrival of stress waves as they undergo multiple reflections between surface and opposite side of plate-like member or from internal defects. Frequency analysis permits determination of distance to reflector if wave speed is known.	Locate a variety of defects within concrete elements such as delaminations, voids, honeycombing, or measure element thickness.
2.2.4	**Spectral analysis of surface waves** – Impact is used to generate a surface wave and two receivers monitor the surface motion; signal analysis allows determination of wave speed as a function of wavelength; inversion process determines elastic constants of layers.	Determine the stiffness profile of a pavement system. Also used to determine depth of deteriorated concrete.
2.3.1	**Sonic-echo** – Hammer impact on surface and a receiver monitors reflected stress wave. Time-domain analysis used to determine travel time.	Determine the length of deep foundations (piles and piers); determine the location of cracks or constrictions (neck-in)
2.3.2	**Impulse-response** – Test is similar to sonic-echo method except that signal processing involves frequency-domain analysis of the received signal and the impact force history.	Determine the length of deep foundations (piles and piers), location of cracks or constrictions (neck in). Provides information on the low-strain dynamic stiffness of the shaft/soil system.
2.3.3	**Impedance logging** – Test is similar to sonic-echo or impulse-response, but the use of more complex signal	Determine the approximate 2-dimensional shape of the deep foundation.

Table 4.1 Cont'd

Section no.	Method and principle	Applications
	analyses (time and frequency domains) allows reconstructing the approximate shape of the deep foundation.	
2.3.4	**Crosshole sonic logging** – Analogous to the ultrasonic pulse velocity test, but transducers are positioned within tubes cast into the deep foundation or holes drilled after construction.	Determine the location of low-quality concrete along the length of the shaft and between transducers. With drilled holes permits direct determination of shaft length.
2.3.5	**Parallel seismic** – Receiver is placed in hole adjacent to the foundation. Foundation is struck with a hammer and signal from receiver is recorded. Test is repeated with receiver at increasing depth.	Determine the foundation depth and determine whether it is of uniform quality.
2.4.2	**Direct transmission radiometry** – Measure the intensity of high energy electromagnetic radiation after passing through concrete.	Determine in-place density of fresh or hardened concrete. Locate reinforcing steel or voids.
2.4.3	**Backscatter radiometry** – Measure the intensity of high-energy electromagnetic radiation that is backscattered (reflected) by the near surface region of a concrete member.	Determine in-place density of fresh or hardened concrete.
2.4.4	**Radiography** – The intensity of high-energy electromagnetic radiation which passes through a member is recorded on photographic film. Locate reinforcing and prestressing steel, conduits, pipes, voids, and honeycombing.	
2.4.5	**Gamma-gamma logging** – See direct transmission and backscatter radiometry.	Locate regions of low density along length of foundation.
2.5.1	**Covermeter** – A low frequency alternating magnetic field is applied on the surface of the structure; the presence of embedded reinforcement alters this field, and measurement of this change provides information on the reinforcement.	Locate embedded steel reinforcement, measure depth of cover, and estimate diameter of reinforcement.
2.5.2	**Half-cell potential** – Measure the potential difference (voltage)	Identify region or regions in a reinforced concrete structure where

Table 4.1 Cont'd

Section no.	Method and principle	Applications
	between the steel reinforcement and a standard reference electrode; the measured voltage provides an indication of the likelihood that corrosion is occurring in the reinforcement.	there is a high probability that corrosion is occurring at the time of the measurement.
2.5.3	**Polarization methods** – Measure the current required to change by a fixed amount the potential difference between the reinforcement and a standard reference electrode; the measured current and voltage allow determination of the polarization resistance, which is related to the rate of corrosion.	Determine the instantaneous corrosion rate of the reinforcement located below the test point.
2.6	**Penetrability methods** – Measure the flow of a fluid (air or water) into concrete under prescribed test conditions; the flow rate depends on the penetrability characteristics of the concrete.	Compare alternative concrete mixtures. Primarily research tools, but have the potential to be used for assessing adequacy of curing process.
2.7	**Infrared thermography** – The presence of flaws within the concrete affects the heat conduction properties of the concrete and the presence of defects are indicated by differences in surface temperatures when the test object is exposed to correct ambient conditions.	Locate delaminations in pavements and bridge decks. Also widely used for detecting moist insulation in buildings.
2.8	**Radar** – Analogous to the ultrasonic-echo methods except that electromagnetic waves are used instead of stress waves. Interface between materials with different dielectric properties results in reflection of a portion of incident electromagnetic pulse.	Locate metal embedments, voids beneath pavements, and regions of high moisture contents; determine thickness of members.

(Tables 4.4 and 4.5) also provide a good overview discussion of techniques. A similar compilation in tabular form also exists in ASCE Standard ASCE 11.

4.4 Stages of condition assessment

Once a problem is identified with a concrete placement or a concrete member, particularly low-strength issues, one of the first actions taken is *in-situ* testing

Table 4.2 Non-destructive test methods for determining material properties of hardened concrete in existing construction – Table 3.1 from ACI 228 2R-98

	Possible methods		
Property	Primary	Secondary	Comment
Compressive strength	Cores for compression testing (ASTM C 42/C 42M and C 39/C 39M)	Penetration resistance (ASTM C 803/C 803M): Pull-out testing (post-installed)	Strength of in-place concrete; comparison of strength in different locations
Relative compressive strength	Rebound number (ASTM C 805); Ultrasonic pulse velocity (UPV) (ASTM C 597)		Rebound number influenced by near surface properties; UPV gives 'average' result through the thickness
Tensile strength	Splitting tensile strength of core (ASTM C 496)	In-place pull-off test (ACI 503R; BS 1881: Part 207)	Assess tensile strength of concrete
Density	Relative density (specific gravity) of samples (ASTM C 642)	Nuclear gage	
Moisture content	Moisture meters	Nuclear gage	
Static modulus of elasticity	Compression test of cores (ASTM C 469)		
Dynamic modulus of elasticity	Resonant frequency testing of sawed specimens (ASTM C 215)	Ultrasonic pulse velocity (ASTM C 597); impact-echo; spectral analysis of surface waves (SASW)	Requires knowledge of density and Poisson's ratio (except C 215); dynamic elastic modulus is typically greater than the static elastic modulus
Shrinkage/ expansion	Length change of drilled or sawed specimens (ASTM C 341)		Measure of incremental potential length change
Resistance to chloride penetration	90-day ponding test (AASHTO T 259)	Electrical indication of concrete's ability to resist chloride ion penetration (ASTM C 1202)	Establish relative susceptibility of concrete to chloride ion intrusion; assess effectiveness of chemical sealers, membranes, and overlays

Table 4.2 Cont'd

Property	Possible methods: Primary	Possible methods: Secondary	Comment
Air content; cement content; and aggregate properties (scaling; alkali–silica reactivity; freeze/thaw susceptibility)	Petrographic examination of concrete samples removed from structure (ASTM C 856 and ASTM C 457); cement content (ASTM C 1084)	Petrographic examination of aggregates (ASTM C 294, C 295)	Assist in determination of cause(s) of distress; degree of damage; quality of concrete when originally cast and current
Alkali-silica reactivity (ASR)	Cornell/SHRP rapid test (SHRP-C-315)		Establish in field if observed deterioration is due to ASR
Carbonation, pH	Phenolphthalein (qualitative indication); pH meter	Other pH indicators (e.g., litmus paper)	Assess corrosion protection value of concrete with depth and susceptibility of steel reinforcement to corrosion; depth of carbonation
Fire damage	Petrography; rebound number (ASTM C 805)	SASW; UPV; impact-echo; impulse-response	Rebound number permits demarcation of damaged surface
Freezing and thawing damage	Petrography	SASW; impulse-response	
Chloride ion content	Acid-soluble (ASTM C 1152C 1152M) and water-soluble (ASTM C 1218/C 1218M)	Specific ion probe (SHRP-S-328)	Chloride ingress increases susceptibility of steel reinforcement to corrosion
Air permeability	SHRP surface airflow method (SHRP-S-329)		Measures in-place permeability index of the near-surface concrete (15 mm)
Electrical resistance of concrete	AC resistance using 4-probe resistance meter	SHRP surface resistance test (SHRP-S-327)	AC resistance useful for evaluating effectiveness of admixtures and cementitious additions; SHRP method useful for evaluating effectiveness of sealers

Table 4.3 Non-destructive test methods to determine structural properties and assess conditions of concrete – Table 3.2 from ACI 228 2R-98

Property/ condition	Method		Comment
	Primary	Secondary	
Reinforcement location	Covermeter; ground-penetrating radar (GPR) (ASTM D 4748)	X-ray and γ-ray radiography	Steel location and distribution; concrete cover
Concrete component thickness	Impact-echo (I-E) (ASTM C 1383); GPR (ASTM D 4748)	Intrusive probing	Verify thickness of concrete; provide more certainty in structural capacity calculations; I-E requires knowledge of wave speed and GPR of dielectric constant
Steel area reduction	Ultrasonic thickness gage (requires direct contact with steel)	Intrusive probing; radiography	Observe and measure rust and area reduction in steel; observe corrosion of embedded post-tensioning components; verify location and extent of deterioration; provide more certainty in structural capacity calculations.
Local or global strength and behavior	Load test, deflection or strain measurements	Acceleration, strain, and displacement measurements	Ascertain acceptability without repair or strengthening; determine accurate load rating
Corrosion potentials	Half-cell potential (ASTM C 876)		Identification of location of active reinforcement corrosio
Corrosion rate	Linear polarization (SHRP-S-324 and S-330)		Corrosion rate of embedded steel; rate influenced by environmental conditions
Location of delaminations, voids, and other hidden defects	Impact-echo; Infrared thermography (ASTM D 4788); Impulse-response; Radiography; GPR	Sounding (ASTM D 4580); pulse echo; SASW; intrusive drilling and borescope	Assessment of reduced structural properties; extent and location of internal damage and defects; sounding limited to shallow delaminations

Table 4.4 Test methods to evaluate hardened concrete in existing structures – Table 5.1 from ACI 364.1R-04

Property	Possible test methods		Comment
	Primary	Secondary	
Compressive strength	Cores for compression testing (ASTM C 42/C 42M and C 39/C 39M: ACI 214.3R)	Penetration resistance (ASTM C 803/ C 803M); pullout testing (drilled in)	Strength of in-place concrete; comprison of strength in different locations; drilled in pullout test not standardized by ASTM
Relative compressive strength	Rebound number (ASTM C 805); ultrasonic pulse velocity (UPV) (ASTM C 597)	–	Rebound number influenced by near-surface properties; UPV gives average result through the thickness
Tensile strength	Splitting tensile strength of cores (ASTM C 496/C 496M)	–	Determine approximate tensile strength of concrete
Flexural strength	Break-off text (Carino and Malbotra 2004)	Sampling and testing of sawed beams (ASTM C 42/C 42M)	Limitations posed by aggregate size and nonhomogeniety
Density	Specific gravity of samples (ASTM C 642)	–	Special technique requiring calibration curve
Moisture content	Moisture meters (ASTM D 3017)	–	–
Static modulus of elasticity	Compression test of cores (ASTM C 469)	–	–
Dynamic modulus of elasticity	Resonant frequency testing of sawed specimens (ASTM C 215)	Ultrasonic pulse velocity (ASTM C 597); impact-echo; spectral analysis of surface waves (SASW)	Requires knowledge of density and Poisson's ratio (except ASTM C215): dynamic modulus is typically greater than static elastic modulus
Shrinkage/expansion	Length change of drilled or sawed specimens (ASTM C 341/C 341M)	–	Measure of incremental potential length change

Resistance to chloride penetration	90-day pending test (AASHTO T 259)	Electrical indication of concrete's ability to resist chloride-ion penetration (ASTM C 1202)	Establish relative susceptibility of concrete to chloride-ion intrusion; determine effectiveness of chemical sealers, membranes, and overlays
Air content; cement content; and aggregate properties (alkalisilica reactivity; freezing-and-thawing susceptibility)	Petrographic examination of concrete samples removed from structure (ASTM C 856 and 457); cement content (ASTM C 1084)	–	Assist in determination of cause(s) of distress; degree of damage; quality of concrete when originally cast and current
Alkali-silica reactivity (ASR)	Petrographic examination of concrete samples removed from structure (ASTM C 856 and C 457)	Comell/SHRP rapid test (SHRP C-315)	Establish in field if observed deteroration is due to ASR
Carbonation, pH	Phenolphthalein (qualitative indication); pH meter	Petrographic examination, pH indicators (for example, litimus paper)	Assess corrosion protection value of concrete with depth and susceptibility of steel reinforcement to corrosion; depth of carbonation
Fire damage	Petrographic examination of cores (ASTM C 856), compressive strength tests (ASTM C 39/C 39M), split tensile strength tests (ASTM C 496/C 496M)	SASW; UPV; impact-echo; impulse-response	Rebound number permits demarcation of damaged surface
Freezing-and-thawing damage	Petrographic examination of cores (ASTM C 856), compressive strength tests (ASTM C 39/C 39M), split tensile strength tests (ASTM C 496/C 496M)	SASW; UPV; impact echo; impulse-response	Freezing and thawing can causes internal cracking in concrete; spilt tensile strength is useful in determining the tensile strength capacity of concrete
Chloride-ion content	Acid-soluble (ASTM C 1152/C 1152M) and water-soluble (ASTM C 12 1218/C 1218M)	Specific ion probe (SHRP S-328)	Chloride ingress increases susceptibility of steel reinforcement to corrosion
Air permeability	SHRP surface airflow method (SHRP S-329) Figg Technique		Measures in-place permeability index of the near-surface concrete 0.60 in. (15 mm); results vary

Table 4.4 Cont'd

Property	Possible test method		Comment
	Primary	Secondary	
			depending on the moisture content of concrete
Electrical resistance of concrete	AC resistance using four-probe resistance meter	SHRP surface resistance test (SHRP S-327)	AC resistance useful for evaluating effectiveness of admixtures and cementitious additions; SHRP method useful for evaluating effectiveness of sealers
Internal voids, delaminations	Acoustic impact (ASTM D 4580), impulse response impact-echo, infrared thermography, UPV, radar	Gamma radiography	Success dependant on text procedure, equipment, and personnel, as well as void geometry

Table 4.5 Test methods to determine structural properties and determine condition of reinforcing steel – Table 5.2 from ACI 364.1R-04

Property/condition	Possible test method		
	Primary	Secondary	Comment
Reinforcement location	Expose reinforcement for measurement, pachometer; ground-penetrating radar (GPR) (ASTM D 4748)	X-ray and γ-ray radiography	Steel location and distribution; concrete cover
Reinforcement cross-sectional area reduction	Expose reinforcement and measure diameter; ultrasonic thickness gauge (requires direct contact with steel)	Intrusive probing; radiography	Observe and measure rust and reduction in steel; observe corrosion of embedded post-tensioning components; provide more certainty in structural capacity calculations
Corrosion potentials	Half-cell potential (ASTM C 876)	–	Identification of active reinforcement corrosion
Corrosion rate	Linear polarization (SHRP S-324 and S-330)	Electrochemical impedance	Corrosion rate of embedded steel; rate influenced by environmental conditions
Tensile testing	Tension testing of metallic materials (ASTM A370 and E8)	–	Tension testing of removed samples
Chemical analysis	Lab test on sample (ASTM A 751)	–	Needed for determining weldability or to confirm bar grade
Protective coating thickness	Remaining coating thickness on exposed surfaces (ASTM E 376, G 12, G 14, G 20)	– –	Requires calibrated test equipment

Note: Other mechanical property testing of metal components, such as hardness and impact, are described in ASTM A 370 and E 8 and their references.

of the subject area using the quick and practical field testing techniques that aim to estimate strength such as a Swiss hammer or a Windsor Probe™. If these tests identify or confirm the existence of a problem, the issue generally escalates towards removal of cores and involvement of consulting engineers. The consulting engineer's role and duties can include the following:

- consultant to perform engineering assessment and provide opinion of cause and extent/severity and implications;
- testing support/sub-consultant to another engineer serving as the consultant;
- preparation of assessment/testing report;
- development of repair recommendations/options which may range from cosmetic repairs to complete removal and replacement;
- design of repair option recommended/selected by the client;
- construction oversight during implementation of repairs;
- deposition/litigation support for the client or client's legal counsel if the issue goes to litigation.

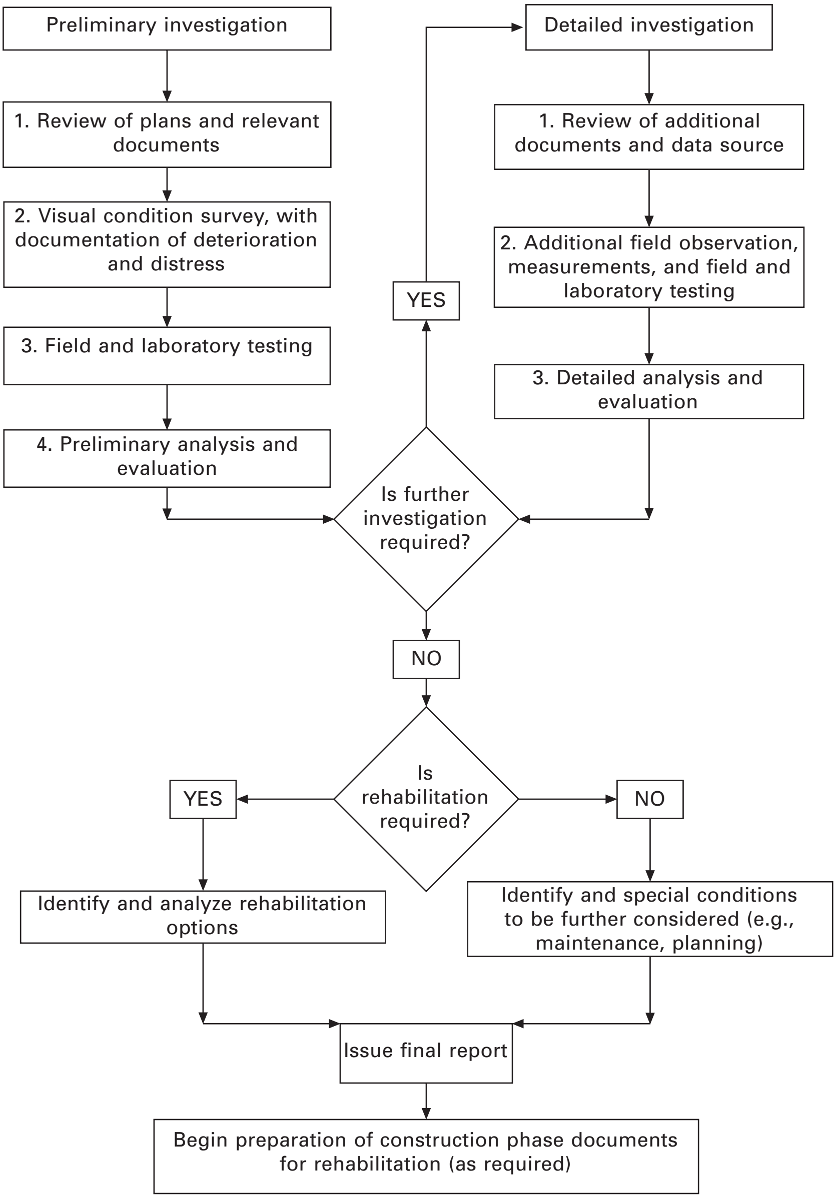
Figure 4.6 provides a typical flowchart for evaluation methodology (published as Figure 1 in ACI 364.1R-04). A similar flowchart representing a slightly different algorithm is provided in ASCE Standard ASCE11. In general, a typical engineering assessment associated with concrete structures may involve one or more of the tasks discussed below.

Document review

Document review generally involves review of drawings with particular attention to the specific details pertinent to the subject issue, material specifications, submittals and the approval process on the submittals, mix designs and strength records submitted by the supplier on the subject concrete mixture used, constituents of the concrete batch, truck tickets and placement records to identify locations of particular truckloads in the structure, technician logs performing quality assurance testing such as mix temperature, slump, and air content on the subject truck or placement.

Interview with involved parties

This task will generally involve interviews with or written requests for information from the designer, the contractor's field personnel, the concrete supplier, and the concrete technician performing quality assurance testing. In the case of an existing structure in service, this task would involve personnel familiar with the structure's history, observed distresses, past repairs, and maintenance performed related to the issue in hand.



4.6 Flowchart for evaluation methodology – Figure 1 from ACI 364. IR-04.

Visual survey and photographic documentation

The purpose of this task is generally to become familiar with the structure, observe the extent and associate patterns to the observed distress or problem, in relation to column lines, expansion joints, exposures, concrete placements, or trucks, etc. Observations and photographic documentation are later used in reporting as well as development of a sampling plan and a testing program.

In-situ non-destructive testing

As explained above, simple or brief *in-situ* testing can occur as a preliminary phase, as one of the first actions later leading to justification of the involvement of consultants and expanding the testing program to above and beyond the quick field tests for indication of strength. The *in-situ* non-destructive testing phase is generally developed by the consulting engineer, as a distinct phase of the consulting services. Scanning the area in question using non-destructive testing techniques may help identify the extent of the issue from a location standpoint, help in fine-tuning the sampling plan based on indicated strength ranges or non-destructive evaluation (NDE) indices, or locate any reinforcing steel to aid in coring operations. These tasks would help in development of the sampling plan and would likely involve non-destructive testing tasks such as Windsor Probe™ indicated strengths (later to be correlated to core strengths), Stress Wave based methods such as impact-echo or ultrasonic pulse velocity, ground-penetrating radar scans, or radiographic exposures, deflection survey using a survey rod and surveyor level or just simply sounding/dragging a chain. Scanning the area using these techniques also helps control the cost of the engineering evaluation as it eliminates the need to perform extensive coring.

Development of a sampling plan

A sampling plan is developed based on the visual survey and results of any non-destructive tests performed. The sampling plan would encompass the potentially affected areas as well as an area from an unaffected portion of the structure, preferably of the same concrete placement or truck, to serve as baseline or control. The core samples in the affected area generally include points of interest based on NDE findings such as areas of low and high indicated strengths (based on Windsor Probes™, for example). This not only enables associating true strength numbers to both ends of the spectrum of indicated strengths, it also establishes a correlation curve which can be used to make strength deductions from indicated strengths if Windsor Probes™ are used in a system-wide manner to scan the affected area. Other considerations include budgetary limitations and removal of cores from structurally non-critical/visually inconspicuous locations.

ASTM E 122 provides a basis for determining the number of samples necessary to achieve a certain degree of confidence based on variance and allowable error. The ACI 437-91R and ACI 437R-03 committee reports provide discussions specifically pertinent to sampling of concrete structures based on the methodology of ASTM E 122. ASTM E 105-04 and ASTM C 823-00 also provide information regarding sampling and sample sizes.

Development of a testing program

The testing program is generally developed in coordination with the sampling plan. The damaging mechanism or the cause in question dictate the test methods selected to be performed as discussed in Section 4.3.

In addition, sample size requirements in the specific ASTM test method selected, whether length-to-diameter ratio of a core, or the number of cores for a valid strength determination, or grams of dust sample necessary for a chloride determination, all come into play during the development of the testing program, determination of the tests to use, number of core locations, selection of core barrel diameter, etc.

Evaluation of the test results and engineering analysis

Once the test results are obtained, the findings are used in an engineering analysis to evaluate the implications. The engineering analysis may involve a structural analysis from a strength or structural integrity standpoint in light of the compromise/deficiency based on the test findings. Depending on the nature of the deficiency or the compromised property, the engineering analysis may also involve a review from a durability standpoint. Engineering analysis can investigate not only the extent and severity of the subject problem from a structural or durability angle, it can also evaluate the findings of the test program and establish a cause and effect relationship as to what may have caused the subject issue.

Development of repair options

The findings of the engineering analysis generally form the basis of development of repair options. Depending on the severity of the problem, the repair options may involve cosmetic/minor repairs, waterproofing repairs if durability is compromised (since deteriorating mechanisms generally involve water intrusion), structural strengthening scenarios through external steel or post-tensioning, or may include complete teardown and rebuild. Items of consideration are, naturally, the cost of repairs, the time to implement the repairs, the downtime for the facility/construction activities, the expected life/longevity of the repairs, and the need to restore the structure to what the owner paid for from a strength/durability standpoint.

Design of repairs

Design of repairs typically involves development of a repair design for one of the options developed during the previous phase and selected based on the recommendation of the consultants. This phase may also involve construction administration and oversight of the repairs as the designer/engineer of record for the repairs.

Litigation support

Deposition/litigation support for the client or client's legal counsel may be necessary if the issue goes to litigation. Depending on the specifics of the project, this phase may never come into play or may occur after the engineering assessment.

4.5 Evaluation techniques and test methodologies

4.5.1 Preliminary survey techniques

Visual survey and photographic documentation

A visual survey and photographic documentation are key elements in every condition assessment. The information gained from the visual evaluation and photographic documentation often shapes the remainder of the investigation in terms of development of the testing program and sampling plan. This task routinely consists of a walkthrough, observing the extent and associating patterns to the observed distress or problem, in relation to column lines, expansion joints, exposures, concrete placements, or trucks, etc. It can also be accompanied by some of the sounding or Swiss hammer techniques or basic deflection measurements with survey equipment. ACI Committee Report 201.1R provides a detailed methodology of visual survey of concrete along with photographs and discussions of various deterioration mechanisms and distresses.

Sounding and chain dragging surveys

Sounding or chain drag surveys are field evaluation techniques generally associated with an effort to identify delaminations, whether the delaminations are a result of finishing issues, dislocation of concrete cover as a result of corrosion of steel or delaminations of floor coatings/toppings. Sounding surveys provide good results in qualitative identification. Minimal training is necessary to perform effective sounding surveys. A general level of familiarity is necessary to evaluate the contrasts in sound associated with sound substrates versus a delaminated section. Chain drag surveys are suitable for flatwork and pavement or bridge decks (ASTM D 4580-03) while sounding can be performed on any surface. Figure 4.7 shows a sounding survey of precast double connections for corrosion damage, while Fig. 4.8 shows a chain drag apparatus equipped with various length chains used on an industrial floor coating exhibiting bulging.

4.7 Sounding survey of precast double-tee connections for dislodged cover due to corrosion.

4.8 Chain drag apparatus used on a bulging industrial floor coating.

Swiss (Schmidt) hammer

The Swiss hammer is a spring-operated hammer assembly used in estimating the near-surface strength of concrete members. The body of the gauge includes a spring-driven hammer which, upon impact, rebounds and moves a slide indicator making a record of the rebound distance. Figure 4.9 shows a commercially available Swiss hammer. The rebound number ranges from 0 to 100 and may be correlated to an apparent near-surface strength. Various correlation charts are provided that take the orientation of the hammer into account. The test method is explained in greater detail in ASTM C 805.

The method is a good first indicator, but is subject to variables such as presence of a surface hardener, machine/hard troweled finishes, finished versus formed surface, presence of delaminations, carbonation of the surface, and moisture content, just to name a few. In summary, the Swiss hammer is a good first indicator of near-surface concrete. It is ideal as an initial scan tool; however, an engineering assessment rarely relies on Swiss hammer values alone and generally involves verification of *in-situ* strength by way of core strengths.

Windsor Probe™

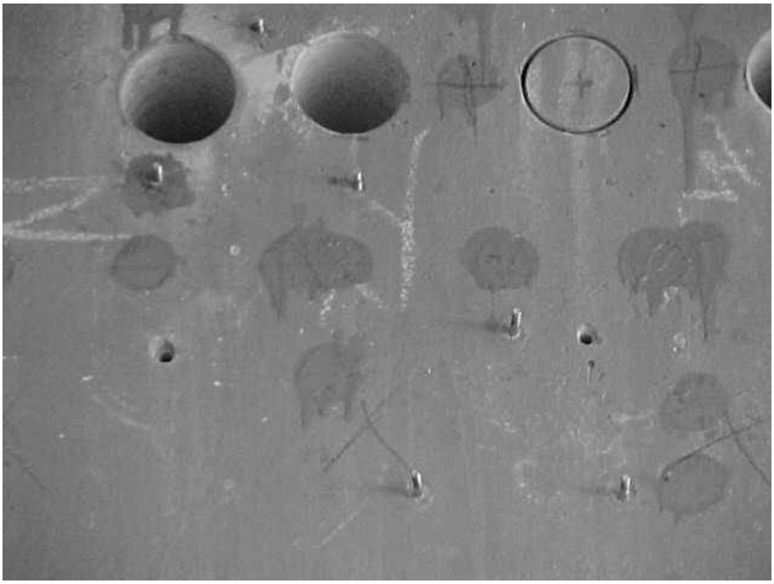
The Windsor Probe™ penetration resistance is based on measuring the penetration of probes driven into the concrete using a gunpowder actuated charge. It utilizes a special handgun to drive the probes into the concrete.

4.9 Swiss hammer.

Different classifications such as low-power or standard power shots are used based on the expected *in-situ* strength. The depth of probe penetration is an indicator of near-surface strength. The probes are driven using templates of three (Fig. 4.10) and the reading is averaged, or determinations can also be made based on individual shots if geometry/access does not permit the pattern of three probes. The penetration of probes is measured using a micrometer and then the indicated strengths are determined using correlation charts that take the aggregate hardness into account. The test method is explained in greater detail in ASTM C 803.

The method is a good first indicator but subject to variables similar to the Swiss hammer. It is ideal as an initial scan tool; however, an engineering assessment rarely relies on indicated strengths by Windsor Probes alone. Generally these techniques are cheaper to perform compared to removing cores from each individual area. Therefore, they are performed in a system-wide scan manner and the results are analyzed during development of the sampling plan. Core sample locations are selected at areas with the low and high probe penetrations (Fig. 4.11). This not only enables associating true strength numbers to both ends of the spectrum of indicated strengths, it also establishes a correlation curve which can be used to make strength deductions from other indicated strengths if Windsor Probes are used in a system-wide manner to scan the affected area.

4.10 Windsor Probe™ determination on a beam using the three-probe template.



4.11 Windsor Probe™ determination with deep penetrations later correlated to strengths.

Pull-out strength

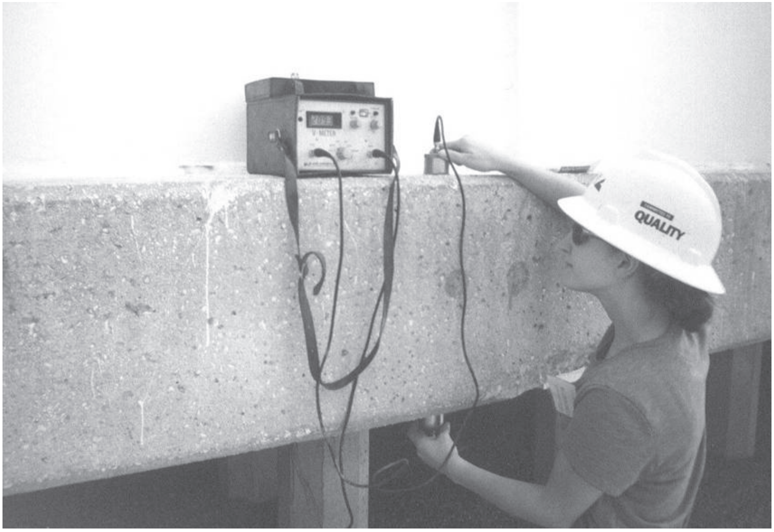
Pull-out strength (ASTM C 900) is also frequently listed in the literature as a field assessment technique. This method involves determination of the force required to pull an embedded metal insert with an increased head size that will cause a conical failure. The technique requires preliminary setup for the determination of pull-out strength and is therefore best suited for new construction and not after-the-fact condition assessments. An alternative procedure is listed for installation into existing structures.

4.5.2 Stress wave based methods

Ultrasonic pulse velocity

The ultrasonic pulse velocity method is based on measurement of the travel time of an ultrasonic wave through concrete over a known path length. The technique has been in use for several decades and has proven to be a versatile method in concrete assessment studies. The commercially available systems consist of two piezoelectric transducers and electronic circuitry to determine the pulse travel time between the transducers (Fig. 4.12). A couplant is necessary between transducers and concrete surfaces.

The technique is described in greater detail in ASTM C 597. The compression



4.12 Ultrasonic pulse velocity determination through an elevated concrete pedestal.

or P-wave speed (C_p) in an isotropic solid is related to Young's modulus, Poisson's ratio and density as shown in Equation [4.1]:

$$C_p = \sqrt{\frac{E(1-\nu)}{\rho(1+\nu)(1-2\nu)}} \qquad [4.1]$$

where C_p = compression wave (P-wave) speed, e = Young's modulus of elasticity, ν = Poisson's ratio and ρ = density.

Structural Condition Assessment by Ratay (2005) provides the following ranges for concrete:

Excellent:	Above 15 000 ft/s
Good:	12 000–15 000 ft/s
Marginal:	10 000–12 000 ft/s
Poor:	7000–10 000 ft/s
Very poor:	below 7000 ft/s

The technique has found a wide basis of application due to the nature of this relationship. Young's modulus is related to compressive strength; therefore both areas of deficient strength as well as strength gain can be evaluated. Similarly, because of the dependency on density, areas of poor consolidation or areas of high air content are easily detected using the ultrasonic pulse velocity technique. Numerous applications are listed in the literature including detection of fire damage, cryogenic fluid exposure, and detection of cracks and low-quality or damaged concrete and masonry (Dilek, 2005, 2006a, 2007b).

One drawback to this technique is the need to access both sides of the member of interest. Access to both sides of the member is not always possible, particularly with water-retaining structures, foundation walls, or slabs on grade. Furthermore, the nature and location of the defect are unidentified as the method only produces one index of quality, i.e., pulse velocity. The defect is expressed as an increase in pulse transit time and therefore a decrease in calculated pulse velocity. The method, therefore, does not distinguish between a shallow zone of very compromised concrete over a layer of sound concrete and a relatively lesser degree of compromise spread throughout the section of the member.

Impact echo

The limitations expressed above, such as the need to access both sides, can be eliminated using the 'echo' methods. The impact-echo method uses impactors of varying size to generate a stress pulse in lieu of a transducer, therefore providing a stress wave that can progress deeper into the member. The technique, in very general terms, has the impactor and the sensor on the same side of the member, and enables determination of reflections of the generated wave from either the end of the member or any kind of inherent discrete defect that will cause a reflection. The return signal is analyzed using a Fourier transform to determine the dominant frequencies (Figs 4.13 and 4.14). The signal transformed into the frequency domain enables the user to distinguish between reflections from the end of the member or intermediate defects, as the wave speed through the material is known and the frequency of the back wall/end reflection can be estimated given the size of the member. The technique and the equipment are explained in greater detail in ASTM C 1383.

The technique has found several applications such as determination of thickness of members and determination of flaws in members in addition to conventional uses of the ultrasonic pulse velocity method. The impact-echo literature contains work on signal attenuation as an indicator of damage and acoustic impedance or signal strength of the reflection from free boundary as an indicator of voids under slabs (Carino, 2001; Kesner *et al.*, 2004; Dilek and Leming, 2007).

Spectral analysis of surface waves

The spectral analysis of surface waves method has been used successfully in geotechnical and pavement applications. The method is also applicable to concrete structural members to assess changes in the elastic properties

4.13 Impact-echo measurements at a fire-damaged precast double-tee using the multiple-sized impactor and transducer assembly.

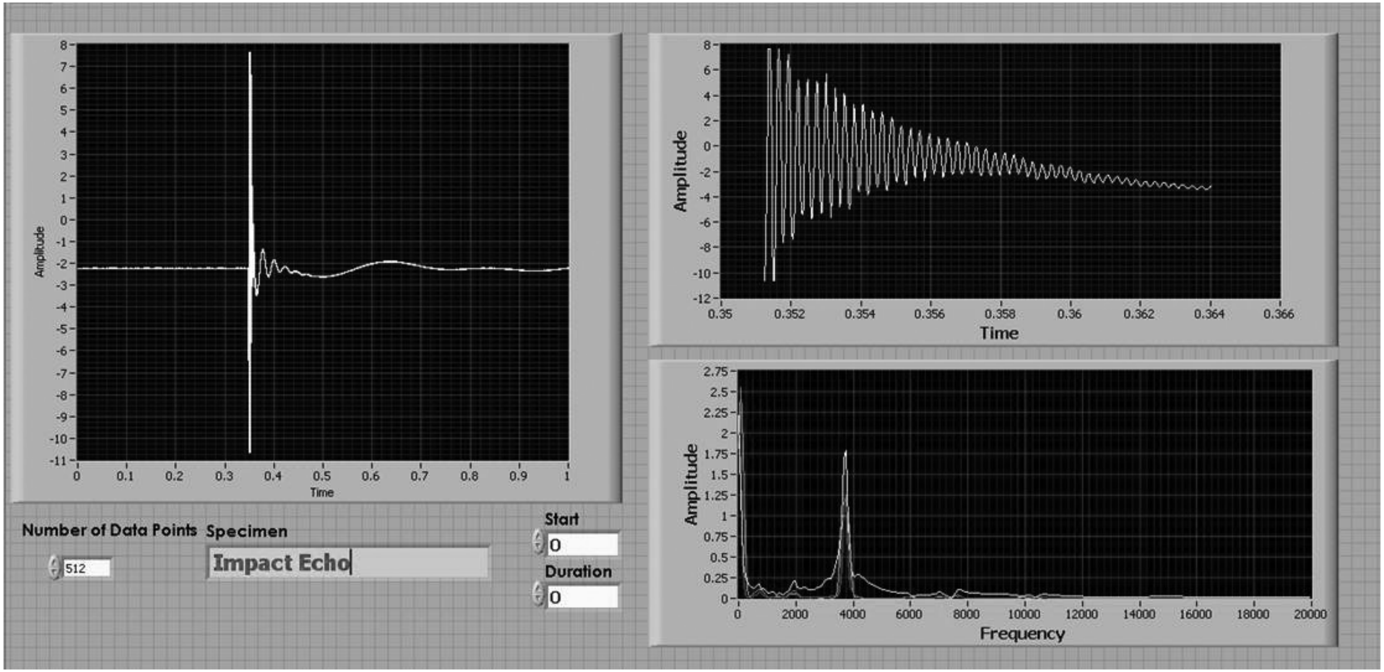
of concrete slabs during curing, the detection of voids, and assessment of damage (ACI Committee Report 228 2R-98).

An impact is used to generate a Rayleigh (surface) wave. A series of sensors is placed along the surface of the member. The signals recorded progressing through the surface-mounted sensors are processed to make inferences on the properties of the underlying material. An informative discussion on the method is included in ACI Committee Report 228 2R-98.

Crosshole sonic logging and sonic echo

These techniques are applicable to inspection of deep foundations and are tests that are similar to ultrasonic pulse velocity and impact-echo in principle. A comprehensive discussion is included in ACI Committee Report 228 2R-98 on the non-destructive stress wave based methods applicable to deep foundations.

The sonic echo method involves delivering a small impact to the accessible top of the deep foundation and monitoring for the return of the stress wave from the end of the foundation. An accelerometer is placed at the top of the foundation, which registers the initial impact as well as the return to the data acquisition hardware. If the length of the pile and the time elapsed for the reflection wave to return to the transducer are known, the stress wave speed can be calculated as a material property for the pile. Similarly, if the



4.14 Screen shot of a commercially available impact-echo system demonstrating the time domain and frequency domain signals.

velocity is known, then the length can be calculated and compared with the design length or the presence of any imperfections can be judged.

The crosshole sonic logging method is based on the ultrasonic pulse velocity principle and is designed to overcome the depth limitations of the sonic echo method. It involves measurement of ultrasonic pulse velocity along the length of the deep foundation by lowering submersible sensors into shafts that are placed in the member (Fig. 4.15). The shafts can be parallel plastic tubes laid during placement of concrete for subsequent testing or can be core holes in an after-the-fact assessment.

The shafts are filled with a slurry or water for coupling. The sensors are lowered and raised simultaneously and ultrasonic pulse velocity determinations are made along the depth of the member, providing information on the quality of the concrete placed between the shafts. Changes in ultrasonic pulse velocity would indicate poor-quality material, whether in terms of strength, modulus, or defects such as contamination, poor consolidation, or voids.

4.5.3 Tests applicable to corrosion of reinforcing steel

Corrosion is an electrochemical process involving the flow of electrons between areas serving as the anode and the cathode. This chapter will discuss basic tools in assessment of concrete structures in which corrosion is an issue.

4.15 Crosshole sonic logging of an existing deep foundation.

One of the visually apparent signs of corrosion activity in a concrete structure is the dislocation or delamination of concrete cover (Fig. 4.4). The product of corrosion is expansive in nature and imposes tensile stresses on the cover concrete. A sounding survey either with a light impact hammer or a chain as described in ASTM D 4580-03 provides an indication of the delaminations and areas where corrosion activity is occurring.

Carbonation testing

Carbonation testing is performed by phenolphthalein staining. Carbonation of concrete depletes the passivating layer surrounding the reinforcing steel by way of reduction in pH. The carbonation test is essentially a pH test indicating the depth of pH reducing effects of carbonation into the concrete member. The test is performed by simply spraying the phenolphthalein solution onto a freshly fractured surface. A color change into pink or purple would indicate the concrete is non-carbonated while no change in color would indicate carbonation has occurred. A carbonation front reaching the depths of reinforcing steel would indicate a higher potential for corrosion activity. A chemical pH determination on dust or samples removed from reinforcing steel depth would numerically indicate the pH in lieu of a visual indicator. Figure 4.16 shows a corner of a concrete column that had already cracked and was loosely in place. The corner is split in half and matching faces are shown facing the camera upon spraying with the phenolphthalein solution.

4.16 Carbonation testing on a column corner.

Note that the carbonation front progressed inward from the perimeter of the cracked corner over time and also at the perimeter of the crack with notable carbonation on the piece shown on the left.

Water-soluble chloride content

Water-soluble chloride testing in accordance with ASTM C 1218 is a key test in corrosion evaluations, particularly in chloride-induced corrosion. The test is useful in identifying the presence of chlorides and also identifying whether chloride-containing de-icers were used on flatwork (Dilek, 2005).

The titration procedure outlined in the ASTM C1218 test method uses a small dust sample. Dust samples can be collected using a hammer drill and a masonry bit. Particular care has to be taken in order not to contaminate the dust samples, particularly if samples are extracted from multiple depths through the same hole.

Alternatively, core samples can be sawed into small disks and later pulverized in the laboratory (Fig. 4.17). This method not only allows stratification of

4.17 Stratification of core for determination of chloride profile.

chlorides for a chloride profile (and ph/carbonation if needed), but also allows visual examination of delaminations or the presence of rust. ACI 318, Chapter 4 Durability Requirements imposes the following limits on water soluble chlorides (Table 4.6).

Half-cell potential testing

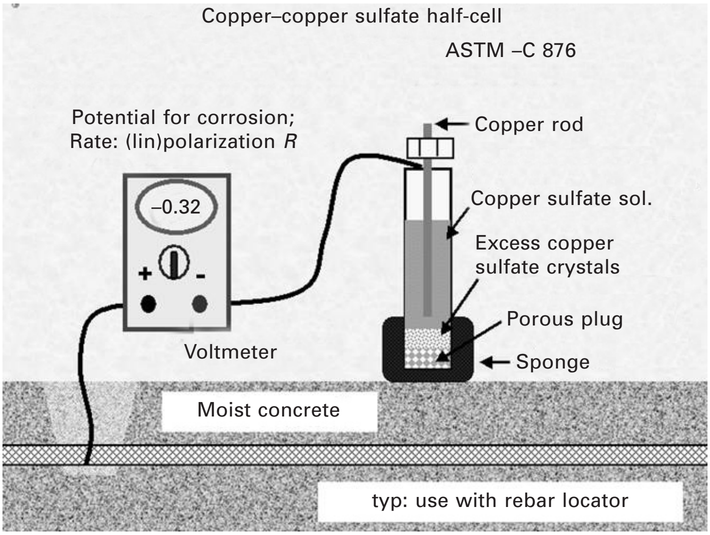
Several electrical methods are used in evaluation of corrosion activity in reinforcing steel. The half-cell potential method ASTM C 876 is used to identify areas of corrosion activity (ACI 222).

Corrosion involves transfer of electrons between areas serving as the anode and the cathode. At the anode, iron atoms lose electrons in a process called half-cell oxidation, creating ferrous ions. Electrons flow to the cathode combining with water and oxygen which is called the reduction reaction. To maintain electrical neutrality, the ferrous ions migrate through the concrete to these cathodic sites where they combine to form hydrated iron oxide or rust. Thus, when the bar is corroding, electrons flow through the bar and ions flow through the concrete (ACI Committee Report 228. 2R). When the bar is not corroding, there is no flow of electrons and ions. As the ferrous ions move into the surrounding concrete, the electrons left behind in the bar give the bar a negative charge. The half-cell potential method is used to detect this negative charge and thereby provide an indication of corrosion activity (ACI Committee Report 228. 2R).

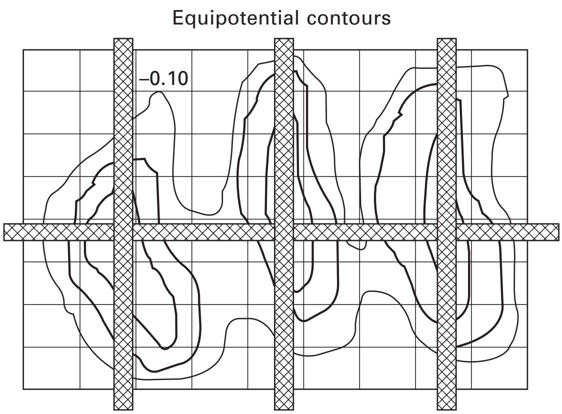
The instrumentation includes a half-cell apparatus containing a copper–copper sulphate solution, a high-impedance voltmeter, and associated connection hardware. The positive terminal of the voltmeter is attached to the reinforcement network, while the negative terminal is connected to the half-cell apparatus (Fig. 4.18). The applicator is contacted to the concrete surface using a moistened sponge. If the bar is corroding, electrons tend to flow from the bar to the half-cell. The voltage reading changes with the extent of corrosion. The half-cell potential is also termed the corrosion potential

Table 4.6 Maximum chloride ion content for corrosion protection of reinforcement – Table 4.4.1 from ACI 318

Type of member	Maximum water-soluble chloride ion (Cl^-) in concrete, percent by weight of cement
Prestressed concrete	0.06
Reinforced concrete exposed to chloride in service	0.15
Reinforced concrete that will be dry or protected from moisture in service	1.00
Other reinforced concrete construction	0.30



4.18 Half-cell potential test configuration.



V_{CSE} > –0.20 V Low prob'ty of corrosion
V_{CSE} > –0.20 V High prob'ty of corrosion*

4.19 Example representation of equipotential lines. (V_{CSE} = volts, copper–copper sulfate electrode)

indicating probability of corrosion activity of reinforcement. In general these limits are considered for corrosion potential:

- –350 mV high likelihood of corrosion
- –200–350 mv corrosion activity is uncertain

Measurements are generally performed on a predetermined grid allowing for a map of equipotential lines to be generated (Fig. 4.19). Alternatively, a cumulative frequency diagram can also be generated. The data are generally interpreted in light of other supporting information such as chloride contents, carbonation depths, delamination surveys, etc.

Linear-polarization method

This method involves measurement of the current to cause a small change in the value of the half-cell potential of the corroding reinforcing steel. For a small change in the open circuit potential, a linear relationship exists between the change in voltage and the current per unit area of the bar surface of which the slope is termed the polarization resistance. Polarization resistance is not a true resistance because the current is expressed per unit area of the bar that is polarized; however, the term is widely accepted. The polarization resistance in ohms-cm^2 can be represented as:

$$R_p = \Delta E / \Delta I \qquad [4.2]$$

where ΔE is change in voltage and ΔI is change in current. The corrosion rate in ampere/cm^2 is given by:

$$I_{corr} = B / R_p \qquad [4.3]$$

where B is a constant in volts.

The corrosion rate is inversely related to the polarization resistance and is expressed as a constant (commonly taken to be 0.026 for reinforced concrete) divided by the polarization resistance in ohms-cm^2. It is possible to convert the corrosion rate into the mass of steel that corrodes per unit of time. If the bar size is known, the corrosion rate can be converted to a loss in diameter of the bar (ACI Committee Report 228 2R-98). A longer discussion on this technique can be found in the ACI 228 Committee Report and the references included.

4.5.4 Locating reinforcing steel

Covermeters

Covermeters are based on electric/magnetic field principles and provide an indication of presence of reinforcing steel in masonry or concrete based on changes in the electric field. Based on the strength of the changes, certain models provide bar size and depth of cover estimations; however, these findings should be verified by isolated chipping. Numerous commercially available models exist in the market for covermeters equipped with audio indicators as well as a visual digital display. Figure 4.20 shows a commercially available covermeter used in locating reinforcing steel in a concrete masonry unit wall. Covermeters are generally suitable for locating reinforcing steel prior to removal of cores from reinforced concrete structures to find safe locations at which cutting through an individual piece of steel would not have detrimental effects. More complex techniques such as radiography or ground-penetrating radar are preferred in critical members or members that are prestressed or post-tensioned using non-bonded tendons.

Ground-penetrating radar

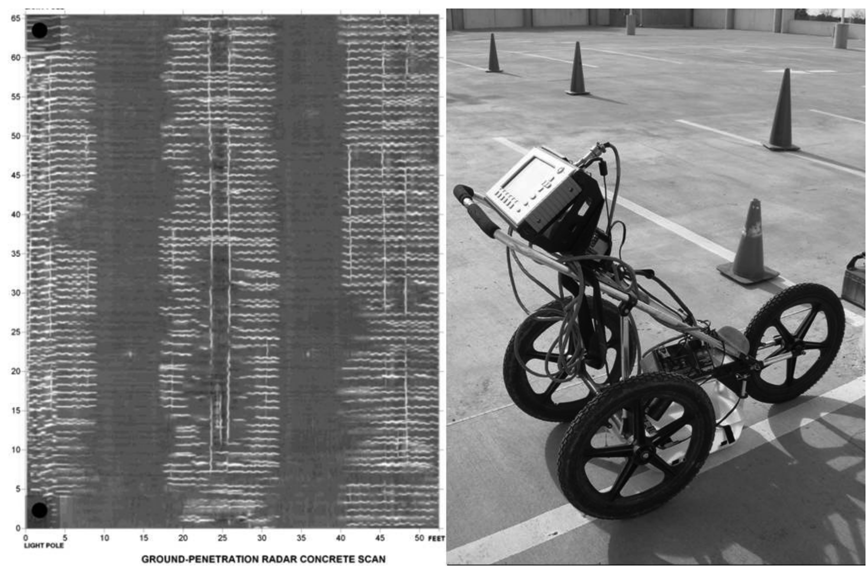
Since the late 1990s, ground-penetrating radar (GPR) has found various applications starting with sub-surface investigations such as locating large objects below grade, for example, vaults and utilities. (ASTM D 6432-99). Improvements in antenna technology and the wavelength and frequency ranges expanded the use in applications involving smaller objects and in shallower members. Current GPR technology allows identification of the steel configuration in concrete members at various depths by analyzing the member in sections changing depths (Figs 4.21 and 4.22).

Radiographic exposures

Radiographic exposures are similar to medical X-rays. They have found widespread use in the construction industry, both in assessment cases to identify the presence and configuration of steel in the member, and in locating steel or prestress/post-tension cables prior to removal of cores or cutting openings in slabs for various reasons.

4.20 A covermeter used in locating reinforcing steel in concrete masonry unit.

4.21 Use of GPR in identification of prestress tendons in a double-tee stem prior to coring.



4.22 A GPR section of a one-way slab and beam system showing draped reinforcing over the beams.

The technique is advantageous when compared with covermeters or GPR in that it provides a direct picture of the steel inside concrete, rather than an indication based on electrical or physics principles. In simple terms, it involves a radioactive source placed on one side of the member and a radiographic film placed on the other side of the member for the duration of a predetermined exposure time (Fig. 4.23).

4.23 Member of interest located between radiographic film and source.

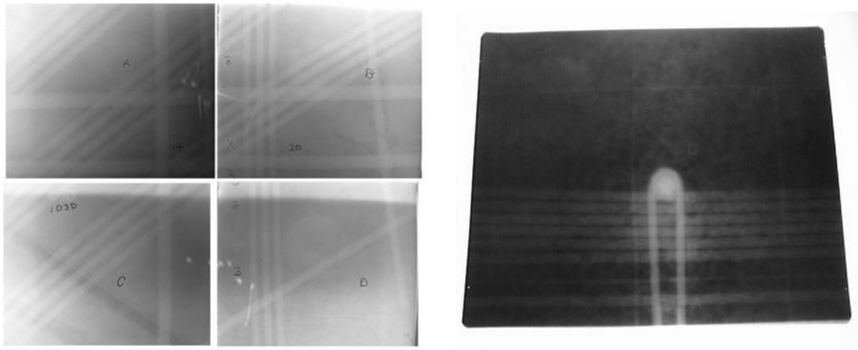
The exposure time is determined based on the thickness and density of the member and the remaining half-life or strength of the radioactive source in curies. The technique requires a safe boundary (generally a 2 mR/hr boundary) to be evacuated during the exposure when the source is engaged. This is generally a 75–100 feet distance depending on the strength of the source, and it creates complications particularly in occupied buildings or downtown settings.

The technique involves placement of lead letters or templates in the frame for reference and identification of the position of objects in the picture with reference to a known location such as a corner or end of the member. Figure 4.24 shows developed films photographed over a light table. Upon developing the film, the measured locations of reinforcing steel on the film need to be corrected for geometric skew (as shown in Fig. 4.25) for identification of true location. Lead letter templates seen in Figs 4.24 and 4.26 provide the reference for these corrections.

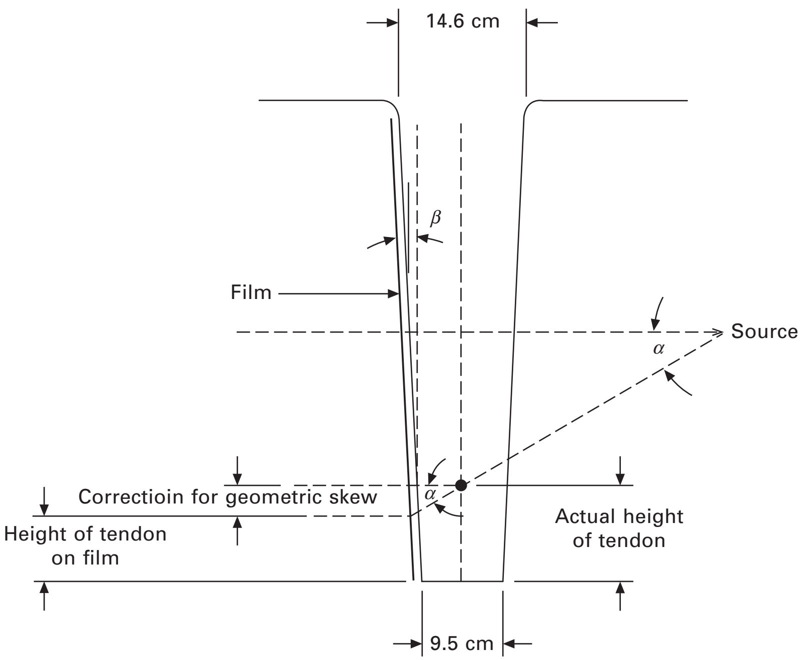
4.6 Laboratory testing on cores

4.6.1 Removal of cores

Upon completion of the steel locating task, cores are removed from the structure or member being investigated (Fig. 4.27). Removal of cores is generally accomplished using an electric drill equipped with a water-cooled



4.24 Radiographic films photographed over a light table upon being developed.



4.25 Correction for geometric skew applied to the position measured on film.

diamond embedded coring bit. Coring bits are commercially available in various sizes; however, the sizes indicated are generally nominal, e.g., a 4 inch core bit has an outside diameter of 4 inches and generally yields a core of 3¾ inches. Removal of cores requires a power source or a generator and a water source to cool the bit. The coring rig generally requires anchorage

4.26 Radiographic source, collimator and lead letter template included in the frame for reference to the end of the member.

4.27 Removal of cores with an electric drill and water-cooled coring bit.

to the member to prevent the rig from rotating in case the bit gets stuck during cutting operations.

Selection of the core bit is important, particularly to meet the requirements of the specific test method to be performed. For example, ASTM C 42 has certain diameter requirements based on nominal maximum aggregate size and the length-to-diameter (*L*/*D*) requirement for the resulting capped core. Generally, an *L*/*D* of 2 is desirable (similar to a laboratory 4 × 8 or 6 × 12 inch specimen) and an *L*/*D* between 1 and 2 is subject to a correction factor. Therefore, it is desirable to study the nominal maximum aggregate size and the depth of a slab, for example, so that a large-diameter core bit is not selected when coring a relatively shallow slab.

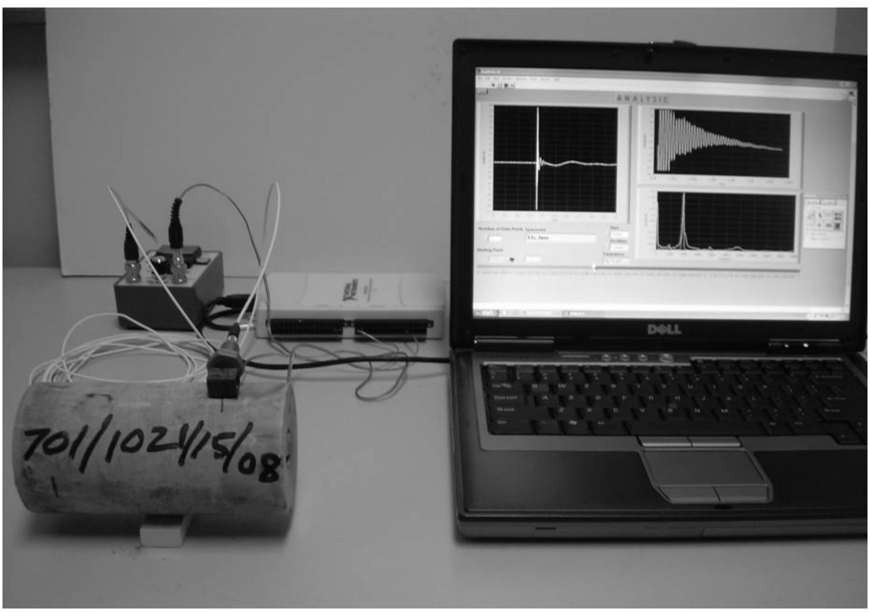
4.6.2 Laboratory testing of core samples

The list below includes frequently used ASTM test methods in assessment of concrete structures, although it is not an all-inclusive list and other test procedures may be useful depending on the specifics of the study as discussed in Section 4.3. The list does not include field testing and NDE procedures discussed in Section 4.5, but includes frequently used laboratory test procedures. Brief descriptions are included when necessary:

1. **ASTM C 39** – This test is used in laboratory strength testing of concrete specimens cast in the field for quality assurance purposes during concrete placements, specimens produced during laboratory mix designs/research or cores removed from structures being evaluated. ACI 318 Chapter 5 Concrete Quality, Mixing and Placing and the ACI 214R-02 Committee Report provide discussions on evaluation and acceptance of strength results. The same compressive strength determination procedure is also used on drilled cores obtained in accordance with ASTM C 42.
2. **ASTM C 42** – The method describes removal procedures, conditioning procedures, and size requirements for sawed cores and beams removed from structures for assessment purposes. The strength testing of the specimens is performed in accordance with ASTM C 39. ACI 318 Chapter 5 Section 5.6.5 Investigation of Low Strengths, Article 5.6.5.4 states: Concrete in an area represented by core tests shall be considered structurally adequate if the average of three cores is equal to 85 percent of f'_c and if no single core is less than 75 percent of f'_c.
3. **ASTM C 496** – This method is used for determination of tensile strength of concrete specimens using the splitting tensile method. Alternatively, sawed beams removed in accordance with ASTM C 42 can be tested in flexure in accordance with ASTM C 78.
4. **ASTM C 215** – This test method involves measurement of resonant frequency of concrete specimens (sawed or cast cylinders/beams) in

compression or torsional modes and provides a dynamic Young's and shear modulus and a dynamic Poisson's ratio determined based on specimen dimensions, unit weight, and resonant frequencies measured. Figure 4.28 shows the test setup of a cylindrical specimen instrumented in torsional mode with the associated accelerometer sensor, data acquisition hardware, and computer software analyzing the signal using a fast Fourier transform for dominant frequency. The resonant principle of ASTM C 215 adapted for thin circular disks sawed from cores found applications in assessment of damage and damage gradients (Leming *et al.*, 1998; Dilek and Leming, 2008; Dilek, 2006b, 2007a, 2008).

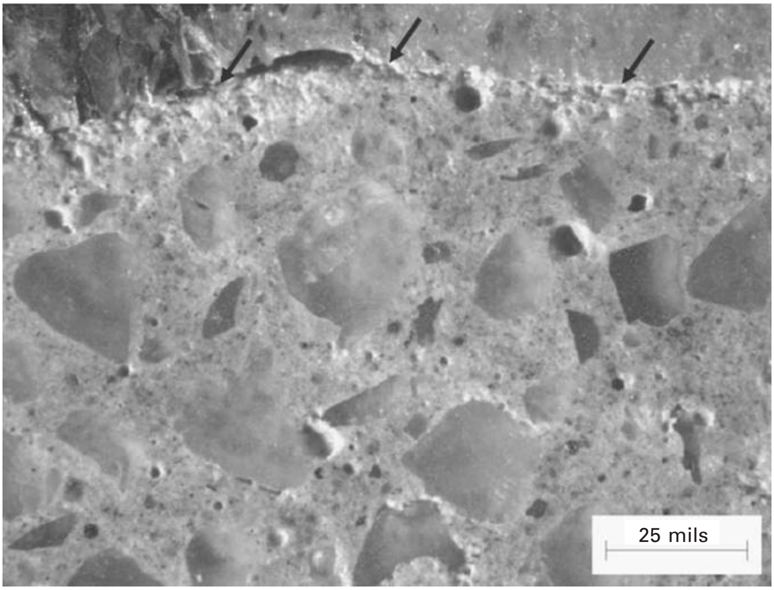
5. **ASTM C 469** – This test involves determination of longitudinal and lateral strains on a concrete specimen during a compressive strength test. It requires a certain frame to instrument the specimen with dial/digital gauges for strain. Measurement of strains during loading enables determination of Young's modulus and Poisson's ratio.
6. **ASTM C 642** – This method involves determination of density absorption and voids in concrete specimens by subjecting the specimens to submersion. Although a simple physical test, ASTM C 642 is useful in identification of changes in unit weight and consolidation of concrete specimens.
7. **ASTM C 457** – ASTM C 457 is based on a visual count of entrapped/entrained air voids in concrete sections under a microscope. It is generally



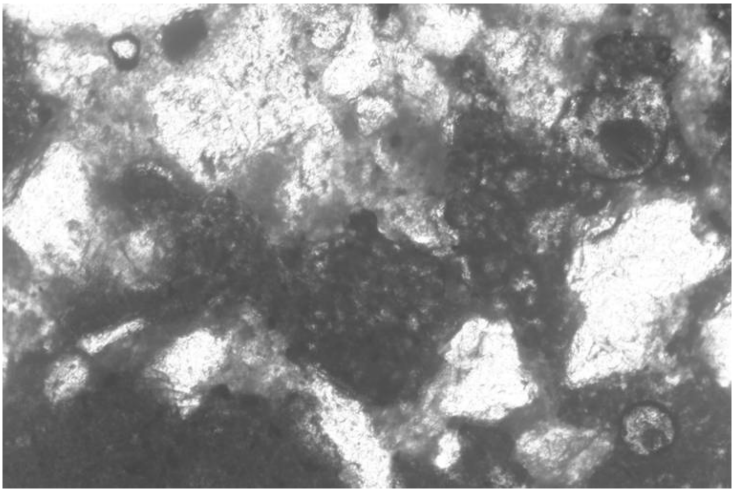
4.28 Determination of resonant frequency of a cylindrical specimen in torsional mode.

performed in combination with petrographic evaluations of concrete. ASTM C 457 results, in conjunction with fresh concrete unit weights, and ASTM C 642 results are generally used in resolving whether the concrete was adequately air-entrained and whether it meets the air-entrainment requirements of the specification.

8. **ASTM C856** – Petrographic evaluation of concrete is one of the most frequently used test methods in assessment of concrete structures and placements. The technique is qualitative in nature and involves microscopic visual evaluations for visually apparent signs such as presence of alkali silica gel, bleed water paths (Fig. 4.29), ettringite formation, finishing effects at the surface, etc. that may provide explanations as to the cause of the problem. Frequently, scratching or grinding of the paste, carbonation tests, or dye penetration tasks (Fig. 4.30) are performed as part of the test. ASTM Special Technical Publication 1215 provides a compilation of papers on evaluation of various deteriorating mechanisms using petrography.
9. **ASTM C1202** – Often referred to as the rapid chloride permeability test, this method involves testing concrete disks sawed from cylinders or cores for permeability. The sample is clamped in a test chamber exposing the side to a chloride solution. Capacitance of the specimen in coulombs is measured over time, as it changes with permeation of the solution into the specimen.



4.29 Screen capture from microscopic evaluation of entrapped bleedwater in aggregate transition zone.



4.30 Dye penetration test performed during petrographic evaluation of concrete.

4.7 Load testing of structures

ACI Committee Report 318, Chapter 20 is the standard for load testing of concrete structures. ACI Committee 437 reports provide detailed discussions pertinent to strength evaluation and load testing of concrete structures. These include:

- 437.1R-07: Load Tests of Concrete Structures: Methods, Magnitude, Protocols, and Acceptance Criteria Purchase;
- 437R-03: Strength Evaluation of Existing Concrete Buildings;
- 437R-91: Strength Evaluation of Existing Concrete Buildings.

Chapter 20 requires a strength evaluation if there is doubt that part or all of a structure meets the safety requirements of ACI 318. The analytical strength evaluation involves measurement of section dimensions, steel configuration, and material properties. If analytical approaches are not feasible, a load test is required. A load test serves as a proof test to ensure the capacity of the structure to bear service loads and therefore the safety of the public. Load tests can be implemented in case of deficient material properties, damaged or deteriorated structures, or if the use or load rating of a structure is proposed to be changed and no drawings or other information are available for numerical analysis.

4.7.1 Load test overview

The load arrangement is required to be designed to maximize the effect in question (deflection, rotation, or stress) at the specific area of the structure under investigation. Multiple load tests or test load arrangements/magnitudes can be organized if one single arrangement does not provide the desired effect such as maximum moment or maximum shear.

If adjoining members carry a portion of the load, the load test should be revised to develop effects consistent with the intent of the load test. In case of precast decking members or double-tees, it may be practical to isolate each member from adjoining neighbor members by severing the connections providing membrane action. In case of a continuous multi-span concrete slab–beam system, however, this would not be practical and the load test may involve skip-loading (loading spans 1 and 3 for example) to create maximum moments at mid-span and loading consecutive spans (loading spans 1 and 2 for example) to create maximum moments at support (Fig. 4.31).

4.7.2 Load test specifics per ACI 318 Chapter 20

Load intensity

The test load including the dead load already in place is required to be no less than 85% of (1.4 dead load + 1.7 live load) combination.

Load application

Initial measurements of deflection are recorded one hour before the application of the load. The test load is applied in four increments, with

4.31 Skip loading of spans to create maximum moment at mid-span.

response measurements taken in-between increments. The load is maintained on the structure for 24 hours, and then final measurements are taken. The structure is also monitored for imminent signs of failure or drastic increases in deflections during the 24 hour period. Then the structure is unloaded, and recovery measurements are taken 24 hours after the test load is removed.

Load test deflection limits and recovery conditions

The measured deflections are required to satisfy one of the two conditions:

1. $$\Delta_{max} < l_t^2 / 20\,000\,h \qquad [4.4]$$

   where l_t = span length of the member, Δ_{max} = maximum measured deflection, and h = height of the member.

2. $$\Delta_{rmax} < \Delta_{max} / 4 \qquad [4.5]$$

   where Δ_{max} = maximum measured deflection, and Δ_{rmax} = residual deflection.

Provision 1 is a maximum limit based on the slenderness of the member. No recovery condition is necessary if the deflections are less than the limit established based on provision 1. For typical flexural members and slab systems, however, this limit is exceeded and Provision 2 with recovery conditions takes effect. Provision 2 does not establish a maximum limit on the exhibited deflections; however, it requires the structure to recover 75% of the exhibited deflections. It is also required that the structure does not exhibit signs of crushing or spalling indicative of failure.

If the measured maximum deflections or the residuals do not satisfy Equations [4.5] and [4.6], a retest is allowed not earlier than 72 hours after removal of the load subject to the following requirement:

$$\Delta_{rmax} < \Delta_{fmax}/5 \qquad [4.6]$$

where Δ_{fmax} = maximum measured deflection during the retest and, Δ_{rmax} = residual deflection.

Shoring

Prior to beginning of load testing, an arrest system typically consisting of shoring is required to provide support in case of failure or excessive deflection (Fig. 4.32). Shoring has to be designed to support the dead load plus the test load in case load-carrying capacity is lost. Depending on the location of the load test in a structure, shoring can be utilized to share the load through multiple floors or convey the load to the sub-grade. Shoring

4.32 Shoring installed below structure for load testing.

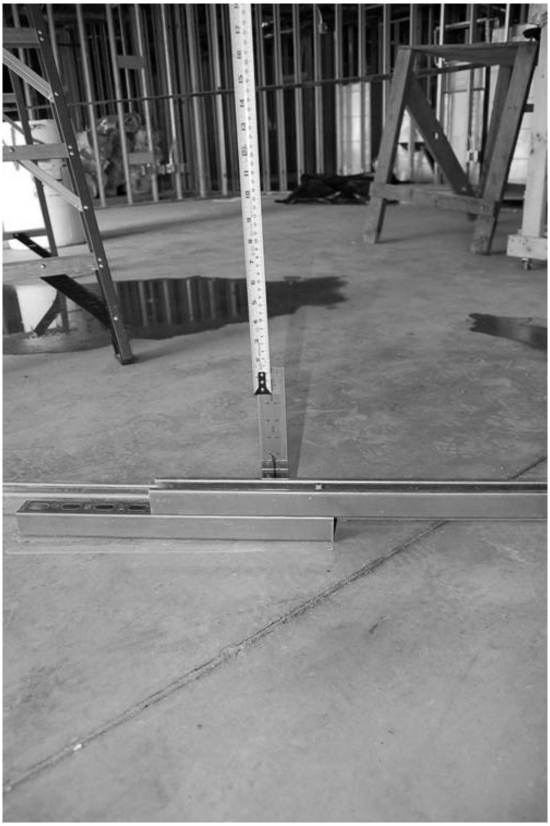
4.33 Shoring installed with gap to aid the structure in case of excessive deflections.

is installed with a gap to the bottom of the structure (Fig. 4.33). This arrangement ensures that the shoring does not provide the structure any assistance during load testing but would provide support in case excessive deflections were to occur. Shoring can also be utilized to support deflection measurement instrumentation.

Deflection measurement instrumentation

The primary criteria for load tests are based on deflections and recovery; therefore accurate measurement of deflections is important. The conventional way of measuring deflections in the field has generally been the use of dial gauges mounted to the shoring with the needle in contact with the bottom of the structure. The use of dial gauges requires personnel physically getting under the area being loaded to read the dial gauges. It is not uncommon to accidentally impact shoring supporting the gauge while collecting readings. The use of suspended measuring devices to provide reference to a fixed benchmark or a tight piano wire is also a practical alternative, although these devices do not quite provide the precision of dial gauges or electronic measuring devices (Fig. 4.34).

Figure 4.34 shows use of linear voltage displacement transducers (LVDT) to measure the deflection of a beam during load testing. LVDTs feed data to data acquisition hardware. The data is monitored real-time and logged using



4.34 Suspended tape measure as a means of measuring deflections.

a field computer running data acquisition and analysis software (Fig. 4.35). Use of an electronic monitoring system enables monitoring of deflections in real-time from a remote location (away from the test area for safety) and provides the ability to react to unexpected deflections/failures if they were to occur. Analog dial gauges were installed as a back-up system in case of loss of power (Fig. 4.35).

Load test materials

The logistics of loading and identification of load testing materials presents one of the challenges of load testing. Load tests may need to be performed on upper levels of a structure with limited or no vehicle access to deliver loading material and no area for staging significant amounts of load or to quickly unload in case a failure occurs. Typical approaches to load tests generally involve use of water, various construction materials, or hydraulic jacks. The use of a hydraulic jack presents a new set of challenges as it requires possible reversal of load direction, innovative design, and use of a reaction frame to support the jack for the magnitude of load necessary. Also, simulation of uniformly distributed loads presents a challenge using point loads. Figure 4.36 shows the use of a reaction frame to load test a concrete roof slab and a load cell for measurement of load.

4.35 Digital LVDT allowing real-time monitoring of deflections and analog dial gauge back-up.

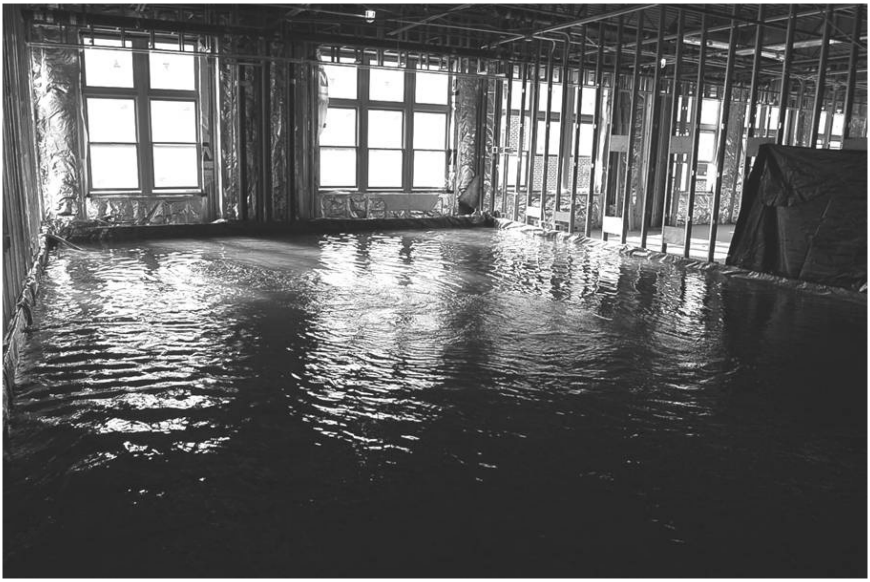
4.36 Use of a reaction frame to load test a concrete roof slab.

The use of water is also a common method in load testing. This method generally requires use of barrels or building temporary pools on-site. The pools have to be equipped with intermediate dividers to prevent water from accumulating in the middle as deflections occur. The use of water presents several challenges, including the need to locate the water at the test site, the large volume of water necessary to provide the required magnitude of test load, and the long time required to unload the water in case a failure were to occur (Fig. 4.37)

One of the more practical ways of loading structures is prearranged construction materials on palettes, if forklift access is possible. This method requires some level of preplanning effort and labor; however, if materials are arranged in increments that match the four load increments, it provides practical advantage over any other method. Figures 4.31 and 4.38 show solid concrete masonry blocks and bags of salt arranged on wood pallets prior to the load test, to form units of precalculated weight. The test load arrangement using pallets provided the capability of quickly loading and unloading the prescribed load. This alternative not only reduced the timeframe and the cost of the testing compared to loading the structure with water, it also permitted quick unloading of the structure in case excessive deflections or failure were encountered.

Load testing of piles

Figure 4.39 shows load testing of a pile using a hydraulic jack for verification of load-carrying capacity. The procedure is different from ACI 318 Chapter 20



4.37 Use of water as load test material.

4.38 Bags of de-icing salt organized in palettes.

and is explained in detail in ASTM D 1143 / D 1143M – 07. The procedure generally involves construction of a reaction frame anchored to an adjacent group of piles or pile caps and application of force using a hydraulic jack. Pile load testing is generally performed on a group of test piles, serves as a

4.39 Pile load testing using a hydraulic jack and reaction frame anchored to adjacent piles.

proof test for the design loading and design criteria, and enables modifications (as needed) to the design prior to production of the remaining piles.

4.8 Sources of further information and advice

Recommended reading

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- ICRI Guideline No. 03731–Guide for Selecting Application Methods for the Repair of Concrete Surfaces, International Concrete Repair Institute, Des Plaines, IL
- ICRI Guideline No. 03732–Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, and Polymer Overlays, International Concrete Repair Institute, Des Plaines, IL
- ICRI Guideline No. 03736–Guide for the Evaluation of Unbonded Post-Tensioned Concrete Structures, International Concrete Repair Institute, Des Plaines, IL

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Relevant ACI Committee Reports

- ACI 201.2R-08 – Guide to Durable Concrete
- ACI 228.IR-03 – In-Place Methods to Estimate Concrete Strength
- ACI 349.3R-02 – Evaluation of Existing Nuclear Safety-Related Concrete Structures
- ACI 362R-85 – State-of-the-Art Report on Parking Structures
- ACI 207.3R-94 – Practices for Evaluation of Concrete in Existing Massive Structures for Service Conditions
- ACI 222R-01 – Protection of Metals in Concrete Against Corrosion
- ACI 224.1R-07 – Causes, Evaluation and Repair of Cracks in Concrete Structures
- ACI 345.1R-06 – A Guide for Maintenance of Concrete Bridge Members
- ACI 362.2R-00 – Guide for the Design of Durable Parking Structures (reapproved 2005)
- ACI 546.1R-80 – Guide for Repair of Concrete Bridge Superstructures
- ACI 546.2R-98 – Guide to Underwater Repair of Concrete
- ACI 503.2-92 – Use of Epoxy Compounds with Concrete (reapproved 2003)

4.9 Acknowledgement

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ACI Committee 318, Building Code Requirements for Structural Concrete and Commentary, *ACI Manual of Concrete Practice*, Farmington Hills, MI, American Concrete Institute.

ACI Committee 364.1R-04, Guide for Evaluation of Concrete Structures Prior to Rehabilitation, *ACI Manual of Concrete Practice*, Farmington Hills, MI, American Concrete Institute.

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ASTM C 215, Standard Test Method for Fundamental Transverse, Longitudinal, and Torsional Frequencies of Concrete Specimens, *Annual Book of ASTM Standards*, West Conshohocken, PA, American Society for Testing and Materials.

ASTM C 39, Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens, *Annual Book of ASTM Standards*, West Conshohocken, PA, American Society for Testing and Materials.

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ASTM C 805, Standard Test Method for Rebound Number of Hardened Concrete, *Annual Book of ASTM Standards*, West Conshohocken, PA, American Society for Testing and Materials.

ASTM C 823-00, Standard Practice for Examination and Sampling of Hardened Concrete in Constructions, *Annual Book of ASTM Standards*, West Conshohocken, PA, American Society for Testing and Materials.

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5

Standards and guidelines for repairing concrete structures

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Abstract: This chapter deals with international regulations for condition evaluation, repair and maintenance of concrete structures and is based largely on the new European series of standards EN 1504. First of all the systematics of concrete repair is explained as defined in EN 1504. It consists of the assessment of the structure, followed by the selection of options, principles and methods for protection and repair and finally maintenance of the structure. According to this order, selected relevant international regulations are listed for the following subjects: condition assessment, surface preparation, concrete restoration, strengthening, surface protection, crack injection, corrosion protection and maintenance. We have tried to list all relevant regulations, but of course it is not possible to find all of them. Consequently, the list of regulations cannot be complete. It was also not intended to give comments to the contents of all these regulations, but to have a structured overview on the international regulations.

Key words: concrete restoration, EN 1504 standard, concrete structure condition assessment, crack injection surface protection, surface preparation, corrosion protection, maintenance.

5.1 Introduction

This chapter presents worldwide legislation, standards, guidelines and recommendations for diagnosis and repair of concrete structures. The structure of the European standard EN 1504 is presented in detail as this standard covers all phases of a repair project, including diagnosis, selection of repair method, choice of repair material and maintenance. However, other relevant standards and regulations are also listed.

5.2 Systematics of concrete repair according to EN 1504

5.2.1 General

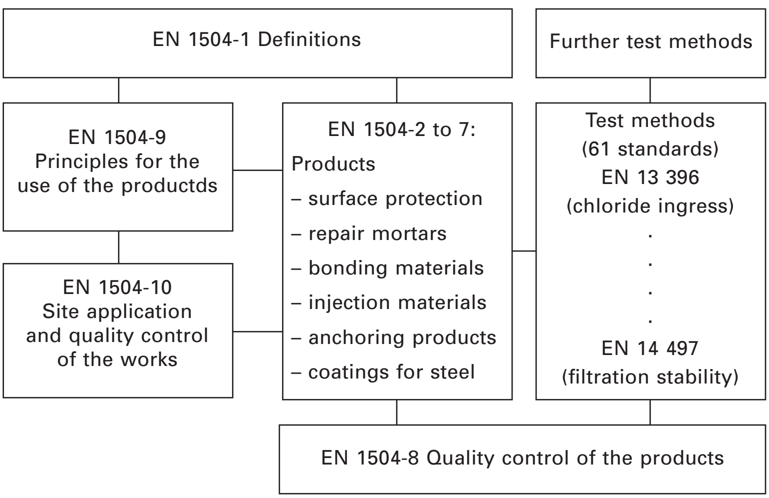
With the EN 1504 a new European standard on protection and repair of concrete structures has been worked out, based on the actual state of knowledge regarding the properties of the products, the principles for the use of the products, application and quality control. It consists of 10 main standards and 61 standards for test methods.

As shown in Fig. 5.1, parts 2 to 7 regulate the performance of the products and systems for protection and repair of concrete structures with respect to CE marking (*Conformité Européenne*). The use of the products is based on a logical sequence consisting of options (repair strategies), repair principles and repair methods, which are listed in tables in part 9 of EN 1504. This scheme ensures that suitable products are selected based on the requirements of the practical case. At the end of 2008 this new series of standards replaced the existing national standards in Europe. This will also have significant impact for other countries.

Part 1 gives definitions, part 8 regulates the quality control of the products and part 10 gives a general guideline for site application and quality control of the works. To allow CE marking of the products, 61 standards describing test methods for the different properties of the products have been prepared. These standards ensure that testing of the products will be according to the same standards for all products for protection and repair of concrete structures used in Europe.

5.2.2 Phases of repair projects

The phases of repair projects follow a logical sequence, which is dominated by engineering aspects. Figure 5.2 gives a general scheme according to figure A1 within annex A of EN 1504-9. This figure shows the well-known elements of assessment, planning, design and quality control. However, the



5.1 Structure of EN 1504.

	Information about the structure	Process of assessment	Management strategy	Design of repair work	Repair work	Acceptance of repair work
Basic considerations and actions	• Condition and history of structure • Documentation • Previous repair and maintenance	• Defects and their classification and causes • Safety/structural appralsal during protection and repair	• Options • Principles • Methods • Safety/structural appraisal before protection and repair	• Intended use of products • Requirements – substrate – products – work • Specifications • Drawings • Safety/structural appraisal after protection and repair	• Choice and use of products and systems and methods and equipment to be used • Tests of quality control • Health and safety	• Acceptance of testing • Remedial works • Documentation
Relevant clauses in EN-1504-9 and other parts of the EN 1504 series	• Clause 4 of this European Standard	• Clause 4 of this European Standard	• Clauses 5 and 6 of this European Standard	• EN 1504-2 to EN 1504.7 • Clauses 6, 7 and 9 of this European Standard	• Clauses 6, 7, 9 and 10 of this European Standard • EN 1504-10	• Clause 8 of this European Standard • EN 1504-10

5.2 The phases of repair projects according to EN 1504-9.

systematics of general planning is quite innovative. It consists of a hierarchy of levels, namely options, principles and methods, which are described in more detail in Section 5.2.4.

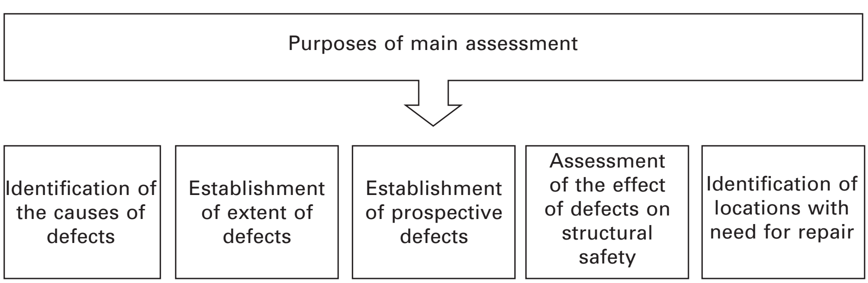
5.2.3 Process of assessment

A broad and detailed assessment is essential for the success of a repair project. The purpose of the main assessment is shown in Fig. 5.3. With respect to later planning of repair, generally a distinction should be made between defects in concrete and defects caused by reinforcement corrosion. Common causes of defects according to figure 1 of EN 1504-9 are shown in Fig. 5.4. More details on requirements for assessment are given in EN 1504-9.

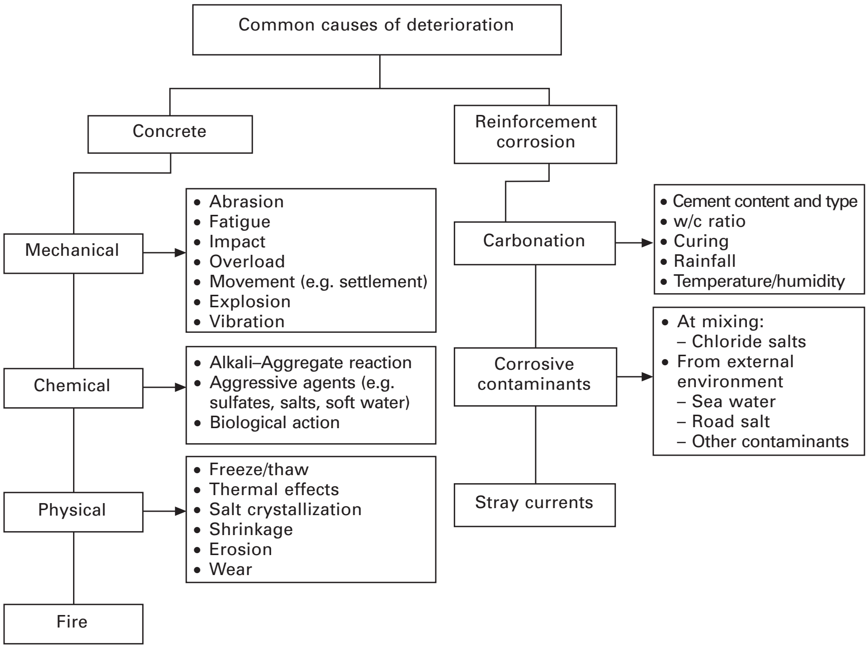
5.2.4 Options, principles and methods for protection and repair

As already mentioned, the rules for the use of products and systems for protection and repair of concrete structures are based on a hierarchy of different levels, namely options, principles and methods. According to EN 1504-9 the options shown in Fig. 5.5 shall be taken into account in deciding the appropriate action to meet the future requirements for the life of the structures:

For protection and repair, different principles have been defined, separately for repair and prevention of damages to the concrete and damages induced by reinforcement corrosion. Table 5.1 shows the six principles for protection and repair of concrete and the five principles to prevent damages due to reinforcement corrosion, respectively. These principles are based on the RILEM Technical Recommendation 124-SRC 'recommendation for repair strategies for concrete structures damaged by reinforcement corrosion'. To protect or repair a concrete structure according to the principles, different methods are available. Figures 5.6–5.9 of this chapter give some examples and short



5.3 Purposes of main assessment according to EN 1504-9.



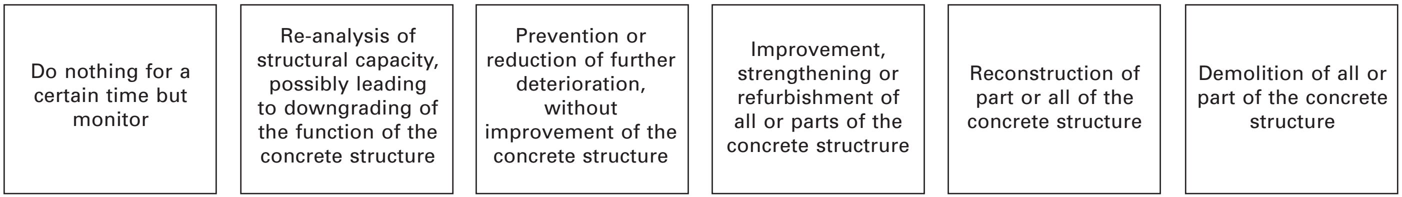
5.4 Common causes of defects according to EN 1504-9.

descriptions of selected principles and methods of EN 1504-9. Altogether, 43 methods are described within EN 1504-9. Not all of them are covered by the EN 1504 series, but some are covered by other standards, and some of them are actually not standardised, but expected to be regulated in the future. The system of options, principles and methods is the basis for the selection of products by the designer. The process of planning and selection of products is described in the next section.

Figures 5.6–5.9 give examples of certain cases, with the performance characteristics relevant for the selected repair methods. EN 1504-2–7 contain the performance characteristics of the products together with the corresponding test methods. In this way, the products are selected individually for the demands of the special case of repair or protection of a concrete structure. As a result, the products are described by a list of required performance characteristics instead of simple classes, resulting in a high level of flexibility. Finally inspection and maintenance requirements shall be defined by the designer.

5.2.5 Planning and selection of products

Figure 5.10 shows the systematics of planning according to EN 1504-9. As already shown in Fig. 5.2, the planning starts with the assessment of the status of the structure. The next steps are selection of options (repair



5.5 Options for deciding the appropriate action to meet the future requirements for the life of the structures according to EN 1504-9.

Table 5.1 Principles for repair and protection for damages of the concrete according to EN 1504-9

Principle	Examples of methods based on the principles	Relevant Part of EN 1504 (where applicable)
Principles and methods related to defects in concrete		
1. Protection against ingress	1.1 Hydrophobic impregnation	2
	1.2 Impregnation	2
	1.3 Coating	2
	1.4 Surface bandaging of cracks	
	1.5 Filling of cracks	5
	1.6 Transferring cracks into joints	
	1.7 Erecting external panels[a]	
	1.8 Applying membranes[a]	
2. Moisture control	2.1 Hydrophobic impregnation	2
	2.2 Impregnation	2
	2.3 Coating	2
	2.4 Erecting external panels	
	2.5 Electrochemical treatment	
3. Concrete restoration	3.1 Hand-applied mortar	3
	3.2 Recasting with concrete or mortar	3
	3.3 Spraying concrete or mortar	3
	3.4 Replacing elements	
4. Structural strengthening	4.1 Adding or replacing embedded or external reinforcing bars	
	4.2 Adding reinforcement anchored in pre-formed or drilled holes	6
	4.3 Bonding plate reinforcement	4
	4.4 Adding mortar or concrete	3,4
	4.5 Injecting cracks, voids or interstices	5
	4.6 Filling cracks, voids or interstices	5
	4.7 Prestressing – (post-tensioning)	
5. Increasing physical resistance	5.1 Coating	2
	5.2 Impregnation	2
	5.3 Adding mortar or concrete	3
6. Resistance to chemicals	6.1 Coating	2
	6.2 Impregnation	2
	6.3 Adding mortar or concrete	3
Principles and methods related to reinforcement corrosion		
7. Preserving or restoring passivity	7.1 Increasing cover with additional mortar or concrete	3
	7.2 Replacing contaminated or carbonated concrete	3
	7.3 Electrochemical realkalisation of carbonated concrete	
	7.4 Realkalisation of carbonated concrete by diffusion	
	7.5 Electrochemical chloride extraction	

Table 5.1 Cont'd

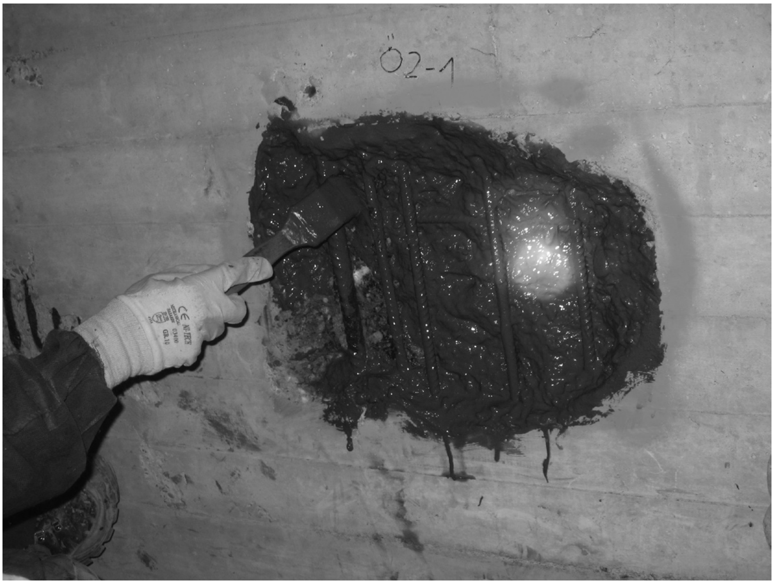
Principle	Examples of methods based on the principles	Relevant Part of EN 1504 (where applicable)
8. Increasing resistivity	8.1 Hydrophobic impregnation 8.2 Impregnation 8.3 Coating	2 2 2
9. Cathodic control	9.1 Limiting oxygen content (at the cathode) by saturation or surface coating	
10. Cathodic protection	10.1 Applying an electrical potential	
11. Control of anodic areas	11.1 Active coating of the reinforcement 11.2 Barrier coating of the reinforcement 11.3 Applying corrosion inhibitors in or to the concrete	7 7

[a] These methods may also be applicable to other principles.

Principle no.	Principle and its definition	Methods basd on the principle
Principle 1 [PI]	'Protection against ingress' Reducing or preventing the ingress of adverse agents, e.g. water, other liquids, vapour, gas, chemicals and biological agents.	1.1 Hydrophobic impregnation 1.2 Impregnation 1.3 Coating 1.4 Surface bandaging of cracks 1.5 Filling of cracks 1.6 Transferring cracks into joints 1.7 Erecting external panels 1.8 Applying membranes

Principle no.	Principle and its definition	Methods basd on the principle
Principle 2 [MC]	Moisture control Adjusting and maintaining the moisture content in the concrete within a specified range of values.	2.1 Hydrophobic impregnation 2.2 Impregnation 2.3 Coating 2.4 Erecting external panels 2.5 Electrochemical treatment

5.6 Examples for surface coating according to methods 1.3 and 2.3 of EN 1504-9.



Principle no.	Principle and its definition	Methods based on the principle
Principle 3 [CR]	**Concrete restoration** Restoring the original concrete of an element of the structure to the originally specified shape and function Restoring the concrete structure by replacing part of it.	3.1 Applying mortar by hand 3.2 Recasting with concrete 3.3 Spraying concrete or mortar 3.4 Replacing elements

5.7 Example of the preparation of a concrete surface for applying mortar by hand according to method 3.1 of EN 1504-9.

strategy), repair principles and repair method, as defined in the previous section. Based on this selection scheme, the repair materials can be chosen. EN 1504-9 defines performance characteristics for every repair method, separately for all intended uses and for certain intended uses. The designer selects the performance characteristics based on the requirements of the special repair project and the selected repair methods.

5.3 International regulations for condition evaluation, repair and maintenance

5.3.1 General

In the following sections international regulations for the different phases of a repair work are summarised. The main focus is on regulations and

5.8 Example for recasting with concrete according to method 3.2 of EN 1504-9.

standards in the English language. An overview of the different topics in this section is given in Fig. 5.11. Additional relevant international standards are summarised in Appendix A. In these sections mainly American standards as well as European standards for testing certain properties of hardened repair products are listed.

5.3.2 Condition assessment

The condition evaluation is the first highly important step in repair of all kinds of structures. The purpose of the condition evaluation is to identify and define causes and distribution of distress. In Table 5.2 different international regulations and guidelines are given, which can help to plan and carry out a condition evaluation. Nevertheless, these publications can only be a helping tool as the condition evaluation has to take into account all factors relating to the structure, which can of course not be part of a general recommendation.

5.3.3 Surface preparation

A careful and intensive surface preparation of both the steel and the concrete is the basis for a successful repair not only for concrete restoration or the

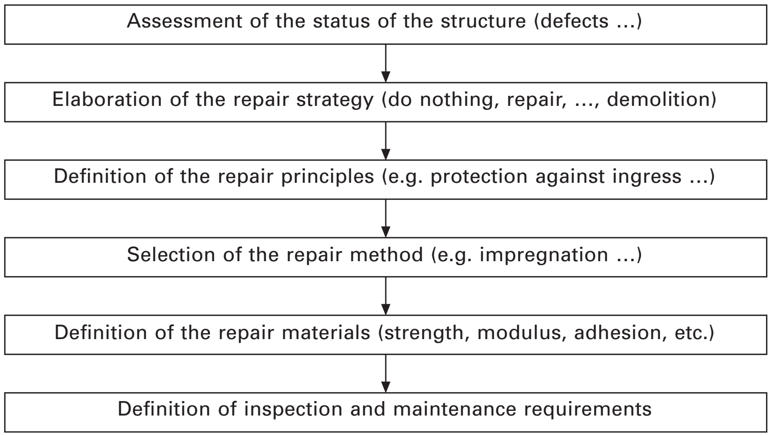
Principle no.	Principle and its definition	Methods based on the principle
Principle 4 [SS]	**Structural strengthening** Increasing or restoring the structural load bearing capacity of an element of the concrete structure.	4.1 Adding or replacing embedded or external reinforcing bars 4.2 Adding reinforcement anchored in pre-formed or drilled holse 4.3 Bonding plate reinforcement 4.4 Adding mortar or concrete 4.5 Injecting cracks, voids or interstices 4.6 Filling cracks, voids or interstices 4.7 Prestressing – (post-tensioning)

5.9 Example for injection of cracks according to method 4.5 of EN 1504-9.

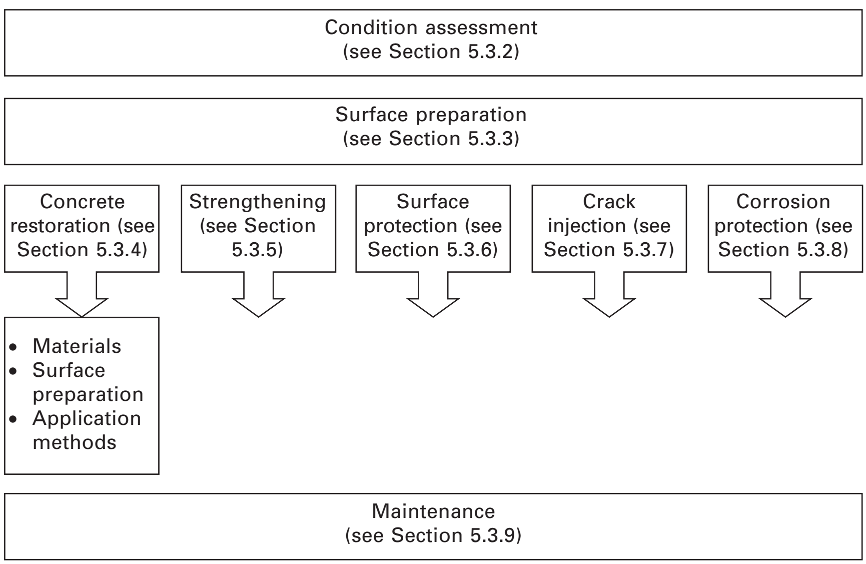
application of surface protection systems, but also for crack injection, strengthening or corrosion protection. Table 5.3 gives an overview.

5.3.4 Concrete restoration

Concrete restoration is one basic part of all repair projects of reinforced concrete structures. It covers materials for repair, preparation, application methods and contractual issues. In Tables 5.4–5.8 important international regulations for concrete restoration, especially the areas of materials, shotcrete, application of mortars as well as contractual issues are summarised.



5.10 Systematic of planning according to EN1504–9.



5.11 Overview of the different topics of regulations within the following sections.

5.3.5 Strengthening

For strengthening, nowadays externally bonded fibre-reinforced polymer (FRP) systems or the application of shotcrete are mainly used. The topic of shotcrete is part of Section 5.3.4, therefore Table 5.9 mainly focuses on the application of externally bonded FRP systems to reinforced concrete elements.

Table 5.2 Overview of international guidelines and recommendations for condition evaluation

Notification	Name	Date of issue	Country
ACI 201.1R-92	Guide for making a Condition Survey of Concrete in Service	1997	USA
ACI 224.1R-93	Causes, Evaluation and Repair of Cracks in Concrete Structures	1998	USA
ACI 228.2R-98	Non-destructive Test Methods for Evaluation of Concrete in Structures	1998	USA
ACI 364.1R-07	Guide for Evaluation of Concrete Structures before Rehabilitation	2007	USA
ACI 437R-03	Strength Evaluation of Existing Concrete Buildings	2003	USA
ICRI Guideline No. 03736	Guide for the Evaluation of unbonded Post-Tensioned Concrete Structures	2005	USA
CS-ES-1.1	System for Rapid Assessment of Quality of Concrete in Existing Structures	1985	USA
CS-ES-1.7	Petrographic Examination of Distress in Concrete	undated	USA
CS-ES-1.13	Site Inspection and Sampling Concrete Damaged by Alkali–Silica Reaction	1996	USA
AIJ	Recommendation for practice of survey, diagnosis and repair for deterioration of reinforced concrete structures	1997	Japan
RILEM Report 36 TC 184-IFE	Industrial Floors, State-of-the-Art Report Evaluation and Quality Assessment	2006	RILEM
RILEM TC-154-EMC	Recommendations of: "Electrochemical techniques for measuring metallic corrosion." Test methods for on-site corrosion rate measurement of steel reinforcement in concrete by means of the polarisation resistance method	2004	RILEM
RILEM TC 43-CND	Draft recommendation for *in situ* concrete strength determination by combined non-destructive methods	1993	RILEM
CEB Bulletin 243	Strategies for Testing and Assessment of Concrete Structures affected by Reinforcement Corrosion	1998	Switzerland
BRE Digest 434	Corrosion of Reinforcement in Concrete: Electrochemical Monitoring	1998	UK
BRE Digest 444	Corrosion of Steel in Concrete Part 1: Durability of Reinforced Concrete Structures Part 2: Investigation and Assessment	2000	UK

Table 5.3 Overview of international guidelines for repair materials in the field of surface preparation

Notification	Name	Date of issue	Country
ICRI Guideline No. 03730	Guide for Surface Preparation for the Repair of Deteriorated Concrete Resulting from Reinforcing Steel Corrosion	1995	USA
CS-MR-1.14	Concrete Removal Techniques: Selection	1994	USA
CS-MR-2.1	Concrete Surface Preparation Prior to Repair	undated	USA

Table 5.4 Overview of international guidelines for materials for repair in the field of concrete restoration

Notification	Name	Date of issue	Country
ACI 530R-93	Use Epoxy Compounds with Concrete	1998	USA
ACI 530.5R-92	Guide for the Selection of Polymer	1997	USA
ACI 546.1R-80	Guide for Repair of Concrete Bridge Superstructures	1980	USA
ACI 546.3R-06	Guide for the Selection of Materials for the Repair of Concrete	2006	USA
ICRI Guideline No. 03733	Guide for Selecting and Specifying Materials for Repair of Concrete Surfaces	1996	USA

Table 5.5 Overview of international guidelines for the use of shotcrete in the field of concrete restoration

Notification	Name	Date of issue	Country
ACI 506.2-95	Specification for Shotcrete	1995	USA
SCA-UK	An Introduction to Sprayed Concrete	1999	UK
ACI 506.4R-94	Guide for the Evaluation of Shotcrete	1994	USA
ACI 506R-90	Guide to Shotcrete	1995	USA

5.3.6 Surface protection

An overview on international regulations for surface protection of concrete parts is given in Table 5.10.

Table 5.6 Overview of international guidelines for application of mortars in the field of concrete restoration

Notification	Name	Date of issue	Country
ACI 503.4-92	Standard Specification for Repairing Concrete with Epoxy Mortars	1997	USA
ACPA TB-020.02P	The Concrete Pavement Restoration Guide: Procedures for Preserving Concrete Pavements	1998	USA
ACPA TB-002.02P	Concrete Paving Technology – Guidelines for Full Depth Repair	1995	USA
ACPA TB-003.02P	Concrete Paving Technology – Guidelines for Partial Depth Spall Repair	1998	USA
ACPA TB-005P	Technical Bulletin – Guidelines for Unbonded Concrete Overlays	1990	USA
ACPA TB-008.01P	Diamond Grinding and Concrete Pavement Restoration	2000	USA
ACPA TB-007P	Technical Bulletin – Guidelines for Concrete Bonded Overlays	1990	USA

Table 5.7 Overview of international guidelines for application methods in the field of concrete restoration

Notification	Name	Date of issue	Country
ICRI Guideline No 03731	Guide for Selecting Application Methods for the Repair of Concrete Surfaces	1996	USA
ACPA TB-007P	Technical Bulletin – Guidelines for Concrete Bonded Overlays	1990	USA

Table 5.8 Overview of international guidelines for contractual issues in the field of concrete restoration

Notification	Name	Date of issue	Country
CRA	Standard Method of Measurement for Concrete Repair	1997	UK
CS TR 38	Patch Repair of Reinforced Concrete Subject to Reinforcement Corrosion	1991	USA
ICRI Guideline No. 03735	Guide for Methods of Measurement and Contract Types for Concrete Repair Work	2000	USA

Table 5.9 Overview of international guidelines for strengthening

Notification	Name	Date of issue	Country
ACPA TB-018P	Concrete Paving Technology–Slab Stabilization Guidelines for Concrete Pavements	1994	USA
ACI 440.2R-02	Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening concrete Structures	2002	USA
CNR-DT-200	Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Existing Structures – Materials, RC and PC structures, masonry structures	2004	Italy
fib bulletin 35	Retrofitting of concrete structures by externally bonded FRPS, with emphasis on seismic applications	2006	Switzerland
fib bulletin 40	FRP reinforcement in RC structures	2007	Switzerland

Table 5.10 Overview of international guidelines for surface protection

Notification	Name	Date of issue	Country
CS TR 50	Guide to Surface Treatments for Protection and Enhancement of Concrete	1997	UK
ICRI Guideline No. 03732	Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings and Polymer Overlays	1997	USA
AASHTO	Guide Specifications for Polymer Concrete Bridge Deck Overlays	1995	USA

5.3.7 Crack injection

The purpose of a crack injection can be to strengthen the structure, either by providing a barrier against aggressive agents or by ensuring the functionality by sealing existing cracks. In order to ensure a successful and durable crack injection, both the cause of the crack and the annual change of the crack width have to be taken into account when choosing an appropriate crack grout. Helpful international regulations for crack injection of concrete parts are listed in Table 5.11.

5.3.8 Corrosion protection of the reinforcement

Concrete normally provides the steel reinforcement with an alkaline corrosion protection. Nevertheless, ingress of chlorides or carbonation may lead to

Table 5.11 Overview of international guidelines for crack injection

Notification	Name	Date of issue	Country
ACI 224.1R-93	Causes, Evaluation and Repair of Cracks in Concrete Structures	1998	USA
ICRI Guideline	Guide for Verifying Field Performance of Epoxy Injection of Concrete Cracks	1998	USA
ACI 504R-90	Guide to Sealing Joints in Concrete Structures	1997	USA
ACPA TB-012P	Concrete Paving Technology–Joint and Crack Sealing and Repair for Concrete Pavements	1995	USA
CS-MR-3.1	Selection of a Crack Repair Method	1985	USA
CS-MR-3.2	Crack Repair Method: Routing and Sealing	1985	USA
CS-MR-3.3	Crack Repair Method: Drilling and Plugging	1985	USA
CS-MR-3.4	Crack Repair Method: External Stressing	1985	USA
CS-MR-3.5	Crack Repair Method: Stitching	1985	USA
CS-MR-3.6	Crack Repair Method: Conventional Reinforcement	1985	USA
CS-MR-3.7	Crack Repair Method: Grouting (Portland-Cement and Chemical)	1985	USA
CS-MR-3.8	Crack Repair Method: Drypacking	1991	USA
CS-MR-3.9	Crack Repair Method: Epoxy Injection	1985	USA
CS-MR-3.10	Crack Repair Method: Flexible Sealing or Mastic Filling	undated	USA
CS-MR-3.11	Crack Repair Method: Polymer Impregnation	1991	USA
CS-MR-3.12	Hydrostatic Tests of Injection Ports Used for In Situ Repair of Concrete	undated	USA

initiation of reinforcement corrosion. Based on the cause of reinforcement corrosion, existence of chlorides or carbonation of the surrounding concrete, different electrochemical, physical or chemical principles exist to stop the corrosion process and to repair corrosion damaged concrete parts. The regulations listed in Table 5.12 may help to understand the typical processes of reinforcement corrosion as well as acting as a guide for selection of suited corrosion protection or repair methods for reinforced concrete parts.

5.3.9 Maintenance

Maintenance is important for new as well as for repaired structures to ensure the functionality over the whole life-cycle. Continuous maintenance helps

Table 5.12 Overview of international guidelines for corrosion management

Notification	Name	Date of issue	Country
ACI 222R-01	Protection of Metals in Concrete Against Corrosion	2001	USA
BRE-Digest 444	Corrosion of Steel in Concrete Part 3: Protection and Remediation	2000	UK
CS TR 36	Cathodic Protection of Reinforced Concrete	1989	USA
CS TR 37	Model Specification for Cathodic Protection of Reinforced Concrete	1991	USA
CPA Monograph No. 2	An Introduction to Electrochemical Rehabilitation Techniques	1998	UK
CPA Monograph No. 4	Monitoring and Maintenance of Conductive Coating Anode Cathodic Protection Systems	1999	UK
CPA Monograph No. 6	The Principles and Practice of Galvanic Cathodic Protection for Reinforced Concrete Structures	2000	UK
RILEM Technical Recommendation 124-SRC	"Strategies for repair of concrete structures damaged by steel corrosion"	1994	RILEM

Table 5.13 Overview of international guidelines for maintenance

Notification	Name	Date of issue	Country
ACI 362.2R-00	Guide for Structural Maintenance of Parking Structures	2000	USA
CPA Monograph No. 4	Monitoring and Maintenance of Conductive Coating Anode Cathodic Protection Systems	1999	UK

to detect defects at an early stage avoiding cost and time-extensive repair work. A short overview of existing regulations for maintenance of reinforced concrete structures is given in Table 5.13.

5.3.10 Outlook

Subcommittee SC 7 'Maintenance and repair of concrete structures' of Technical committee TC 71 'Concrete, reinforced concrete and pre-stressed concrete' of the ISO (International Organization for Standardization) is currently working on an international standard regarding the protection and repair of concrete structures. This standard will cover the whole area of general principles, condition assessment and design of repair as well as the execution of the repair. The publication of this standard is planned for the end of 2010.

5.4 References

EN 1504-1 2005-7. Products and systems for the protection and repair of concrete structures – Definitions, requirements, quality control and evaluation of conformity – Part 1: Definitions, British Standards Institution, London UK.

EN 1504-2 2004-10. Products and systems for the protection and repair of concrete structures – Definitions, requirements, quality control and evaluation of conformity - Part 2: Surface protection systems for concrete, British Standards Institution, London UK.

DIN EN 1504-3 2005-12 Products and systems for the protection and repair of concrete structures – Definitions, requirements, quality control and evaluation of conformity - Part 3: Structural and non-structural repair, Deutsches Institut für Normung, Berlin, Germany.

EN 1504-4 2004-11. Products and systems for the protection and repair of concrete structures – Definitions, requirements, quality control and evaluation of conformity – Part 4: Structural bonding, British Standards Institution, London UK.

EN 1504-5 2004-12. Products and systems for the protection and repair of concrete structures – Definitions, requirements, quality control and evaluation of conformity – Part 5: Concrete injection, British Standards Institution, London UK.

EN 1504-6 2006-08. Products and systems for the protection and repair of concrete structures – Definitions, requirements, quality control and evaluation of conformity – Part 6: Anchoring of reinforcing steel bar, British Standards Institution, London UK.

EN 1504-7 2006-06. Products and systems for the protection and repair of concrete structures – Definitions, requirements, quality control and evaluation of conformity – Part 7: Reinforcement corrosion protection, British Standards Institution, London UK.

EN 1504-8 2004-11. Products and systems for the protection and repair of concrete structures – Definitions, requirements, quality control and evaluation of conformity – Part 8: Quality control and evaluation of conformity, British Standards Institution, London UK.

prEN 1504-9 2007-12. Products and systems for the protection and repair of concrete structures – Definitions, requirements, quality control and evaluation of conformity – Part 9: General principles for the use of products and systems, British Standards Institution, London UK.

EN 1504-10 2003-12. Products and systems for the protection and repair of concrete structures – Definitions, Requirements, Quality control and evaluation of conformity – Part 10: Site application of products and systems and quality control of the works, British Standards Institution, London UK.

RILEM TC 124-SRC: Draft recommendation for repair strategies for concrete structures damaged by reinforcement corrosion. RILEM Technical Committees 1994, Ritel Publications Sarl, Bagneux, France.

5.5 Appendix

Table A1 International standards for condition evaluation

Notification	Name	Date of issue	Country
ASTM A 370	Standard Test Methods and Definitions for Mechanical Testing of Steel Products	2007	USA
ASTM A 751	Standard Test Methods, Practices, and Terminology for Chemical Analysis of Steel Products	2007	USA
ASTM C 39	Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens	2005	USA
ASTM C 42	Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete	2004	USA
ASTM C 295	Standard Guide for Petrographic Examination of Aggregates for Concrete	2003	USA
ASTM C 341	Standard Practice for Length Change of Cast, Drilled, or Sawed Specimens of Hydraulic-Cement Mortar and Concrete	2006	USA
ASTM C 457	Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete	2008	USA
ASTM C 469	Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression	2002	USA
ASTM C 496	Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens	2004	USA
ASTM C 597	Standard Test Method for Pulse Velocity Through Concrete	2002	USA
ASTM C 642	Standard Test Method for Density, Absorption, and Voids in Hardened Concrete	2006	USA
ASTM C 666	Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing	2003	USA
ASTM C 672	Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals	2003	USA
ASTM C 803	Standard Test Method for Penetration Resistance of Hardened Concrete	2003	USA
ASTM C 805	Standard Test Method for Rebound Number of Hardened Concrete	2002	USA

Table A1 Cont'd

Notification	Name	Date of issue	Country
ASTM C 806	Standard Test Method for Restrained Expansion of Expansive Cement Mortar	2004	USA
ASTM C 823	Standard Practice for Examination and Sampling of Hardened Concrete in Constructions	2007	USA
ASTM C 845	Standard Specification for Expansive Hydraulic Cement	2004	USA
ASTM C 856	Standard Practice for Petrographic Examination of Hardened Concrete	2004	USA
ASTM C 876	Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete	1999	USA
ASTM C 900	Standard Test Method for Pullout Strength of Hardened Concrete	2006	USA
ASTM C 1040	Standard Test Methods for In-Place Density of Unhardened and Hardened Concrete, Including Roller Compacted Concrete, By Nuclear Methods	2005	USA
ASTM C 1152	Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete	2004	USA
ASTM C 1202	Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration	2007	USA
ASTM C 1218	Standard Test Method for Water-Soluble Chloride in Mortar and Concrete	2008	USA
ASTM C 1383	Standard Test Method for Measuring the P-Wave Speed and the Thickness of Concrete Plates Using the Impact-Echo Method	2004	USA
ASTM D 4258	Standard Practice for Surface Cleaning Concrete for Coating	2005	USA
ASTM D 4262	Standard Test Method for pH of Chemically Cleaned or Etched Concrete Surfaces	2005	USA
ASTM D 4263	Standard Test Method for Indicating Moisture in Concrete by the Plastic Sheet Method	2005	USA
ASTM D 4541	Standard Test Method for Pull-Off Strength of Coatings Using Portable Adhesion Testers	2002	USA
ASTM D 4580	Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding	2007	USA

Table A1 Cont'd

Notification	Name	Date of issue	Country
ASTM D 4748	Standard Test Method for Determining the Thickness of Bound Pavement Layers Using Short-Pulse Radar	2006	USA
ASTM D 4788	Standard Test Method for Detecting Delaminations in Bridge Decks Using Infrared Thermography	2007	USA
ASTM E 84	Standard Test Method for Surface Burning Characteristics of Building Materials	2008	USA
ASTM E 105	Standard Practice for Probability Sampling Of Materials	2004	USA
ASTM E 122	Standard Practice for Calculating Sample Size to Estimate, With Specified Precision, the Average for a Characteristic of a Lot or Process	2007	USA
ASTM G 57	Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method	2006	USA

Table A2 International standards for concrete restoration

Notification	Name	Date of issue	Country
ASTM C 1116	Standard Specification for Fibre-Reinforced Concrete	2008	USA
ASTM C 1140	Standard Practice for Preparing and Testing Specimens from Shotcrete Test Panels	2003	USA
ASTM C 1141	Standard Specification for Admixtures for Shotcrete	2006	USA
Products and systems for the protection and repair of concrete structures – structural and non–structural repair according to EN 1504-3			
Test methods – Determination of compressive strength of repair mortar	EN 12190	1998	Europe
Measurement of bond strength by pull-off	EN 1542	1999	
Determination of shrinkage and expansion	EN 12617-4	2002	
Determination of resistance to carbonation	EN 13295	2004	

Table A2 Cont'd

Notification	Name	Date of issue	Country
Determination of modulus of elasticity in compression	EN 13412	2006	
Part 1: Freeze–thaw cycling with de-icing salt immersion Part 2: Thunder–shower cycling (thermal shock) Part 4: Determination of thermal compatibility	EN 13687-1, 2, 4	2002	
Method for measurement of slip/skid resistance of a surface: The pendulum test	EN 13036-4	2003	
Determination of the coefficient of thermal expansion	EN 1770	1998	
Determination of resistance of capillary absorption	EN 13057	2002	
Measurement of chloride ion ingress	EN 13396	2004	
Determination of creep in compression for repair products	EN 13584	2003	
Resistance to severe chemical attack	EN 13529	2003	

Table A3 International standards for strengthening

Notification	Name	Date of issue	Country
ASTM C 881	Standard Specification for Epoxy-Resin-Base Bonding Systems for Concrete	2002	USA
ASTM C 1059	Standard Specification for Latex Agents for Bonding Fresh To Hardened Concrete	1999	USA
ASTM D 3165	Standard Test Method for Strength Properties of Adhesives in Shear by Tension Loading of Single-Lap-Joint Laminated Assemblies	2007	USA
ASTM D 3528	Standard Test Method for Strength Properties of Double Lap Shear Adhesive Joints by Tension Loading	2002	USA

Table A3 Cont'd

Notification	Name	Date of issue	Country
Structural bonding according to DIN EN 1504-4			
Determination of compressive strength of repair mortar	EN 12190	1998	Europe
Determination of flexural properties	EN ISO 178	2003	
Determination of adhesion steel-to-steel for characterisation of structural bonding agents	EN 12188	1999	
Determination of open time	EN 12189	1999	
Determination of modulus of elasticity in compression	EN 13412	2006	
Determination of glass transition temperatures of polymers	EN 12614	2004	
Determination of the coefficient of thermal expansion	EN 1770	1998	
Determination of linear shrinkage for polymers and surface protection systems (SPS)	EN 12617-1	2003	
Determination of early age linear shrinkage for structural bonding agent	EN 12617-3	2002	
Adhesion by tensile bond strength	EN 12618-2	2004	
Determination of the durability	EN 13733	2002	
of structural bonding			
Determination of slant shear strength	EN 12615	1999	
Determination of the adhesion of injection products, with or without thermal cycling.	EN 12618-2	2004	
Determination of adhesion concrete to concrete	EN 12636	1999	
Determination of slant shear strength	EN 12615	1999	
Determination of fatigue under dynamic loading .	EN 13894-1, 2	2002/03	
Part 1: During cure			
Part 2: After hardening			

Table A3 Cont'd

Notification	Name	Date of issue	Country
Anchoring of reinforcing steel bar according to DIN EN 1504-6			
Testing of anchoring products by the pull-out method	EN 1881	2006	Europe
Determination of glass transition temperatures of polymers	EN 12614	2004	
Determination of creep under sustained tensile load for synthetic resin products (PC) for the anchoring of reinforcing bars	EN 1544	2006	

Table A4 International standards for surface protection

Notification	Name	Date of issue	Country
Surface protection systems for concrete according to EN 1504-2			
Determination of indentation hardness by means of a durometer (Shore hardness)	EN ISO 868	2003	Europe
Determination of compressive strength of repair mortar	EN 12190	1998	
Determination of linear shrinkage for polymers and surface protection systems (SPS)	EN 12617-1	2003	
Determination of the coefficient of thermal expansion	DIN EN 1770	1998	
Taber abrader	EN ISO 5470-1	1999	
Cross-cut test	EN ISO 2409	2007	
Determination of carbon dioxide permeability	EN 1062-6	2002	
Part 1: Determination of water-vapour transmission rate	EN ISO 7783-1, 2	1999	
Part 2: Determination and classification of water-vapour transmission rate			
Determination of liquid water permeability	EN 1062-3	2008	
Freeze–thaw cycling with de-icing salt immersion	EN 13687-1	2002	
Thunder-shower cycling	EN 13687-2	2002	
Thermal cycling without de-icing salt impact	EN 13687-3	2002	
Methods of conditioning before testing	EN 1062-11	2002	

Table A4 Cont'd

Notification	Name	Date of issue	Country
Resistance to temperature shock	EN 13687-5	2002	
Immersion in liquids other than water	EN ISO 2812-1	2007	
Resistance to severe chemical attack	EN 13529	2003	
Determination of crack bridging properties	EN 1062-7	2004	
Fallingweight test, large area indenter	EN ISO 6272-1	2004	
Measurement of bond strength by pull-off	EN 1542	1999	
Classification using data from reaction to fire tests	EN 13501-1		
Determination of loss of mass of hydrophobic impregnated concrete after freeze–thaw salt test	EN 13581	2007	
Method for measurement of slip/skid resistance of a surface: The pendulum test	EN 13036-4	2003	
Drying test for hydrophobic impregnation	EN 13579	2002	
Surface protection systems for concrete	EN 1504-2	2004	
Methods of conditioning before testing	EN 1062-11	2002	
Determination of the electrical resistance	EN 1081	1998	
Compatibility on wet concrete;	EN 13578	2003	
Water absorption and resistance to alkali for hydrophobic impregnations	EN 13580	2002	
Evaluation of degradation of coatings	EN ISO 4628	2003	
Determination of film thickness	EN ISO 2808	2007	

Table A5 International standards for crack injection

Notification	Name	Date of issue	Country
Concrete injection according to EN 1504-5			
Plastics – Urea Formaldehyde and urea/melamine-formaldehyde powder mouldering compounds	EN ISO 527-1, DIN EN ISO 527-2	2000	Europe
Determination of compressive strength of repair mortar	EN 12190	1998	
Determination of the adhesion of injection products, with or without thermal recycling – Adhesion by tensile bond strength	EN 12618-2	2004	
Determination of the adhesion of injection products, with or without thermal recycling – Slant shear method	EN 12618-3	2004	

Table A5 Cont'd

Notification	Name	Date of issue	Country
Adhesion and elongation capacity of injection products with limited ductility	EN 12618-1	2003	
Shrinkage of crack injection products based on polymer binder: volumetric shrinkage	EN 12617-2	2004	
Determination of glass transition temperatures of polymers	EN 12614	2004	
Determination of tensile strength development for polymers	EN 1543	1998	
Determination of water tightness of injected cracks without movement in concrete	EN 14068	2003	
Compatibility with concrete	EN 12637-1	2004	
Effect of injection products on elastomers	EN 12637-3	2003	
Volume and weight changes of injection products after air drying and water storage cycles	EN 14498	2004	
Determination of temperature and enthalpy of melting and crystallisation	ISO 11357-3	1999	
Determination of thermal compatibility – Thermal cycling without de-icing salt impact	EN 13687-3	2002	
Determination of thermal compatibility – Resistance to temperature shock	EN 13687-5	2002	

Table A6 International standards for corrosion protection

Notification	Name	Date of issue	Country
ASTM A 706	Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement	2006	USA
ASTM A 775	Standard Specification for Epoxy-Coated Steel Reinforcing Bars	2007	USA
ASTM G 57	Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method	2006	USA
EN 12696	Cathodic Protection of Steel in Concrete	2000	Europe

Table A6 Cont'd

Notification	Name	Date of issue	Country
Reinforcement corrosion protection according to EN 1504-7			
Determination of indentation hardness by means of a durometer (Shore hardness)	EN ISO 868	2003	Europe
Corrosion protection test	EN 15183	2006	
Determination of glass transition temperatures of polymers	EN 12614	2004	
Shear adhesion of coated steel to concrete (pull-out test)	EN 15184	2006	

6
Methods of crack repair in concrete structures

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Abstract: Cracks in concrete have many causes. They may affect appearance only, or they may indicate significant structural distress or a lack of durability. Cracks may represent the total extent of the damage, or they may point to problems of greater magnitude. Their significance depends on the type of structure, as well as the nature of the cracking. For example, cracks that are acceptable for buildings may not be acceptable in water-retaining structures. The proper repair of cracks depends on knowing the causes and selecting the repair procedures that take these causes into account; otherwise, the repair may be only temporary. Successful long-term repair procedures must attack the causes of the cracks as well as the cracks themselves.

Key words: crack repair, method selection, crack identification, method description.

6.1 Crack repair methods selection

6.1.1 Introduction

Based on the careful evaluation of the extent and cause of cracking, procedures can be selected to accomplish one or more of the following objectives:

1. restore and increase strength;
2. restore and increase stiffness;
3. improve functional performance;
4. provide watertightness;
5. improve appearance of the concrete surface;
6. improve durability;
7. prevent development of a corrosive environment at the reinforcement.

Depending on the nature of the damage, one or more repair methods may be selected. For example, tensile strength may be restored across a crack by injecting it with epoxy or other high-strength bonding agent. However, it may be necessary to provide additional strength by adding reinforcement or by using post-tensioning. Epoxy injection alone can be used to restore flexural stiffness if further cracking is not anticipated (ACI Committee 503R).

Cracks causing leaks in water-retaining or other storage structures should be repaired unless the leakage is considered minor or unless there is an indication that the crack is being sealed by autogenous healing. Repairs to stop leaks may be complicated by a need to make the repairs while the structures are in service.

Cosmetic considerations may require the repair of cracks in concrete. However, the crack locations may still be visible and it is likely that some form of coating over the entire surface may be required. To minimize future deterioration due to the corrosion of reinforcement, cracks exposed to a moist or corrosive environment should be sealed.

6.1.2 Procedure for identifying causes of cracks

Successful repair of cracks in concrete depends upon identifying the cause or causes of the cracks and selecting a repair method or methods that will take the cause into account. Conditions that can cause cracks are listed in Table 6.1, along with a general guide to whether the crack being considered will remain 'active' or 'dormant'. Active cracks are defined as those for which the mechanism causing the cracking is still at work. (Any crack for which an exact cause cannot be determined is considered to be active.) Dormant cracks are those which were caused by a condition which is not expected to recur. The steps outlined below will assist in identifying the cause of cracking.

Step 1: Examine the appearance and the depth of the cracking to establish the basic nature of the occurrence:

1. Pattern or individual cracks?
2. Depth of cracks?
3. Open or closed cracks?
4. Extent of cracking?

Step 2: Determine when the cracking occurred. This step wi1l require talking with the individuals who operate and maintain the structure and possibly with those involved in the construction.

Step 3: Determine if the cracks are active or dormant. This step may require monitoring the cracks for a period of time to determine if crack movement is taking place. Also, attempt to determine if the crack movement detected is growth or simply cyclical opening and closing such as caused by thermal expansion. Cracks which are moving but not growing should be treated as active cracks.

Step 4: Determine the degree of restraint. This step will require a thorough examination of the structure and the construction drawings, if available. Both internal restraint (caused by reinforcing steel, embedded items, etc.) and external restraint (caused by foundation conditions, bonding to other concrete or adjacent structures) must be considered.

Table 6.1 Conditions that cause cracks in concrete

Cause	Type of crack		Comment
	Active	Dormant	
Accidental loading		×	
Design error (inadequate reinforcement)	×		Limit loading according to current capacity and repair, or redesign and repair as indicated by the redesign
Temperature stresses (excessive expansion due to elevated temperature and inadequate expansion joints)	×		It may be desirable to redesign to include adequate expansion joints
Corrosion of reinforcing steel	×		Simple crack repair methods should not be used as the steel will continue to corrode and crack the concrete
Foundation settlement	×	×	Measurements must be made to determine if the foundation is still settling
Alkali–aggregate reaction	×		Concrete will continue to deteriorate as long as moisture is present. Crack repair methods will be ineffective
Poor construction procedure (inadequate curing, formwork, etc.)		×	
Design faults –use of exposed rigidly connected material to concrete which has a significantly different modulus of expansion –stress concentrations –faulty joint systems	×		

Note: This listing is intended to serve as a general guide only. It should be recognized that there will be exceptions to all of the items listed.

Step 5: Determine the cause of the cracking using the information gained in Steps 1 through 4 and the guidance presented in Table 6.1. A checklist for determining the cause of cracking is presented below. Using the checklist, eliminate as many potential causes as possible. If more than one potential cause remains, the final determination may require a laboratory analysis of concrete samples or a detailed stress analysis.

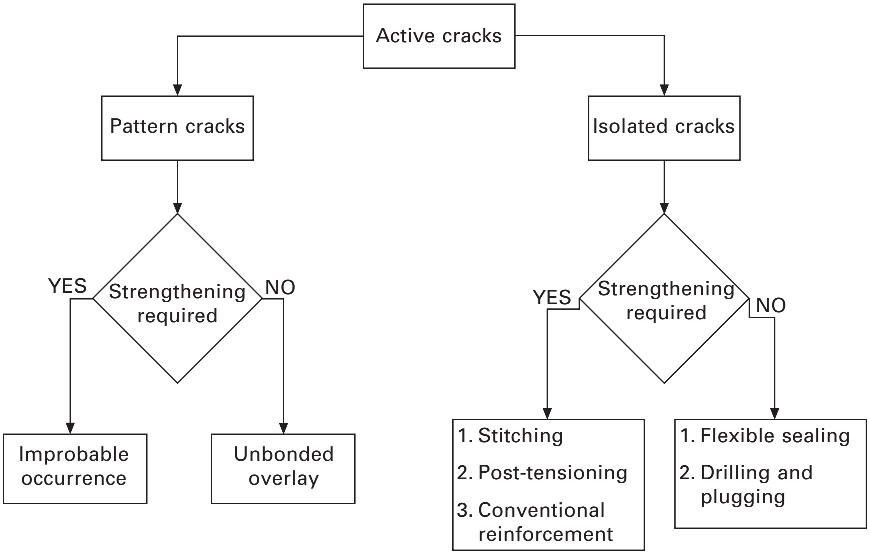
1. Check for major errors in design.
2. Check easily identifiable causes:
   (a) corrosion of reinforcement;
   (b) accidental or impact loading;
   (c) poor design detailing;
   (d) foundation movement.
3. Check other possible causes:
   (a) incidents during construction;
   (b) shrinkage-induced stresses;
   (c) temperature-induced stresses:
      (i) during hydration
      (ii) post-hydration
   (d) volume changes:
      (i) chemical reactions
      (ii) moisture changes
      (iii) freezing and thawing.

6.1.3 Selecting an appropriate crack repair method

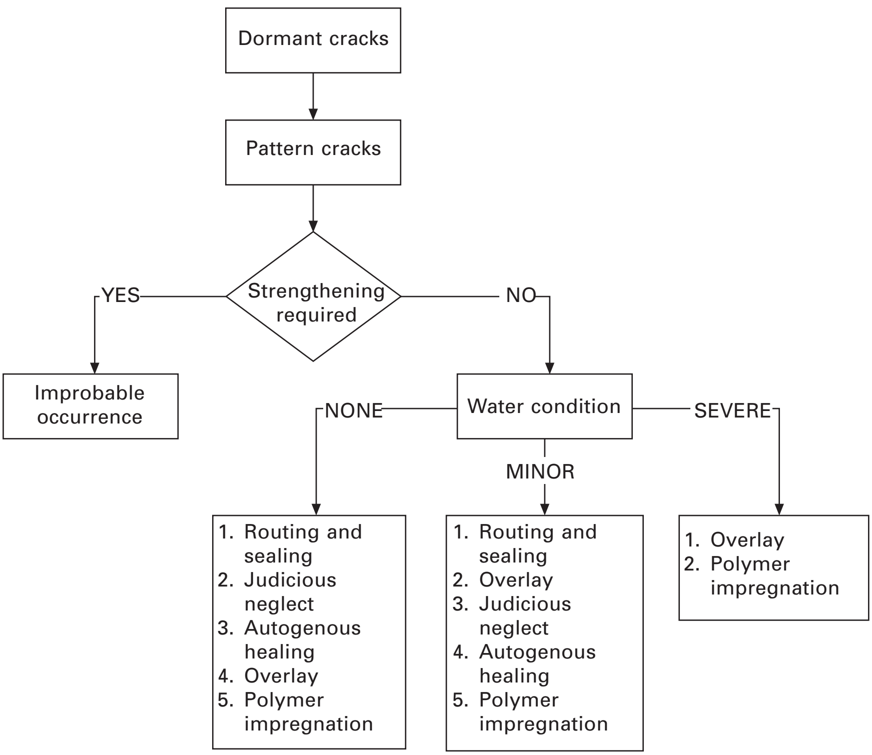
Once the cause of the cracking has been established, the following questions should be answered based upon knowledge of the cause of the crack:

- Is repair indicated? Repair of cracking caused by expansion products of internal chemical reactions may not be feasible.
- Should the repair be treated as spalling rather than cracking? If the damage is such that loss of concrete mass is probable, treatment of the cracks may not be adequate. For example, cracking due to corrosion of embedded metal or freezing and thawing would be better treated by removal and replacement of concrete than by one of the crack repair methods.
- Is it necessary that the condition causing the crack be remedied? Is doing so economically feasible?
- Is strengthening across the crack required?
- What is the moisture environment of the crack?

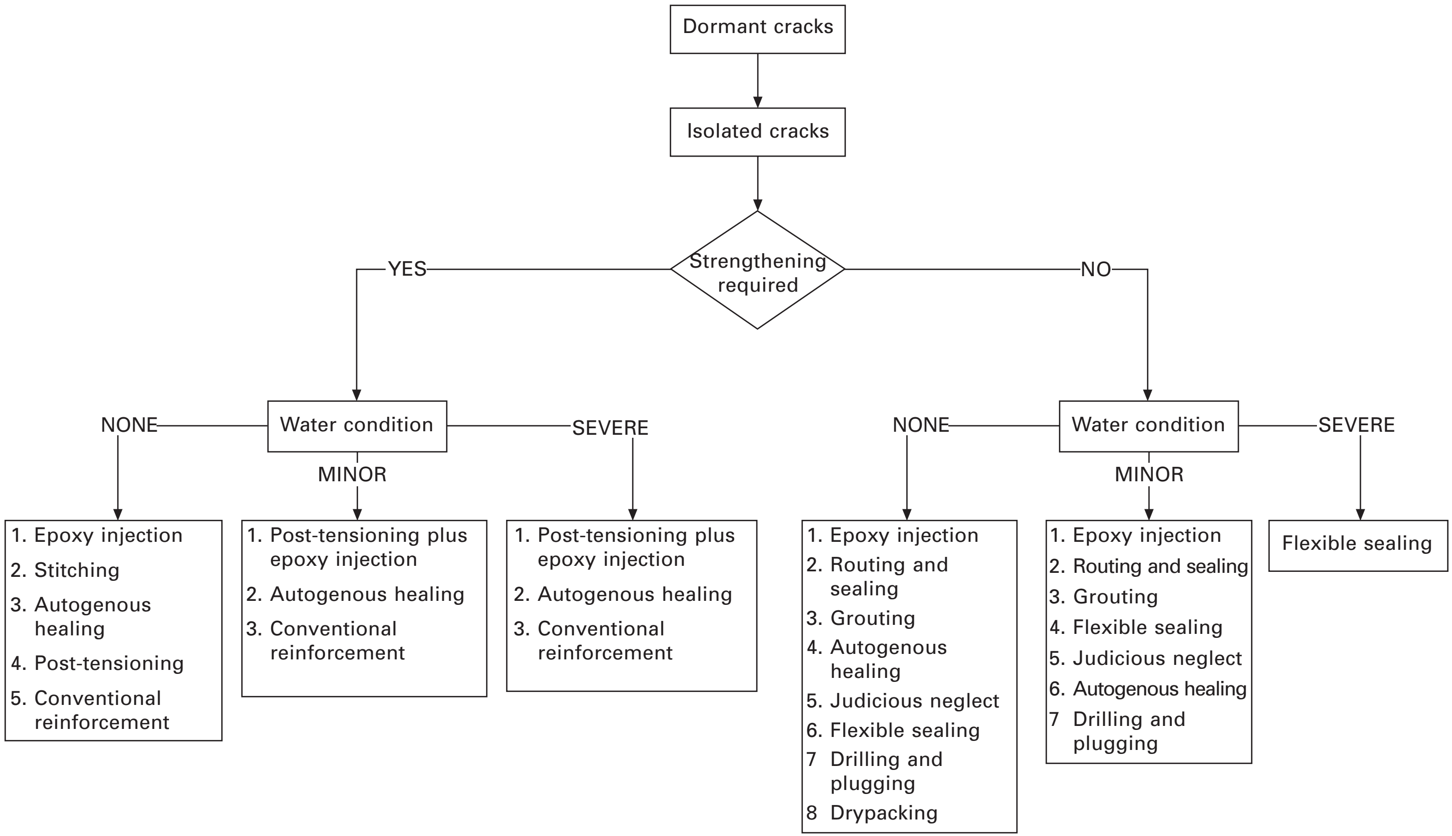
Once these questions have been answered, potential repair methods can be as outlined in Figs 6.1–6.3. The use of Figs 6.1–6.3 would lead to the consideration of several applicable crack repair methods. Final selection of a method and a repair material should take into account ease of application, durability, life-cycle cost, available labor skills and equipment, and appearance of the final product. The key methods of crack repair available to accomplish the objectives outlined are described in the section below.



6.1 Selection of repair method for active cracks (after Johnson, 1965).



6.2 Selection of repair method for dormant pattern cracks (after Johnson, 1965).



6.3 Selection of repair method for dormant isolated cracks (after Johnson, 1965).

6.1.4 Judicious neglect

In some instances, particularly for dormant cracks and cracks caused by alkali–aggregate reaction, the best treatment may be no treatment at all. If there is no moisture problem, this approach should be considered. According to Johnson (1965), dormant cracks, such as those due to shrinkage or to some inadvertent error in construction such as premature removal of the forms or settlement of the sills supporting the shores, are frequently self-healing. This does not imply an autogenous healing and gain of strength as described above, but merely that the cracks clog with dirt, grease, or oil, or perhaps a little re-crystallization occurs, and so on. The result is that the cracks are-plugged (or sealed, if you will), and problems which may have been encountered with leakage will disappear without doing any repair, particularly if leakage is due to some intermittent cause rather than to a continuing pressure head. Perhaps this is not engineering, but it works, and where there is a chance that it will occur, if aesthetic considerations permit, and if there is time to give it a try, it is by far the best answer. Judicious neglect will certainly be an attractive repair method from a cost standpoint.

6.2 Crack repair methods

6.2.1 Introduction

Following the evaluation of the cracked structure, a suitable repair procedure can be selected. Successful repair procedures take into account the cause(s) of the cracking. For example, if the cracking was primarily due to drying shrinkage, then it is likely that after a period of time the cracks will stabilize. On the other hand, if the cracks are due to a continuing foundation settlement, repair will be of no use until the settlement problem is corrected.

This section provides a survey of crack repair methods, including a summary of the characteristics of the cracks that may be repaired with each procedure, the types of structures that have been repaired, and a description of the procedures that are used.

6.2.2 Epoxy injection

Cracks as narrow as 0.002 in. (0.05mm) can be bonded by the injection of epoxy. The technique generally consists of establishing entry and venting ports at close intervals along the cracks, sealing the crack on exposed surfaces, and injecting the epoxy under pressure.

Epoxy injection has been successfully used in the repair of cracks in buildings, bridges, dams, and other types of concrete structures (ACI Committee 503R). However, unless the cause of the cracking has been corrected, it will probably recur near the original crack. If the cause of the cracks cannot be removed, then two options are available. One is to rout and seal the crack, thus treating it as a joint. The second is to establish a joint that will accommodate the movement and then inject the crack with epoxy or other suitable material. Epoxy materials used for structural repairs should conform to ASTM C 881 (Type IV). ACI Committee 504R describes practices for sealing joints, including joint design, available materials, and methods of application.

With the exception of certain moisture-tolerant epoxies, this technique is not applicable if the cracks are actively leaking and cannot be dried out. Wet cracks can be injected using moisture-tolerant materials, but contaminants in the cracks (including silt and water) can reduce the effectiveness of the epoxy for structurally repairing the cracks. The use of a low-modulus, flexible adhesive in a crack will not allow significant movement of the concrete structure. The effective modulus of elasticity of a flexible adhesive in a crack is substantially the same as that of a rigid adhesive (Adams and Wake, 1984) because of the thin layer of material and high lateral restraint imposed by the surrounding concrete. Epoxy injection requires a high degree of skill for satisfactory execution, and application of the technique may be limited by the ambient temperature. The general procedures involved in epoxy injection are as follows (ACI Committee 503R).

Clean the cracks: The first step is to clean the cracks that have been contaminated, to the extent that this is possible and practical. Contaminants such as soil, grease, dirt, or fine particles of concrete prevent epoxy penetration and bonding and reduce the effectiveness of repairs. Preferably, contamination should be removed by vacuuming or flushing with water or other particularly effective cleaning solutions.

The solution is then flushed out using compressed air and a neutralizing agent or adequate time is provided for air drying. It is important, however, to recognize the practical limitations of accomplishing complete crack cleaning. A reasonable evaluation should be made of the extent, and necessity, of cleaning. Trial cleaning may be required.

Seal the surfaces: Surface cracks should be sealed to keep the epoxy from leaking out before it has gelled. Where the crack face cannot be reached, but where there is backfill, or where a slab-on-grade is being repaired, the backfill material or sub-base material is sometimes an adequate seal; however, such a condition can rarely be determined in advance, and uncontrolled injection can cause damage such as plugging a drainage system. Extreme caution must therefore be exercised when injecting cracks that are not visible on all surfaces. A surface can be sealed by applying an epoxy, polyester, or other appropriate sealing material to the surface of the crack and allowing it to harden. If a permanent glossy appearance along the crack is objectionable and if high injection pressure is not required, a strippable plastic surface sealer may be applied along the face of the crack. When the job is completed, the surface sealer can be stripped away to expose the gloss-free surface. Cementitious seals can also be used where appearance of the completed work is important. If extremely high injection pressures are needed, the crack can be cut out to a depth of 1/2 in. (13mm) and width of about 3/4 in. (20mm) in a V-shape, filled with an epoxy, and struck off flush with the surface.

Install the entry and venting ports: Typical settings for entry and venting ports are shown in Fig. 6.4. Three methods are in general use:

- ***Fittings inserted into drilled holes*** – This method was the first to be used, and is often used in conjunction with V-grooving of the cracks. The method entails drilling a hole into the crack, approximately 3/4 in. (20mm) in diameter and 1/2–1 in. (13–25mm) below the apex of the V-grooved section, into which a fitting such as a pipe nipple or tire valve stem is usually bonded with an epoxy adhesive. A vacuum chuck and bit, or a watercooled corebit, is useful in preventing the cracks from being plugged with drilling dust.
- ***Bonded flush fitting*** – When the cracks are not V-grooved, a method frequently used to provide an entry port is to bond a fitting flush with the concrete face over the crack. The flush fitting has an opening at the top for the adhesive to enter and a flange at the bottom that is bonded to the concrete.

6.4 Structure prepared for epoxy injection through the ports shown (FDOT).

- ***Interruption in seal*** – Another system of providing entry is to omit the seal from a portion of the crack. This method can be used when special gasket devices are available that cover the unsealed portion of the crack and allow injection of the adhesive directly into the crack without leaking.

Mix the epoxy: This is done either by batch or continuous methods. In batch mixing, the adhesive components are premixed according to the manufacturer's instructions, usually with the use of a mechanical stirrer, like a paint mixing paddle. Care must be taken to mix only the amount of adhesive that can be used prior to commencement of gelling of the material. When the adhesive material begins to gel, its flow characteristics begin to change, and pressure injection becomes more and more difficult. In the continuous mixing system, the two liquid adhesive components pass through metering and driving pumps prior to passing through an automatic mixing head. The continuous mixing system allows the use of fast-setting adhesives that have a short working life.

Inject the epoxy: Hydraulic pumps, paint pressure pots, or air-actuated caulking guns may be used. The pressure used for injection must be selected carefully. Increased pressure often does little to accelerate the rate of injection. In fact, the use of excessive pressure can propagate the existing cracks, causing additional damage. If the crack is vertical or inclined, the injection process should begin by pumping epoxy into the entry port at the lowest elevation until the epoxy level reaches the entry port above. The lower injection port is then capped, and the process is repeated until the crack has been completely filled and all ports have been capped. For horizontal cracks, the injection should proceed from one end of the crack to the other in the same manner. The crack is full if the pressure can be maintained. If the pressure cannot be maintained, the epoxy is still flowing into unfilled portions or leaking out of the crack.

Remove the surface seal: After the injected epoxy has cured, the surface seal should be removed by grinding or by other means as appropriate.

Alternative procedure: For massive structures, an alternative procedure consists of drilling a series of holes [usually 7/8–4 in. (20–100 mm) diameter] that intercepts the crack at a number of locations. Typically, holes are spaced at 5 ft (1.5 m) intervals.

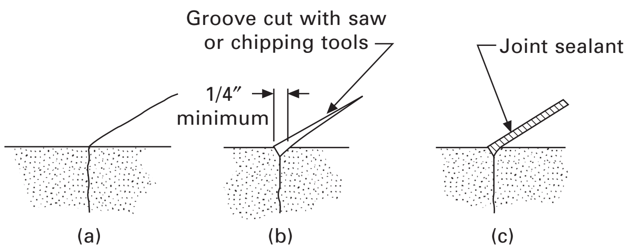
Another method recently being used is a vacuum or vacuum assist method. There are two techniques: one is to entirely enclose the cracked member with a bag and introduce the liquid adhesive at the bottom and to apply a vacuum at the top. The other technique is to inject the cracks from one side and pull a vacuum from the other.

Typically, epoxies are used; however, acrylics and polyesters have proven successful. Stratton and McCollum (1974) describe the use of epoxy injection as an effective intermediate-term repair procedure for delaminated bridge decks. As reported by Stratton and McCollum, the first, second, and sixth steps are omitted and the process is terminated at a specific location when epoxy exits from the crack at some distance from the injection ports. This procedure does not arrest on-going corrosion. The procedure can also be attempted for other applications, and is available as an option, although is not accepted universally. Success of the repair depends on the absence of bond-inhibiting contaminants from the crack plane. Epoxy resins and injection procedures should be carefully selected when attempting to inject delaminations. Unless there is sufficient depth or anchorage to surrounding concrete, the injection process can be unsuccessful or increase the extent of delamination. Smith (1992) provides information on bridge decks observed for up to seven years after injection. Smithson and Whiting (1992) describe epoxy injection as a method to rebond delaminated bridge deck overlays.

6.2.3 Routing and sealing

Routing and sealing of cracks can be used in conditions requiring remedial repair and where structural repair is not necessary. This method involves enlarging the crack along its exposed face and filling and sealing it with a suitable joint sealant (Fig. 6.5). This is a common technique for crack treatment and is relatively simple in comparison with the procedures and the training required for epoxy injection. The procedure is most applicable to relatively flat horizontal surfaces such as floors and pavements. However, routing and sealing can be accomplished on vertical surfaces with a non-sag sealant as well as on curved surfaces (pipes, piles, and poles).

Routing and sealing is used to treat both fine pattern cracks and larger, isolated cracks. A common and effective use is for waterproofing by sealing cracks on the concrete surface where water stands, or where hydrostatic pressure is applied. This treatment reduces the ability of moisture to reach



6.5 Repair of crack by sealing: (a) original crack; (b) routing; (c) sealing (after Johnson, 1965).

the reinforcing steel or pass through the concrete, causing surface stains or other problems.

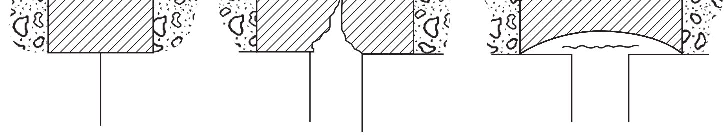
The sealants may be any of several materials, including epoxies, urethanes, silicones, polysulfides, asphaltic materials, or polymer mortars. Cement grouts should be avoided due to the likelihood of cracking. For floors, the sealant should be sufficiently rigid to support the anticipated traffic. Satisfactory sealants should be able to withstand cyclic deformations and should not be brittle.

The procedure consists of preparing a groove at the surface ranging in depth, typically, from 1/4–1 in. (6–25 mm). A concrete saw, hand tools, or pneumatic tools may be used. The groove is then cleaned by air blasting, sandblasting, or water blasting, and dried. A sealant is placed into the dry groove and allowed to cure.

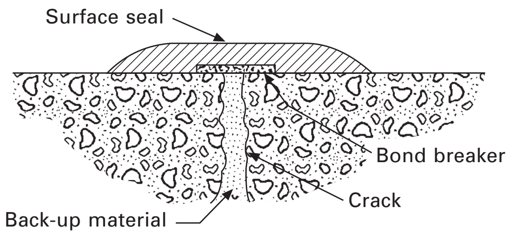
A bond breaker may be provided at the bottom of the groove to allow the sealant to change shape, without a concentration of stress on the bottom (Fig. 6.6). The bond breaker may be a polyethylene strip or tape which will not bond to the sealant. Careful attention is required when detailing the joint, so that its width to depth aspect ratio will accommodate anticipated movement (ACI Committee 504R).

In some cases, overbanding (strip coating) is used independently of, or in conjunction with, routing and sealing. This method is used to enhance protection against edge spalling and for aesthetic reasons to create a treatment with more uniform appearance. A typical procedure for overbanding is to prepare an area approximately 1–3 in. (25–75 mm) on each side of the crack by sandblasting or other means of surface preparation, and to apply a coating (such as urethane) 0.04–0.08 in. (1–2 mm) thick in a band over the crack. Before overbanding in non-traffic areas, a bond breaker is sometimes used over a crack that has not been routed, or over a crack previously routed and sealed. In traffic areas, a bond breaker is not recommended. Cracks subject to minimal movement may be overbanded (Fig. 6.7) but, if significant movement can take place, routing and sealing must be used in conjunction with overbanding to ensure a waterproof repair.

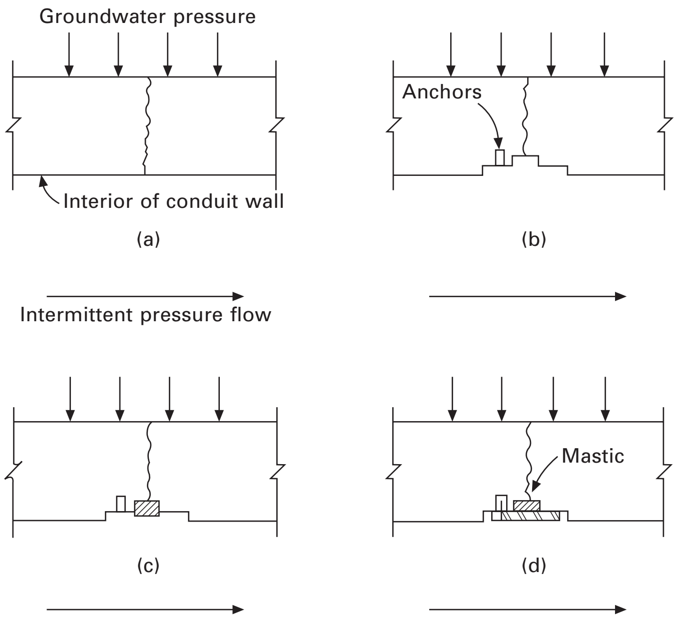
For example, when sealing canal and reservoir linings subjected to low groundwater pressure, the crack should be routed to provide a mastic area



6.6 Effect of bond breaking (ACI Committee 224.1R).



6.7 Example of surface sealing for cracks subject to movement (ACI Committee 224.1R).



6.8 Crack repair using a retaining plate to hold mastic in place against pressure: (a) crack in conduit wall; (b) cut out for plate and mastic – install expandable anchors; (c) fill with mastic; (d) fasten retainer plate on one side only to provide for movement (ACI Committee 224.IR).

(slot) that complies with the requirements for width and shape factor of a joint having equivalent movement. To maintain hydraulic efficiency in some structures, it may be necessary to cut the concrete surface adjacent to the crack and to place the retaining cap flush with the original flow lines. The crack should then be cleaned by sandblasting, air and/or water jetting. The mastic is placed into the routed crack slot and a retaining cap placed over the mastic to confine it (Fig. 6.8). A simple retainer can be made by positioning a metal strip across the crack and fastening it to expandable anchors or grouted bolts installed in the concrete along one side of the crack.

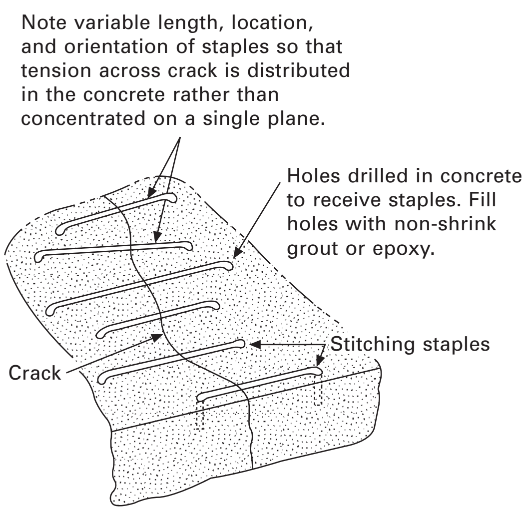
6.2.4 Stitching

Stitching involves drilling holes on both sides of the crack and grouting in inverted U-shaped metal units with short legs (staples or stitching dogs) that span the crack as shown in Fig. 6.9 (Johnson, 1965). Stitching may be used when tensile strength must be re-established across major cracks (Hoskins *et al.*, 1991). Stitching a crack tends to stiffen the structure, and the stiffening may increase the overall structural restraint, causing the concrete to crack elsewhere. Therefore, it may be necessary to strengthen the adjacent section or sections using appropriate reinforcing methods. Because stresses are often concentrated, using this method in conjunction with other methods may be necessary. The stitching procedure consists of drilling holes on both sides of the crack, cleaning the holes, and anchoring the legs of the staples in the holes, with either a non-shrink grout or an epoxy resin-based bonding system. The staples should be variable in length, orientation, or both and they should be located so that the tension transmitted across the crack is not applied to a single plane within the section but is spread over an area.

6.2.5 Additional reinforcement

Conventional reinforcement

Cracked reinforced concrete bridge girders have been successfully repaired by inserting reinforcing bars and bonding them in place with epoxy (Stratton *et al.*, 1978, 1982; Stratton, 1980). This technique consists of sealing the crack, drilling holes that intersect the crack plane at approximately 90° (Fig. 6.10),



6.9 Repair of crack by stitching (Johnson, 1965).

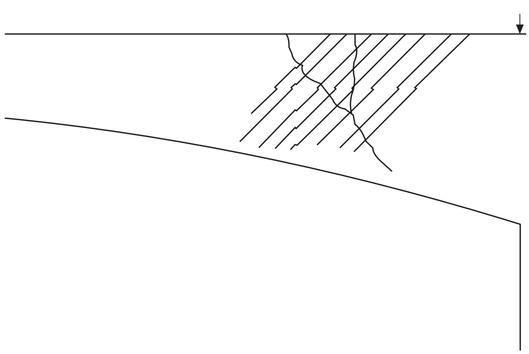
filling the hole and crack with injected epoxy, and placing a reinforcing bar into the drilled hole. Typically, No. 4 or 5 (10 M or 15 M) bars are used, extending at least 18 in. (0.5m) each side of the crack. The reinforcing bars can be spaced to suit the needs of the repair. They can be placed in any desired pattern, depending on the design criteria and the location of the in-place reinforcement. The epoxy bonds the bar to the walls of the hole, fills the crack plane, bonds the cracked concrete surfaces back together in one monolithic form, and thus reinforces the section. The epoxy used to rebond the crack should have a very low viscosity and conform to ASTM C 881 Type IV.

Prestressing steel

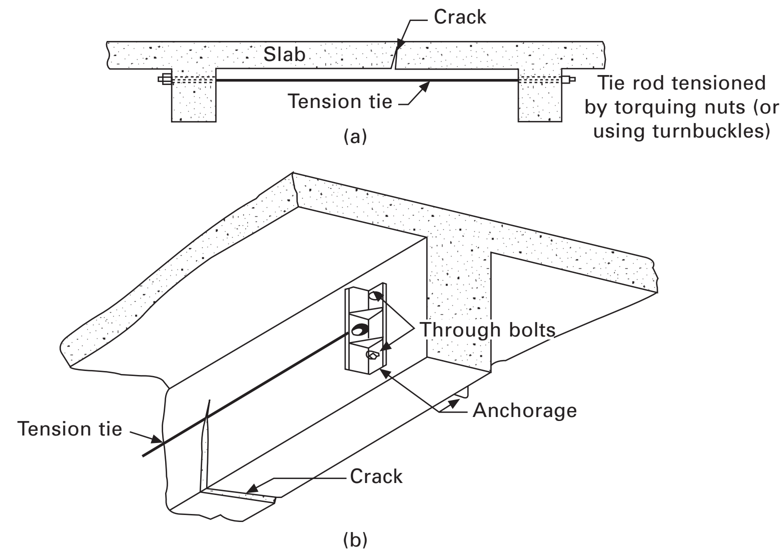
Post-tensioning is often the desirable solution when a major portion of a member must be strengthened or when the cracks that have formed must be closed (Fig. 6.11). This technique uses prestressing strands or bars to apply a compressive force. Adequate anchorage must be provided for the prestressing steel, and care is needed so that the problem will not merely migrate to another part of the structure. The effects of the tensioning force (including eccentricity) on the stress within the structure should be carefully analyzed. For indeterminate structures post-tensioned using this procedure, the effects of secondary moments and induced reactions should be considered (Lin and Burns, 1981; Nilson, 1987).

6.2.6 Drilling and plugging

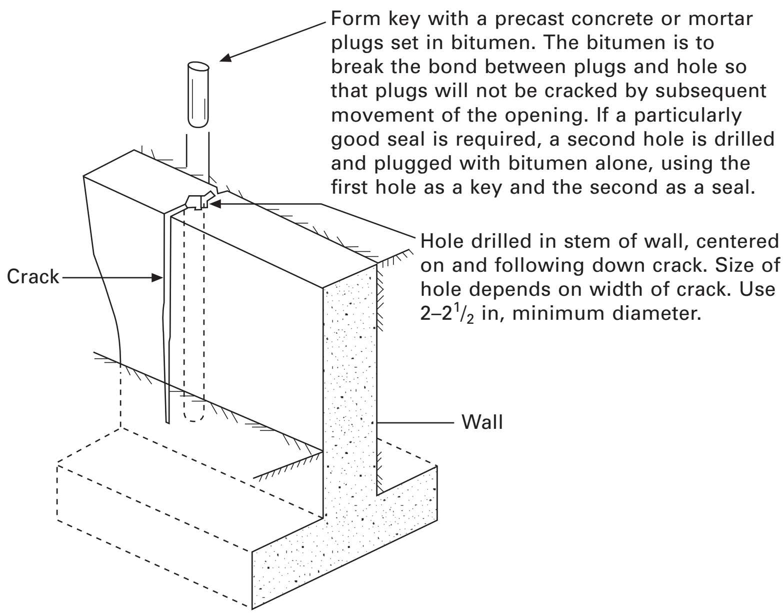
Drilling and plugging a crack consists of drilling down the length of the crack and grouting it to form a key (Fig. 6.12). This technique is only applicable when cracks run in reasonably straight lines and are accessible at one end. This method is most often used to repair vertical cracks in retaining walls. A



6.10 Reinforcing bar orientation used to effect the repair (Stratton *et al.*, 1978).



6.11 Examples of external prestressing: (a) to correct cracking of slab; (b) to correct cracking of beam (after Johnson, 1965).



6.12 Repair by drilling and plugging (ACI Committee 224.1R).

hole [typically 2–3 in. (50–75 mm) in diameter] should be drilled, centered on and following the crack. The hole must be large enough to intersect the crack along its full length and provide enough repair material to structurally take the loads exerted on the key. The drilled hole should then be cleaned, made tight, and filled with grout. The grout key prevents transverse movements of the sections of concrete adjacent to the crack. The key will also reduce heavy leakage through the crack and loss of soil from behind a leaking wall. If watertightness is essential and structural load transfer is not, the drilled hole should be filled with a resilient material of low modulus in lieu of grout. If the keying effect is essential, the resilient material can be placed in a second hole, the first being grouted.

6.2.7 Gravity filling

Low-viscosity monomers and resins can be used to seal cracks with surface widths of 0.001–0.08 in. (0.03–2 mm) by gravity filling (Rodler *et al.*, 1989). High-molecular-weight methacrylates, urethanes, and some low-viscosity epoxies have been used successfully. The lower the viscosity of the monomers and resins, the finer the cracks that can be filled.

The typical procedure is to clean the surface by air blasting and/or water blasting. Wet surfaces should be permitted to dry several days to obtain the best crack filling. The monomer or resin can be poured onto the surface and spread with brooms, rollers, or squeegees. The material should be worked back and forth over the cracks to obtain maximum filling since the monomer or resin penetrates slowly into the cracks. Excess material should be broomed off the surface to prevent slick, shining areas after curing. If surface friction is important, sand should be broadcast over the surface before the monomer or resin cures.

If the cracks contain significant amounts of silt, moisture, or other contaminants, the sealant cannot fill them. Water blasting followed by a drying time may be effective in cleaning and preparing these cracks. Cores taken at cracks can be used to evaluate the effectiveness of the crack filling. The depth of penetration of the sealant can be measured. Shear (or tension) tests can be performed with the load applied in a direction parallel to the repaired cracks (as long as reinforcing steel is not present in the core in or near the failure area). For some polymers the failure crack will occur outside the repaired crack.

6.2.8 Grouting

Portland cement grouting

Wide cracks, particularly in gravity dams and thick concrete walls, may be repaired by filling with Portland cement grout. This method is effective in stopping water leaks, but it will not structurally bond cracked sections. The procedure consists of cleaning the concrete along the crack; installing built-up seats (grout nipples) at intervals astride the crack to provide a pressure-tight connection with the injection apparatus; sealing the crack between the seats with a cement paint, sealant, or grout; flushing the crack to clean it and test the seal; and then grouting the whole area.

Grout mixtures may contain cement and water or cement plus sand and water, depending on the width of the crack. However, the water–cement ratio should be kept as low as practical to maximize the strength and minimize shrinkage. Water reducers or other admixtures may be used to improve the properties of the grout. For small volumes, a manual injection gun may be used; for larger volumes, a pump should be used. After the crack is filled, the pressure should be maintained for several minutes to ensure good penetration.

Chemical grouting

Chemical grouts consist of solutions of two or more chemicals (such as urethanes, sodium silicates, and acrylomides) that combine to form a gel, a solid precipitate, or a foam, as opposed to cement grouts that consist of suspensions of solid particles in a fluid. Cracks in concrete as narrow as 0.002 in. (0.05 mm) have been filled with chemical grout. The advantages of chemical grouts include applicability in moist environments (excess moisture available), wide limits of control of gel time, and ability to be applied in very fine fractures. Disadvantages are the high degree of skill needed for satisfactory use and lack of strength.

6.2.9 Drypacking

Drypacking is the hand placement of a low water content mortar followed by tamping or ramming of the mortar into place, producing intimate contact between the mortar and the existing concrete (US Bureau of Reclamation, 1975). Because of the low water–cement ratio of the material, there is little shrinkage, and the patch remains tight and can have good quality with respect to durability, strength, and watertightness.

Drypack can be used for filling narrow slots cut for the repair of dormant cracks. The use of drypack is not advisable for filling or repairing active cracks. Before a crack is repaired by drypacking, the portion adjacent to the surface should be widened to a slot about 1 in. (25 mm) wide and 1 in. (25 mm) deep. The slot should be undercut so that the base width is slightly greater than the surface width. After the slot has been thoroughly cleaned and dried, a bond coat, consisting of cement slurry or equal quantities of cement and fine sand mixed with water to a fluid paste consistency, or an appropriate latex bonding compound (ASTM C 1059), should be applied.

Placing of the drypack mortar should begin immediately. The mortar consists of one part cement, one to three parts sand passing a No. 16 (1.18 mm) sieve, and just enough water so that the mortar will stick together when molded into a ball by hand. If the patch must match the color of the surrounding concrete, a blend of grey Portland cement and white Portland cement may be used. Normally, about one-third white cement is adequate, but the precise proportions can be determined only by trial.

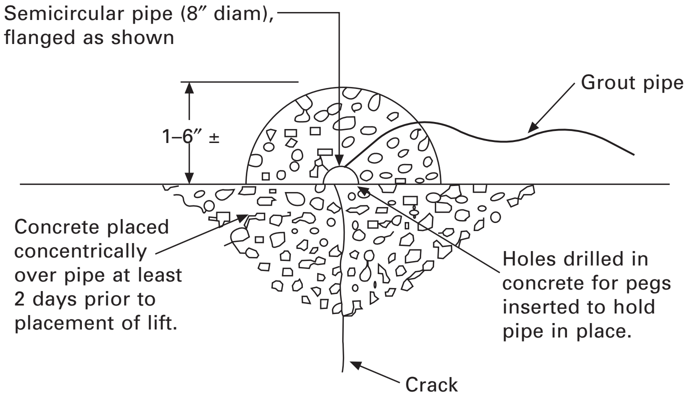
To minimize shrinkage in place, the mortar should stand for 1/2 hour after mixing and then should be remixed prior to use. The mortar should be placed in layers about 3/8 in. (10 mm) thick. Each layer should be thoroughly compacted over the surface using a blunt stick or hammer, and each underlying layer should be scratched to facilitate bonding with the next layer. There need be no time delays between layers. The mortar may be finished by laying the flat side of a hardwood piece against it and striking it several times with a hammer. Surface appearance may be improved by a few light strokes with a rag or sponge float. The repair should be cured by using either water or a curing compound. The simplest method of moist curing is to support a strip of folded wet burlap along the length of the crack.

6.2.10 Crack arrest

During construction of massive concrete structures, cracks due to surface cooling or other causes may develop and propagate into new concrete as construction progresses. Such cracks may be arrested by blocking the crack and spreading the tensile stress over a larger area (US Army Corps of Engineers, 1995). A piece of bond-breaking membrane or a grid of steel mat may be placed over the crack as concreting continues. A semicircular pipe placed over the crack may also be used (Fig. 6.13). A description of installation procedures for semicircular pipes used during the construction of a massive concrete structure follows: (i) the semicircular pipe is made by splitting an 8 in. (200 mm), 16 gauge pipe and bending it to a semicircular section with about a 3 in. (75 mm) flange on each side; (ii) the area in the vicinity of the crack is cleaned; (iii) the pipe is placed in sections so as to remain centered on the crack; (iv) the sections are then welded together; (v) holes are cut in the top of the pipe to receive grout pipes; and (vi) after setting the grout pipes, the installation is covered with concrete placed concentrically over the pipe by hand. The installed grout pipes are used for grouting the crack at a later date, thereby restoring all or a portion of the structural continuity.

6.2.11 Polymer impregnation

Monomer systems can be used for effective repair of some cracks. A monomer system is a liquid consisting of monomers which will polymerize into a



6.13 Crack arrest method (ACI Committee 224.1R).

solid. Suitable monomers have varying degrees of volatility, toxicity, and flammability. They do not mix with water. They are very low in viscosity and will soak into dry concrete, filling the cracks, much as water does. The most common monomer used for this purpose is methyl-methacrylate. Monomer systems used for impregnation contain a catalyst or initiator plus the basic monomer (or combination of monomers). They may also contain a cross-linking agent. When heated, the monomers join together, or polymerize, creating a tough, strong, durable plastic that greatly enhances a number of concrete properties.

If a cracked concrete surface is dried, flooded with the monomer, and polymerized in place, some of the cracks will be filled and structurally repaired. However, if the cracks contain moisture, the monomer will not soak into the concrete at each crack face and, consequently, the repair will be unsatisfactory. If a volatile monomer evaporates before polymerization, it will be ineffective. Polymer impregnation has not been used successfully to repair fine cracks. Polymer impregnation has primarily been used to provide more durable, impermeable surfaces (Hallin, 1978; Webster *et al.*, 1978). Badly fractured beams have been repaired using polymer impregnation. The procedure consists of drying the fracture, temporarily encasing it in a watertight (monomer proof) band of sheet metal, soaking the fractures with monomer, and polymerizing the monomer. Large voids or broken areas in compression zones can be filled with fine and coarse aggregate before being flooded with monomer, providing a polymer concrete repair.

6.2.12 Overlay and surface treatments

Fine surface cracks in structural slabs and pavements may be repaired using either a bonded overlay (see Chapter 8) or surface treatment if there will not be further significant movement across the cracks. Unbonded overlays may be used to cover, but not necessarily repair a slab. Overlays and surface treatments can be appropriate for cracks caused by one-time occurrences and which do not completely penetrate the slab. These techniques are not appropriate for repair of progressive cracking, such as that induced by reactive aggregates, and D-cracking.

Slabs-on-grade in freezing climates should not be repaired by an overlay or surface treatment that is a vapor barrier. An impervious barrier will cause condensation of moisture passing from the sub-grade, leading to critical saturation of the concrete and rapid disintegration during cycles of freezing and thawing.

Surface treatments

Low solids and low-viscosity resin-based systems have been used to seal the concrete surfaces, including treatment of very fine cracks. They are most suited for surfaces not subject to significant wear. Bridge decks and parking structure slabs, as well as other interior slabs, may be coated effectively after cracks are treated by injecting with epoxy or by routing and sealing. Materials such as urethanes, epoxies, polyesters, and acrylics have been applied in a thickness of 0.04–2.0 in. (1–50mm), depending on the material and purpose of the treatment. Skid-resistant aggregates are often mixed into the material or broadcast onto the surface to improve traction.

Overlays

Slabs containing fine dormant cracks can be repaired by applying an overlay, such as polymer, modified Portland cement mortar or concrete, or by silica fume concrete. Slabs with working cracks can be overlaid if joints are placed in the overlay directly over the working cracks. In highway bridge applications, an overlay thickness as low as $1^1/_4$ in. (30mm) has been used successfully (NCHRP, 1970). Suitable polymers include styrene butadiene or acrylic latexes. The resin solids should be at least 15% by weight of the Portland cement, with 20% usually being optimum (Clear and Chollar, 1978). More information on overlays is given in Chapter 8.

6.2.13 Autogenous healing

A natural process of crack repair known as 'autogenous healing' can occur in concrete in the presence of moisture and the absence of tensile stress (Lauer and Slate, 1956). It has practical application for closing dormant cracks in a moist environment, such as may be found in mass concrete structures.

Healing occurs through the continued hydration of cement and the carbonation of calcium hydroxide in the cement paste by carbon dioxide, which is present in the surrounding air and water. Calcium carbonate and calcium hydroxide crystals precipitate, accumulate, and grow within the cracks. The crystals interlace and twine, producing a mechanical bonding effect, which is supplemented by a chemical bonding between adjacent crystals and between the crystals and the surfaces of the paste and the aggregate. As a result, some of the tensile strength of the concrete is restored across the cracked section, and the crack may become sealed.

Healing will not occur if the crack is active and is subjected to movement during the healing period. Healing will also not occur if there is a positive flow of water through the crack, which dissolves and washes away the lime deposits, unless the flow of water is so slow that complete evaporation occurs at the exposed face causing re-deposition of the dissolved salts.

Saturation of the crack and the adjacent concrete with water during the healing process is essential for developing any substantial strength. Submergence of the cracked section is desirable. Alternatively, water may be ponded on the concrete surface so that the crack is saturated. The saturation must be continuous for the entire period of healing. A single cycle of drying and re-immersion will produce a drastic reduction in the amount of healing strength. Healing should be commenced as soon as possible after the crack appears. Delayed healing results in less restoration of strength than does immediate correction.

6.3 Summary

The need to determine the causes of cracking as a necessary prerequisite to repair is emphasized. The selection of successful repair techniques should consider the causes of cracking, whether the cracks are active or dormant, and the need for repairs. Criteria for the selection of crack repair procedures are based on the desired outcome of the repairs. Twelve methods of crack repair are presented, including the techniques, advantages and disadvantages, and areas of application of each.

6.4 Sources of further information and advice

The documents of the various standards-producing organizations were referred to in this chapter and they were sources of major information. The documents are listed below with their serial designation.

American Association of State Highway and Transportation Officials (AASHTO)

- Standard Specification for Highway Bridges

American Concrete Institute (ACI)

- 201.1R Guide for Conducting a Visual Inspection of Concrete in Service
- 201.2R Guide to Durable Concrete
- 207.1R Guide to Mass Concrete
- 207.2R Report on Thermal and Volume Change Effects on Cracking of Mass Concrete
- 207.4R Cooling and Insulating Systems for Mass Concrete
- 224R Control of Cracking in Concrete Structures
- 224.1R Causes, Evaluation and Repair of Cracks in Concrete Structures
- 224.2R Cracking of Concrete Members in Direct Tension
- 224.3R Joints in Concrete Construction
- 302.1R Guide for Concrete Floor and Slab Construction
- 304R Guide for Measuring, Mixing, Transporting and Placing Concrete
- 305R Hot Weather Concreting
- 308 Guide to Curing Concrete
- 309R Guide for Consolidation of Concrete
- 309.2R Identification and Control of Visible Effects as Consolidation on Formed Concrete Surfaces
- 318 Building Code Requirements for Structural Concrete
- 343R Analysis and Design of Reinforced Concrete Bridge Structures
- 345R Guide for Concrete Highway Bridge Deck Construction
- 347 Guide to Formwork for Concrete
- 350.4R Design Considerations for Environmental Engineering for Concrete
- 503R Use of Epoxy Compounds with Concrete
- 517.2R Accelerated Curing of Concrete at Atmospheric Pressure—State of the Art
- 546.1R Guide for Repair of Concrete Bridge Superstructures
- 548R Guide for the Use of Polymers in Concrete

American Society for Testing and Materials (ASTM)

- C 150 Standard Specification for Portland Cement
- C 595 Standard Specification for Blended Hydraulic Cements
- C 876 Standard Test Method for Half Cell Potentials of Reinforcing Steel in Concrete
- C 881 Standard Specifications for Epoxy-Resin-Base Bonding Systems for Concrete
- C 1059 Standard Specification for Latex Agents for Bonding Fresh to Hardened Concrete

Florida Department of Transportation (FDOT)

- Construction Project Administration Manual, 2007

National Cooperative Highway Research Program (NCHRP)

- Synthesis 57, Concrete Bridge Deck Durability, 1970

US Army Corps of Engineers (USACE)

- Maintenance and Repair of Concrete and Concrete Structures, Engineer Manual 1110-2-2002, 1995

US Department of Interior, Bureau of Reclamation

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7
Repair materials for concrete structures

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Abstract: This chapter discusses many types of cementitous and polymeric repair materials and factors that affect material selection. Cementitous materials include those based on Portland cement, modified Portland cements, and other hydraulic cements including magnesium phosphates and high alumina. The two types of polymeric materials are polymer-modified concrete or mortar and polymer concrete. Material selection is based on many factors including construction, load carrying and durability requirements.

Key words: repair materials, cementitious materials, polymer concrete, polymer-modified concrete, material selection.

7.1 Introduction

The selection of repair materials is one of the most important decisions involved in making a successful repair. There are hundreds of repair materials available, and the specifier must determine the required properties of the repair material and match those properties with a suitable material or materials.

Two basic types of repair materials, cementitous and polymeric, and sealants are discussed here. Cementitous materials include those based on Portland cement, rapid-setting Portland cement, modified Portland cements, e.g. silica fume, and other hydraulic cements including magnesium phosphates and high alumina. The two types of polymeric materials are (i) polymer-modified concrete or mortar in which latex, redispersible polymer powder, water-soluble polymer, or liquid polymer are incorporated into a Portland cement concrete and (ii) polymer concrete that incorporates a polymer binder, e.g. epoxy, acrylic, or polyester for the aggregates without the use of Portland cement and water. Cementitous materials are generally used for deep repairs and polymer-based materials are used primarily for shallow repairs or coatings. Sealants include elastomerics such as polyurethanes, polysulfides, and silicones. Polyurethanes are some of the newest repair materials and offer great promise due to their low modulus and excellent bond and durability properties.

Material selection is based on many factors including construction, load-carrying, and durability requirements. Material selection is made even more difficult, since manufacturers often do not readily provide the required properties, and the furnished properties are often based on different tests and different test criteria. It is essential that the cause of the original distress be known in order to make the proper material selection. A brief summary will be presented.

7.2 Cementitous materials

Portland cement concrete repair materials are often the repair material of choice since the properties match the properties of the concrete being repaired.

7.2.1 Conventional Portland cement concrete (PCC)

Most cementitous materials usually include Portland cement, although other cementing materials are available. Portland cement-based materials have the advantages of being widely available, being low in cost compared to many other repair materials, having stiffness and thermal expansion properties which are compatible with the material that is being repaired, and being relatively easy to place, finish, and cure. The limitations of Portland cement are shrinkage that often results in plastic shrinkage or drying shrinkage cracking; relatively high modulus; inability to be placed in thin sections; and the possibility of reactions with certain aggregates or chemicals in the environment. The type of cement can be varied according to requirements. For rapid-setting requirements, high early strength cement is normally used. For applications involving exposure to sulfates, sulfate-resisting cement should be used. For large volume repairs in which the heat of hydration needs to be minimized, low heat of hydration cement can be used. Pozzolanic materials, e.g. fly ash and silica fume, can be used to reduce cost, to reduce heat of hydration, to provide later-age strength development, or to provide increased resistance to alkali–silica reaction.

PCC is used for full-depth and partial-depth repairs with a minimum thickness of about 100mm, although some overlays have been constructed with thicknesses of about 75mm. Conventional concrete is a suitable repair material for marine environments since the high humidity minimizes the likelihood of shrinkage (ACI 546R-04).

7.2.2 Portland cement mortar

Portland cement mortar is similar to PCC except no coarse aggregate is used. Water-reducing admixtures and other modifiers are often used to reduce shrinkage. There are many prepackaged mortar systems available that are particularly appropriate for small repairs. The advantages are similar to those of PCC; in addition, mortars can be placed in thinner sections. The limitations are increased drying shrinkage due to higher water content, higher cement concrete, and higher paste-to-aggregate ratio. Higher air contents may be required to provide adequate resistance to freezing and thawing and salt scaling, and higher air contents reduce strength.

7.2.3 Rapid-setting

Many repairs require rapid-setting concrete. Portland cement, usually Type III (high early strength), can be used with accelerators and high-range water-reducing agents. These materials have been used for many years for full-depth repairs. The advantages are very rapid strength development, e.g. in excess of 7 MPa in three hours or less; relatively low cost compared to proprietary materials; availability; and contractor familiarity. Limitations are similar to those of conventional concrete; in addition, some rapid-setting materials contain high levels of alkalis or aluminate to provide expansion, and it is necessary to limit their exposure to reactive aggregates and sulfates. Moreover, some materials obtain their strength gain and expansion from the formation of ettringite and, under similar conditions, strength regression may occur (ACI 546R-04).

7.2.4 Low slump dense concrete (LSDC)

LSDC has a moderate to high cement factor, water–cement ratio of less than 0.4, and a slump of 50 mm or less. A Portland cement grout is usually scrubbed into the substrate surface just prior to placement. The advantages are its rapid strength gain; ability to be placed in thin sections, e.g. 38 mm; relatively low cost; and ability to be placed with conventional equipment. LSDC is often used for overlays to obtain abrasion resistant and durable concrete surfaces. The limitations are the need for maximum consolidation to achieve optimum density and low permeability; required moist curing of at least seven days due to low water content; and possibility of shrinkage cracks that can permit the intrusion of moisture and chlorides that can result in corrosion of the reinforcing steel (ACI 546R-04).

7.2.5 Drypack

Drypack mortar is a very dry mixture used for making shallow, small repairs especially for vertical or overhead surfaces. One part cement to two or three parts by mass fine sand is used with enough water to produce mortar that can be molded into a ball by slight hand pressure without exuding water or wetting hands. The material is placed by tamping using a block of wood and hammer. Curing is very important due to the low water content. Preshrunk mortar is a low water content mortar that is premixed and allowed to stand for up to 90 minutes, depending on the temperature. These materials are remixed after the idle period. They are suitable when the repairs are too small for the tamping process. The advantage is low shrinkage due to the low water content. Drypack is not suitable for filling areas behind reinforcing bars or for patching holes that extend through the structure. Drypack is useful for repairing form tie hole cavities and any cavity that requires compaction (ACI 546R-04).

7.2.6 Shotcrete

Shotcrete typically consists of one part cement and four parts sand by weight with approximately 7% water by mass of dry ingredients. It can also include coarse aggregate and admixtures. Dry mix shotcrete involves the premixing of dry ingredients with water added at the nozzle. Wet mix shotcrete involves premixing of all ingredients, including water. Advantages include the ability to place concrete on irregular, vertical, and overhead surfaces that are difficult or expensive to form. Materials can be mixed and pumped long distances to the point of application. The limitations are the dependency on the skill and training of the nozzleman to achieve a successful application and the dust and rebound that occur during the application. Shotcrete has been used successfully on many types of structures including dams, bridges, parking structures, buildings, and tunnels (ACI 546R-04).

7.2.7 Fiber-reinforced concrete

Fiber-reinforced concrete is usually Portland cement concrete with either metallic or polymer fibers. The fibers are useful in providing greater resistance to plastic shrinkage cracking and service-related cracking. Fibers are not intended as primary reinforcing. The fibers are added during concrete production. They are useful in shotcrete and in thin overlays that are not sufficiently thick to accommodate reinforcing bars, and they have good resistance to impact, vibration, and blasts. The disadvantages of fiber-reinforced concrete are the reduced workability and the possibility of corrosion stains if the fibers are exposed at the surface (ACI 546R-04).

7.2.8 Silica fume concrete (SFC)

SFC is used where low permeability, high strength, and high durability are required. Typically, 5–10% by mass of the cement of silica fume is used along with a high-range water reducer that is required for workability. The advantages are the low permeability and high strength that result in higher durability. The limitations are reduced workability with increased levels of silica fume, and the potential for plastic shrinkage cracking. The limited amount of bleed water may make a steel trowel finish difficult (ACI 546R-04).

7.2.9 Self-consolidating concrete (SCC)

SCC is a highly flowable, non-segregating concrete that can spread into place, fill the formwork, and encapsulate the reinforcement without any mechanical consolidation. The ability to flow through and around obstructions such as reinforcing steel without consolidation makes it a useful repair material for overhead applications and other hard-to-access locations. The limitations are the necessity of careful mixture proportioning, sensitivity to change in proportions, and need for use of admixtures including high-range water reducers.

7.2.10 Magnesium–phosphate cement (MPC)

MPC is a blend of magnesium oxide (M_gO) and ammonium dihydrogen phosphate ($NH_4H_2PO_4$) that reacts with water to rapidly produce heat and strength. MPC concrete comes prepackaged with cement, aggregates, and other fillers and admixtures. Manufacturers provide a hot weather formulation or retarder, e.g. citric acid, to slow the setting time. The regular formulation will set so rapidly in hot weather that it may be difficult to finish the surface properly for large repairs. The strength and stiffness properties of MPC concrete are similar to those of PCC. The advantages are the very short setting times of 10–20 minutes at room temperature and strengths of 14 MPa within two hours. The material is prepackaged and similar to PCC technology. The limitations are that it should be used with non-calcareous aggregates since reaction of phosphoric acid with the carbonate surfaces produces carbon dioxide and weakens the paste-to-aggregate bond. MPC must be used on well-prepared surfaces that have the carbonation layer removed since the reaction with the carbonated zone or with dust of fracture can cause a reduction of bond strength. MPC is normally used for repairs in which the down time is limited and in cold weather since the exothermic reaction results in rapid setting (ACI 546R-04).

7.2.11 Bonding agents

It is generally recommended that bonding agents be avoided since (i) they are normally not required to achieve adequate bond for Portland cement-based repair materials and (ii) if premature setting occurs, they can become a bond breaker. The three types of bonding agents normally used are cement-based systems, epoxies, and latexes. If bonding agents are used, the manufacturer should be consulted on their proper use (ACI 546R-04).

7.3 Polymeric materials

Polymers are relatively new materials for repair. They are considerably more expensive than conventional repair materials and must be used for applications in which they are cost-effective. They are used in one of two ways: (i) polymer-modified concrete or mortar in which latex, redispersible polymer powder, water-soluble polymer, or liquid polymer are incorporated into a PCC and (ii) polymer concrete that incorporates a polymer binder, e.g. epoxy, acrylic, or polyester, for the aggregates without the use of Portland cement and water (ACI 548.1R-09).

7.3.1 Polymer-modified concrete (PMC)

PMC, sometimes referred to as latex-modified concrete (LMC), is widely used as a repair material, particularly in overlays to resurface or repair concrete substrates. The technology is similar to normal PCC since the polymer modifier is added similar to other admixtures to the mixture which is primarily PCC or mortar. It is less expensive than polymer concrete since less polymer is used, and the polymer is the most expensive component.

The organic polymers are dispersed or redispersed in water. During the hydration of cement, coalescence of the polymer occurs that results in a comatrix of hydrated cement and polymer film throughout the mixture. The film formation reduces the loss of water and makes it available for hydration. This eliminates the need for extended curing beyond 24–48 hours. It is important, however, to apply plastic membrane or other suitable moisture barrier as soon as the surface is finished in order to prevent the film from forming prematurely and causing plastic shrinkage cracks. After 24–48 hours, when the concrete has gained strength, the moisture barrier should be removed so that the polymer film can be formed within the concrete.

The advantages of PMC are improvements in adhesion to concrete, improved resistance to water intrusion, durability, and some strength properties (ACI 548.1R-09). It is used in applications in which protection from steel corrosion, attack by dilute acids and salts, waterproof coatings, leveling of floors, repairs of spalls, bonding of fresh and old concrete, and adhesion of tile are needed (ACI 548.1R-09).

Most PMC utilizes a polymer in latex form. Most latexes are made by emulsion polymerization, where the polymer is formed in water. The most common latexes are:

- **Styrene–butadiene (S–B) copolymers** – S–B has been widely used for nearly 50 years in the USA in PMC for use in overlaying bridge decks and parking structures. The overlays reduce the penetration of de-icing salts that accelerate the corrosion of the reinforcing steel and provide excellent adhesion to the substrate concrete. S–B is generally not used when color fastness is important (ACI 548.1R-09).
- **Acrylic ester homopolymers (PAE) and copolymers with styrene (S–A)** – These latexes have been used for many years as adhesives,

stuccos, exterior insulation finish systems (EIFS), floor overlayments, and concrete repair. Acrylics are often used when color is important.

- **Vinyl acetate copolymers (VAC)** – Typical comonomers are ethylene and the vinyl ester of versatic acid. The reaction products are less water-soluble, and these materials are used as plaster bonding agents and for modification of Portland cement mortars in less severe environments (ACI 548.1R-09).
- **Redispersible powders** – Vinyl acetate copolymers and acrylics have been used in powder form that redisperses when mixed with water. The polymer powder is premixed with cement and aggregates, and water is added at the jobsite. Mixture proportioning is more accurate and convenient compared with two-component latex systems (ACI 548.1R-09).

Major applications are for providing corrosion protection to reinforcing steel that includes more impermeable surfaces to minimize intrusion of salts; providing water-resistant coatings, including architectural finishes; leveling of floors; repairing damaged concrete; and bonding of fresh to hardened concrete and tiles to different surfaces. Parking garage decks and bridge decks have been the recipient of thousands of latex-modified concrete overlays. The minimum overlay thickness is typically 25mm (ACI 548.1R-09).

Mixture proportioning is similar to normal PCC, except that polymer levels of 10–25% by mass of the Portland cement are used; a typical value is 15%. Higher levels usually do not provide improved properties commensurate with the increased cost, since the polymer is the most expensive component of the mix. Antifoaming agents are required to control the air content of PMC and must be added unless the polymer comes with a suitable antifoaming agent. Types I, II, and III Portland, calcium aluminate, and blended cements have been successfully used. Air-entrained cements should not be used. Water–cementitous material (w/cm) ratios of PMC generally range from 0.3–0.4. The low w/cm ratios are possible, since the latex provides lubricating qualities that improve workability. It is important that the water content of the latex be included when calculating the w/cm ratio. When mortars are used, the w/cm ratios are often higher, perhaps as much as 0.6, when very fine aggregates are used, when the surface is highly porous, or when high evaporation rates occur.

ACI 548.1R-09 gives examples of mixture proportions for concrete deck overlays, polymer-modified mortars, and polymer-modified cementitous coatings. For concrete, 415 kg/m^3 Portland cement, fine-to-coarse aggregate ratios of 55:45–65:35, 100/m^3, w/cm of 0.25–0.40 are recommended. For mortar, the following proportions are recommended in parts by mass: cement, 100; fine aggregate, 150–450; polymer powder or latex non-volatile content, 10–20; anti-foam agent, 0.02–0.10; and total water, 25–40. For coatings, the following proportions are recommended in parts by mass: cement, 100;

polymer powder or latex non-volatile content, 10–20; anti-foaming agent, 0.02–0.10; and total water, 25–65.

7.3.2 Polymer concrete (PC)

PC consists of aggregate with a polymer binder; there is no water or hydraulic cement, unless it is used as a non-reactive filler. There are many advantages of PC for repair including rapid curing from –18–40 °C; high tensile, compressive, and flexural strength; very good adhesion to concrete substrates, particularly when dry; good resistance to freezing and thawing; low permeability to water; good chemical resistance; availability of a wide range of elastic moduli, strength, and other properties, and ability to be placed in thin sections. Since polymers have a high range of elastic moduli and elongation, the strength, modulus, and other properties of the PC can vary significantly depending on the polymer, the ratio of polymer to aggregate, and the type and gradation of aggregate. It is very important that the properties of the PC be known before the material is used in repairs (ACI 548.1R-09).

PC has been used for nearly 50 years for repairing concrete, including patching spalls, delaminations and other distressed concrete, and overlays for skid resistance and protection of concrete surfaces. In addition, some precast PC components are used as stay-in-place forms for lining tunnels and other structures.

Polymer concrete formulations

Many PC formulations have been used for repair of concrete. It is necessary to select the right formulation for a specific application. Some of the most widely used monomers for PC repair materials include methyl methacrylate (MMA), unsaturated polyester resins, polyurethane, and vinyl esters.

Epoxy resins with their curing agents have been widely used. The aggregates are generally similar to those used in PCC; they must be sound, clean and dry and well graded to minimize the amount of monomer and resin system to fill the voids. The aggregate-to-resin ratio ranges from 1:1–15:1 by mass, depending on the aggregate gradation and maximum size and the workability required.

Polymer concrete constituents

Initiators for monomers and resins vary depending on the type of resin and monomer. Manufacturers provide the appropriate initiators and, in the case of prepackaged PC, provide the appropriate amount of initiator for a given amount of monomer or resin as a function of ambient temperature.

Promoters for monomers and resins vary according to the type of monomer or resin and are added to permit polymerization, or curing, in the desired time. Epoxy materials are generally formulated in at least two parts. Part A is normally the epoxy resin and Part B is the hardener system. The ratio of resin to hardener is generally 1:1 but varies according to epoxy type and application.

Polymerization, or curing, times can be varied significantly by the amount of initiator and promoter for monomers and resins and the amount of promoter used in epoxies. It is possible to cure in a reasonable time at temperatures well below freezing, e.g. –20 °C.

Aggregates should be clean, dry, sound, and well graded; silica, quartz, granite, good limestone, and other high-quality stone should be used. Rounded shapes are preferable for workability, but crushed, non-rounded aggregates can be successfully used. Moisture on the aggregates generally reduces the bond between the polymer and aggregate, although some polymers are less sensitive to moisture than others (Fowler *et al.*, 1981). Aggregates should be clean and free from asphalt or other contaminants. The maximum size of aggregate is a function of the thickness of the repair and should never be more than one-third the minimum dimension of the repair. The aggregate gradation should provide for a minimum void volume to minimize the amount of monomer or resin required to achieve the least cost and to achieve proper bonding of all aggregate particles.

Safety and handling of chemicals

A complete discussion of safety and handling is beyond the scope of this chapter and other references (e.g. Fowler *et al.*, 1978) and manufacturer's guidelines should be consulted. Generally, monomers and resins used to produce PC are volatile, combustible, and toxic liquids, but practice has shown that stability and safety can be realized by following recommended handling and storage guidelines provided by manufacturers and others.

Most monomers are supplied with inhibitors by the manufacturer prior to shipping to prevent premature polymerization. The inhibitors react with and deactivate the free radicals in growing polymer chains and prevent polymerization by reacting with oxidation products that may be formed in storage containers. The inhibitor may be depleted with time, especially if drums are stored in high temperatures, including exposure to the sun. Storage of monomers and resins should be in a cool, dry location away from sources of ignition. For long-term storage, the level of inhibitor should be regularly monitored; additional inhibitor can be added to maintain the desired level. The monomer manufacturer should be consulted for additional information. Some monomers, e.g. MMA, are classified as flammable, and should be treated accordingly.

The material safety data sheet (MSDS) should be consulted to determine if the monomer or resin is toxic and for appropriate handling and storage instructions. Some people are allergic to some monomers and resins, and handling precautions, including use of respirators, gloves, and other protective clothing, are necessary. Skin that is contacted by these materials should be immediately cleaned by washing. The odor of some monomers and resins may be objectionable, and care must be exercised when using the materials in closed spaces. It may be necessary to use exhaust fans to remove the fumes from the space.

Initiators, generally in liquid form, are the chemicals that start the polymerization process. Most initiators are organic peroxides or azo-compounds including benzoyl peroxide (BPO), 2,2′-azo*bis*(isobutyronitrile) (AIBN), and various ketones such as methyl ethyl ketone peroxide (MEKP). Initiators are used in relatively small quantities compared to the monomer or resin. They should be stored in a dry, cool place, away from heat or sources of sparks or light. (Solid peroxide initiators are very sensitive to fire, shock, friction, or reaction with other chemicals, and should be avoided.) Many initiators have a limited shelf life. Initiators accidentally spilled should be immediately cleaned up and disposed as properly.

Promoters are used to increase rate of polymerization, particularly at ambient temperatures. Promoters are chemicals that greatly increase the decomposition of the initiators, allowing the polymerization to occur in a reasonable time over a wide range of temperatures. Increasing the amount of promoter increases the rate of polymerization. Some of the more common promoters are *N*,*N*-dimethyl-*p*-toluidine, *N*,*N*-dimethyl aniline, cobalt octate, and cobalt naphthanate. Like initiators, they are used in relatively small quantities, often less than the initiators. It is extremely important to avoid mixing promoters directly with peroxide initiators since the mixture can react explosively. A good practice is to add the promoter to half the monomer and to add the initiator to the other half of the monomer and then mix the two containers of monomer, either by in-line mixing or manual mixing. Handling and storage procedures of promoters should be similar to those used for monomers.

Epoxies

There are literally hundreds of epoxy resins and, when combined with a large number of curing agents, flexibilizers, fillers, and other chemicals, a very large number of end products can be produced. Most epoxy formulations are skin sensitizers, and care must be exercised in their handling and use. Workers should wear protective gloves, clothing, glasses, and shoes. Soiled garments should be immediately removed and replaced with clean clothing. Skin contacted with epoxy should be immediately cleaned with soap and water. It is very important that the manufacturer's instructions be carefully followed. Generally, storage should follow the same guidelines as for monomers and resins. It should be noted that epoxies tend to become more viscous at low temperatures, and mixing is more difficult.

Other concerns

A high level of safety should be observed when working with chemicals used in PC. Normally, sources of ignition and flame should be carefully monitored and eliminated if possible. Smoking in the work area should be strictly forbidden. Workers should be thoroughly educated on the proper uses and handling of the materials, clean-up procedures, and measures to be taken in case of worker contact with the chemicals. Local medical clinics and hospitals should be notified at the beginning of the job as to the chemicals being used so that they will be equipped to handle medical emergencies. It should be noted, however, that there have been many large projects using PC without any significant problems.

7.4 Joint sealants

Joint sealants are flexible polymers that provide waterproofing in openings between rigid concrete elements. They must be able to accommodate joint movement and prevent the intrusion of water and solid particulates such as sand and dust. There are many types of materials used for field-molded sealants, but the most common are the thermosetting, chemically cured, of which the most widely used are polysulfides, polyurethanes, and silicones. These materials are one or two component systems that cure by chemical reaction from liquid form to a solid state. They have very good resistance to weathering and ozone, flexibility and resilience over a wide range of temperatures, and inertness to many chemicals. Urethane sealants have good resistance to indentation and abrasion. The expansion–compression range for silicones is +100/–50%; polysulfides, 25%; and polyurethanes, 25%. Silicones remain more flexible over a wider temperature range than other field-molded sealants. For clean and sound substrates, the thermosetting, chemically cured sealants will generally have a much greater service life than other sealants (ACI 504R-04).

7.5 Sealers

7.5.1 Penetrating sealers

Penetrating sealers are intended to penetrate the pores of the substrate concrete. Depth of penetration is often quite variable and depends on the properties of both the sealer and the concrete to which it is applied. The size of the sealer molecule and size of the concrete pore structure play a large role in determining the depth of penetration. The amount of moisture in the pores can also influence the penetration depth. Silanes and siloxanes are two of the most commonly used sealers. They can be applied with rollers or squeegees or can be sprayed on the surface. The surface should be clean, since penetrating sealers are sensitive to contaminants and previously applied sealers. Ultraviolet and abrasion resistance are usually good, but penetrating sealers will not bridge new or existing cracks (ACI 546R-04).

7.5.2 Surface sealers

Surface sealers are materials that lie on the surface and have a thickness of 0.25mm or less. The most common surface sealers are polyurethanes, acrylics, various epoxies, moisture-cured urethanes, and some latex-based and oil-based paints. These materials will not bridge moving cracks, but may close small, non-moving cracks. Some of these materials are affected by ultraviolet light and will wear; however, epoxies and acrylics perform well when subjected to wear. Surface preparation is important to assure good adhesion to the substrate (ACI 546R-04).

7.6 New materials

There are thousands of repair materials, and new materials are introduced on a regular basis. Polymeric materials are the newest, and new materials for repairing spalls, sealing cracks, and sealing surfaces are regularly introduced. The low-modulus repair materials for spalls are some of the most promising.

7.7 Material selection

One of the most important and most challenging decisions in repair is the selection of the appropriate material. The International Concrete Repair Institute (ICRI) Guideline No. 03733 *Guide for Selecting and Specifying Materials for Repair of Concrete Surfaces* provides a rational and complete methodology for the selection of materials and should be consulted. A brief summary of some of the criteria for various applications will be presented.

7.7.1 Structural properties

For structural repairs, the essential requirements are:

- good tensile bond to substrate;

- modulus of elasticity similar to substrate;
- very low compressive creep to ensure that the repair carries its portion of the load;
- very low drying shrinkage to ensure that the repair carries its portion of the load.

7.7.2 Environmental exposure properties

Ambient temperature changes require a coefficient of thermal expansion similar to that of substrate, unless a low-modulus material is used; low modulus can offset a high thermal coefficient to prevent cracking when thermal contraction occurs or spalling when thermal expansion occurs. Generally, materials that exhibit high exotherm during cure should be avoided to prevent spalling or cracking; low-modulus materials can accommodate some of these effects. For materials subject to moisture and freezing and thawing, the materials must have low permeability and low drying shrinkage.

Latex-modified concrete or mortar is often used when the repair is to be exposed to chlorides and/or moisture and the thickness is greater than 25 mm. For thinner repairs, a polymer mortar is the preferred choice.

7.7.3 Effects of thickness

For full-depth repairs, Portland cement materials are usually the material of choice because of cost. For moderately thick repairs, e.g. 25–75 mm, LMC is a potential material, particularly if durability is of concern. For thin repairs, 25 mm or less, polymer mortars, e.g. epoxy or MMA, generally works well.

7.7.4 Importance of coefficient of thermal expansion/contraction and modulus of elasticity

For non-structural repairs such as pavement spalls, low-modulus polymer materials have proven to be very durable. The thermal coefficient is usually much higher than for cement materials, but it is offset by the much lower modulus. In addition, low-modulus materials have a much higher tensile elongation that reduces the likelihood of cracking.

7.8 Sources of further Information and advice

- ACI 237R-07 Self-Consolidating Concrete
- ACI 546R-96 (Reapproved 2001) Concrete Repair Guide
- ACI 503R-93 (Reapproved 2008) Use of Epoxy Compounds with Concrete

- ACI 503R.5R-92 (Reapproved 2003) Guide for the Selection of Polymer Adhesives with Concrete
- ACI 503.4-92 (Reapproved 1997) Standard Specification for Repairing Concrete with Epoxy
- ACI 504R-04 (Reapproved 1997) Guide to Sealing Joints in Concrete
- ACI 506.2-95 Specification for Shotcrete
- ACI 506.4-94 (Reapproved 2004) Guide for the Evaluation of Concrete
- ACI 506R-05 Guide to Shotcrete
- ACI 548.1R-09 Guide for the Use of Polymers in Concrete
- ACI 548.3R-03: Polymer-Modified Concrete
- ACI 548.4-93: Standard Specification for Latex-Modified Concrete (LMC) Overlays (Reapproved 1998)
- ACI 548.5R-94: Guide for Polymer Concrete Overlays (Reapproved 1998)
- ACI 548.8-07: Specification for Type EM (Epoxy Multi-Layer) Polymer Overlay for Bridge and Parking Garage Decks
- ACI 548.9-08: Specification for Type ES (Epoxy Slurry) Polymer Overlay for Bridge and Parking Garage Decks
- ICRI Guideline No. 03733 Guide for Selecting and Specifying Materials for the Repair of Concrete
- www.fixconcrete.org

7.9 References

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8 Bonded concrete overlays for repairing concrete structures

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Abstract: The bonded concrete overlay is an important and frequently used repair method. The aim is either to replace deteriorated concrete or to increase the cross-section and, hence, the load-carrying capacity. Since the bonded overlay and the substrate structure act monolithically, the overlay contributes substantially to both load-carrying capacity and structural stiffness. A prerequisite for obtaining monolithic action is to provide good bond between substrate and overlay. The measures start with the selection of removal method. The chapter discusses more than 20 factors influencing bond and identifies the most important ones. Based on these factors and some case studies presented, the chapter is finalized by providing recommendations for obtaining a good result.

Key words: bonded concrete overlays, removal methods, factors influencing bond, case studies, recommendations.

8.1 Introduction

The bonded concrete overlay is an important and frequently used repair method which has been used successfully for more than half a century. Its applications cover various types of pavements, industrial floors, and bridge decks. Its popularity is due to its simplicity. Since it is placed on top of an existing concrete structure, the formwork can be limited to low vertical forms at the overlay's edges. Usually, the overlay consists of ordinary Portland cement concrete, but there are also other alternatives, e.g. polymer-modified concrete, steel fibre-reinforced concrete (SFRC), and preplaced-aggregate concrete (ACI Committee 546, 2006). The overlay may either be reinforced or plain. Plain concrete overlays are usually thin. However, reinforcement bars, welded-wire fabric (WWF), or steel fibres may be used in thick overlays primarily for crack control.

The aim of the bonded concrete overlay is either to replace deteriorated concrete or to increase the cross-section and, hence, the load-carrying capacity. The bonded overlay has several advantages compared with an unbonded overlay. Since the bonded overlay and the substrate structure act monolithically, the overlay contributes substantially to both load-carrying capacity and structural stiffness. The bonded overlay also prevents water and moisture from finding a path for transport along the interface. Finally, good bond provides crack distribution potential to the overlay if it cannot withstand cracking due to prevented shrinkage, prevented thermal movements, or mechanical loading.

8.2 Preparation of existing concrete for repair

A chain is never stronger than its weakest link. The bonded concrete overlay is a part of a composite concrete structure also composed of the remaining part of the old structure – the substrate or the base – and the interface between substrate and overlay. If the aim of the overlay is to restore or enhance the load-carrying capacity, all three links in the composite structure chain have to be sound and well-conditioned. Consequently, the existing concrete has to be assessed and possibly repaired prior to the overlay placement. The aim of the assessment is to determine the cause, the degree, the severity, and the extent of the deterioration, if any. Before the damage cause is determined, any repair action is without a sound basis, since the action might not solve the problem and, in the worst cases, even worsen it. For example, some dense surface protection systems may lead to moisture trapped inside the concrete, causing frost damage or reinforcement corrosion. The degree and severity of damages influence the decision whether deteriorated concrete has to be removed or not. The extent – concerning both area and depth – determines the selection of a suitable removal method. More information on assessment is given in Chapter 4.

There are at least four good reasons for removing poor concrete:

- Deteriorated concrete has low strength, leading to reduced load-carrying capacity.
- Deteriorated concrete might contain detrimental substances, e.g. chlorides, which may damage reinforcement both in the existing concrete and in the new concrete overlay.
- Deteriorated concrete is often porous and water-permeable.
- The reinforcement bars or WWF in poor concrete are often corroded and need to be uncovered, cleaned from corrosion products (rust), and partly replaced by new bars or other units.

In many cases, the result of the concrete removal is the creation of a sound and suitable substrate constituting the horizontal formwork and promoting the placement of a good bonded concrete overlay. In other cases, the deterioration has gone so far that the concrete removal has to be stopped before all poor concrete has been removed in order to maintain minimum load-carrying capacity and avoid installation of horizontal formwork. The degree and extent of deterioration may also vary considerably along the concrete bridge deck or other concrete structure to be repaired. The probability of a successful concrete overlay diminishes if its thickness varies dramatically. To secure a more even overlay thickness, the most extensive damages may be repaired prior to the overlay placement. Here, patch repair might be the best solution. Patch repairs are characterized by limited extent, which enables the use of specific repair mixes that cannot be used in the entire overlay due to economical or technical reasons. In a limited patch repair, the technical advantages of zero or very low shrinkage outweigh the disadvantage of deviating mechanical properties, e.g. strength, stiffness, creep, and coefficient of thermal expansion. For use in the entire overlay, they would, however, be detrimental. More information on patch repair is given in Chapter 11.

As mentioned above, it may not always be possible to remove all deteriorated concrete. Cracks are frequent in deteriorated concrete and, in order to improve the structural integrity and prevent liquid (water with and without de-icing agents) and gas (CO_2) transport, detrimental cracks ought to be repaired prior to the overlay placement. Before the crack repair, the cause, the condition, and the width of the cracks have to be determined. When did the crack arise? How wide is it? Is it still growing in length or width? Does the width vary with temperature differences between day and night? Does it go through the entire structure? Is it dry, moist, water-filled, or subjected to unilateral water pressure? Does it transfer load? The answers to these questions is important since they determine the selection of appropriate crack repair materials and methods. More information on crack repair is given in Chapter 6.

Finally, deterioration processes such as carbonization-initiated or chloride ingress-initiated reinforcement corrosion often lead to area losses of the reinforcement bars. If this loss is more than negligible, additional reinforcement has to be installed and anchored in the substrate concrete prior to overlay placement.

8.3 Methods of removing damaged concrete

The method of concrete removal has a major influence on both the surface of the remaining concrete and the properties of the uppermost layer of the remaining concrete. Some removal methods leave a rough and sound surface that promotes a good bond, while others may even introduce microcracks to a certain depth in the remaining concrete. Some removal methods can only remove a thin layer of concrete, while others have the ability to remove concrete to a significant depth. Some of the most frequently used methods for removing concrete are summarized in Table 8.1. Additional methods, e.g. other hand-held tools, spring-action hammers, concrete crushers, drills and saws, non-explosive demolition agents, and jet-flame cutting, are described by ACI Committee 555 (2002). According to Silfwerbrand (1990), only water-jetting (or hydrodemolition) has the ability to remove concrete selectively,

Table 8.1 Methods of concrete removal

Removal method	Principle	Depth action [mm]	Important advantages	Important disadvantages
Sandblasting	Blasting with sands	No	No microcracking	Not selective, leaves considerable sand
Scrabbling	Pneumatically driven bits impact the surface	No (6)	No microcracking, no dust	Not selective
Shotblasting	Blasting with steel balls	No (12)	No microcracking, no dust	Not selective
Grinding (planning)	Grinding with rotating lamella	No (12)	Removes uneven parts	Dust development, not selective
Flame-cleaning	Thermal lance	No	Effective against pollutions and painting, useful in industrial and nuclear facilities	The reinforcement may be damaged, smoke and gas development, safety considera-tions limit use, not selective
Milling (scarifying)	Longitudinal tracks are introduced by rotating metal lamellas	Yes (75)	Suitable for large-volume work, bond if followed by water flushing	Microcracking is likely, reinforcement may be damaged, dust development, noisy, not selective
Pneumatic (jack) hammers (chipping), hand-held or boom-mounted	Compressed-air-operated chipping	Yes	Simple and flexible use, large ones are effective	Microcracking, damages reinforcement, poor working environment, slow production rate, not selective
Explosive blasting	Controlled blasting using small, densely spaced blasting charges	Yes	Effective for large removal volumes .	Difficult to limit to solely damaged concrete, safety and environmental regulations limit use, not selective
Water-jetting (hydro-demolition)	High-pressure water jet from a unit with a movable nozzle	Yes	Effective (especially on horizontal surfaces), selective, does not damage reinforcement or concrete, improved working environment	Water handling, removal in frost degrees, costs for establishment

i.e. remove poor, damaged, and cracked concrete while leaving strong, sound, and undamaged concrete.

Since water-jetting holds a unique position, it is worth its own paragraph. For concrete removal, it was originally developed in Italy in the early 1980s, but further developed in Sweden (Silfwerbrand, 2000). For horizontal surfaces, e.g. bridge decks and pavements, the water-jetting is run by a robot slowly progressing along a bridge lane, while an arm provided with rotating or oscillating nozzle(s) is continuously and repeatedly moved from left to right. The water-jet leaves the nozzle at the speed of sound and with a pressure in the vicinity of 100 MPa or even higher. A water pressure is developed inside the concrete, and when the pressure exceeds the tensile strength of concrete the concrete spalls. Since poor concrete has lower strength than sound concrete and more high-pressure water contributes simultaneously to the spalling in porous and cracked concrete, the process is more effective in poor concrete. This is the explanation of its selectivity. However, water-jetting is also used for removal to specified depth. By changing pressure, flow, exposure time, nozzle type, and nozzle motion pattern, the effect of the water-jet can be controlled. In Sweden, the selectivity of all newly developed robots to be used on Swedish Road Administration bridges has to be checked by laboratory tests on concrete slabs cast with two significantly different concrete qualities. The test goal is to remove all low-quality concrete, while leaving all high-quality concrete intact.

8.4 Factors influencing bond

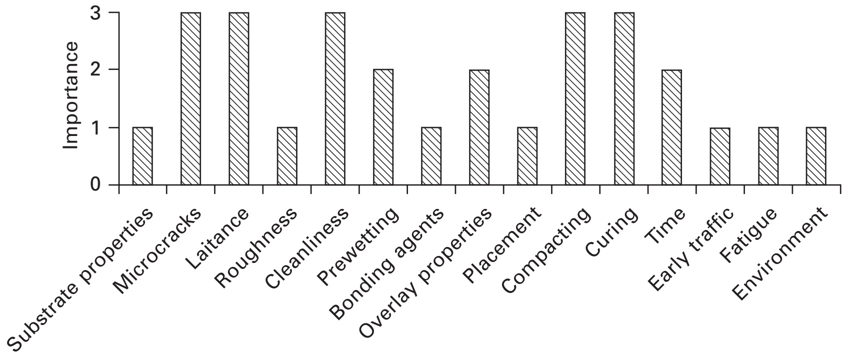
8.4.1 General

The bond between substrate and overlay is dependent on a large number of factors (Silfwerbrand, 1990). Substrate, surface treatment, overlay properties, concrete production, curing, time, and exposure all have influence on the bond and could be subdivided into further factors. Figure 8.1 shows a trial to estimate the importance of the factors listed in chronological order.

The importance estimation is based on test data and case studies. The following five factors are considered to be the most important ones:

- absence of microcracks;
- absence of laitance;
- cleanliness;
- compaction of the overlay;
- curing.

The factors are listed in chronological order. The moisture condition of the substrate (e.g. prewetting), overlay properties, and time after overlay placement belong to the next category of factors. All other factors have less



8.1 Factors affecting bond listed in chronological order and evaluated according to their individual importance (Silfwerbrand and Beushausen, 2005).

importance. In the following subsections, the 15 most important factors will be discussed.

Prior to thoroughly dealing with the factors affecting bond, it is appropriate to discuss the need for a certain bond strength for our goal, monolithic action. Theoretically, the stress at the interface between the overlay and the substrate must not exceed the bond strength. Since the state and magnitude of stress are dependent on the loading, the load level, the kind of structural element, the geometry including boundary conditions, and the stiffness ratio between overlay and base, it is easy to understand that it is almost impossible to determine either type of stress (tension, shear, combined shear and tension) or quantitative stress levels for general cases. Certainly, interfacial shear stresses are usually of uppermost interest in composite beams and slabs. However, usually the bond strength is determined through pull-off tests, i.e. tests providing the tensile strength and especially tensile bond strength in cases where the failure is an interface failure. (For other failures, the maximum stress obtained only gives lower bound of the bond strength.) Comparative tests show that the shear bond strength is approximately twice as high as the tensile bond strength (Silfwerbrand, 2003). Consequently, when conformity is checked by pull-off tests whereas design is based on allowed shear strength, there is an inherent conservatism.

A great deal of research has been devoted to bond strength, whereas very few researchers have investigated the magnitude of bond strength sufficient for monolithic action between the substrate structure and the overlay. Efforts with both composite beams and composite slabs have, however, been conducted in, for example, Sweden (Silfwerbrand, 1984, 1987). Three different substrate surface treatments were investigated in the beam tests: (i) hand-held pneumatic hammer, (ii) hand-held pneumatic hammer and bonding agent (epoxy), and (iii) steel grinding. Composite beams with the first two treatments failed in shear at roughly equal loads as a homogeneous reference beam, whereas an interface failure occurred in the smooth, steel-ground substrate surface at less than 1/3 of the general ultimate load. Subsequent pull-off tests averaged at 1.5 MPa for treatments Nos 1 and 2 and varied between 0 (six tests) and 3.4 MPa (one single test) for treatment No. 3.

Five composite concrete slabs and two homogeneous concrete slabs of the same dimensions, equally reinforced, were simply supported on the four corners and loaded to failure with a central point load. All seven slabs performed equally well, showing similar load-displacement curves and load-carrying capacity (Fig. 8.2). After these tests, the bond strength was determined by pull-off tests. Here, the average bond strength varied depending on the surface preparation method. The lowest value (1.5 MPa) was obtained for the slabs containing substrate surfaces treated by pneumatic hammers.

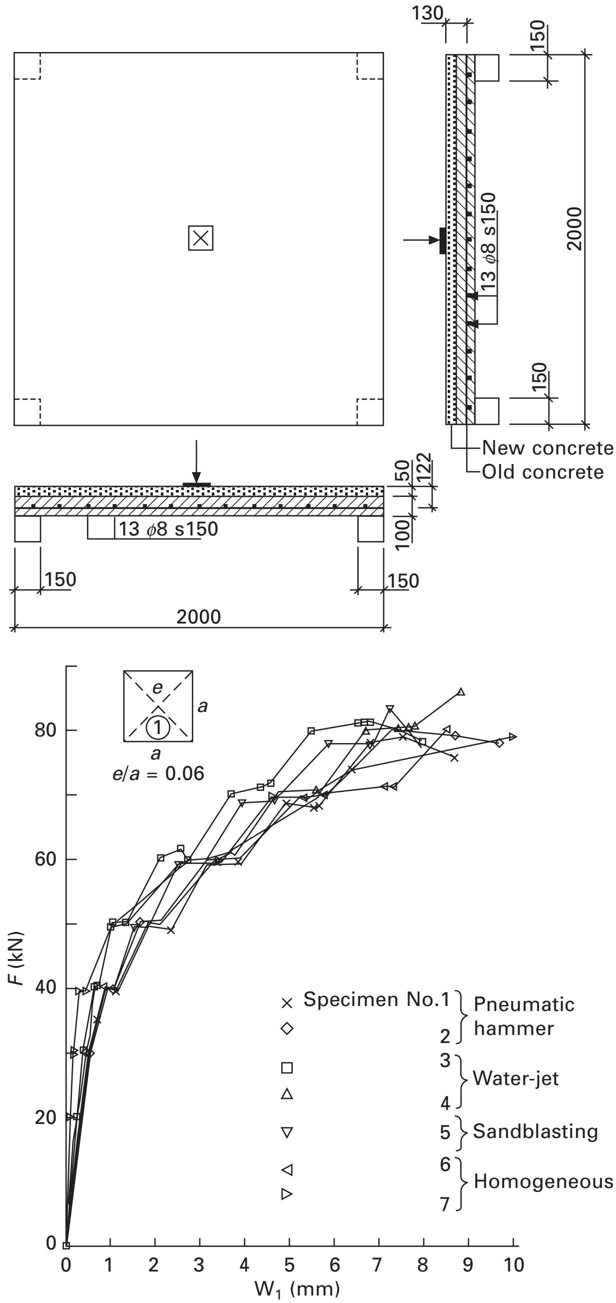
Obviously, a strength value equal to 1.5 MPa was sufficient to provide monolithic action in both cases. This value might be considered as a default value of necessary bond strength, despite the fact that differently loaded composite structures of other shapes may lead to both higher and lower stresses in the interface.

8.4.2 Substrate properties

In this subsection, substrate properties are defined as mechanical properties of the substrate, whereas other substrate characteristics, e.g. presence of microcracks, roughness, and moisture condition, are discussed in other subsections. Mechanical properties are primarily compressive strength, tensile strength, modulus of elasticity, and coefficient of thermal expansion. Of those, tensile strength is the most important.

In sound concrete, the difference in tensile strength between two concrete qualities is much less than the difference in compressive strength. If the compressive strength increases by 50%, the tensile strength only increases by 30%. Concrete bridge decks and concrete pavements are usually cast with relatively high-quality concrete, implying that the tensile strength variation is usually fairly small. Since concrete is subjected to stress development, the tensile strength of the substrate is usually higher, or at least of the same magnitude as, the overlay concrete strength. Consequently, the tensile strength of the substrate has only a minor influence on the bond strength as long as the substrate concrete is sound. This changes if the substrate concrete is deteriorated or poor. In Section 8.2, the importance of removing deteriorated concrete prior to overlay placement was highlighted. To relate bond strength to the tensile strength of concrete containing microcracks, macrocracks, or other damage is, however, hardly meaningful.

Since compression does not constitute a decisive state of stress in the interface, the compressive strength *per se* does not have any influence on the



8.2 Load displacement curves for five composite 2 × 2 × 0.15m RC slabs with varying treatment of the substrate and two homogeneous RC slabs (Silfwerbrand, 1987). The load is denoted *F* and the mid-span displacement is denoted w_1.

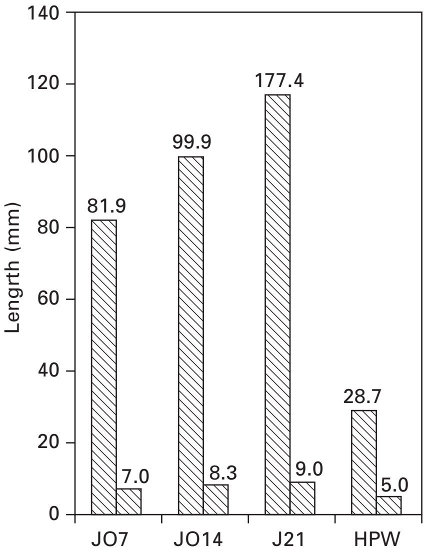
bond strength. Traditionally, several concrete properties have been related to the compressive strength, and some of those have influence on the bond strength. We have already discussed the tensile strength. The modulus of elasticity E_c is another property. It increases slowly when the compressive strength increases. A 50% compressive strength increase leads to a 15% increase in modulus of elasticity. Several researchers have stressed the importance of obtaining equal E_c values for both substrate concrete and overlay concrete. Their hypothesis is that deformations due to, for example, shrinkage and creep, otherwise will lead to detrimental stress concentrations. Both theoretical and empirical evidences of this problem are, however, missing.

In most concrete design textbooks and codes, there is only one single value given for the coefficient of thermal expansion; $\alpha = 1 \cdot 10^{-5}$ (°C)$^{-1}$. This is, however, not completely true. In reality, α varies with the α value of the predominant aggregate and values in the interval $0.5 \cdot 10^{-5} < \alpha < 1.5 \cdot 10^{-5}$ are possible. Limestone aggregates give low values, whereas quartzite gives high values. If the coefficient of thermal expansion is different between the substrate and the overlay, every thermal change will lead to internal stresses in both layers. These stresses will be superposed to the stresses due to differential shrinkage (see Section 8.4.16) and may either increase or decrease the crack risk in the overlay. If, for example, $\alpha_{\text{overlay}} > \alpha_{\text{substrate}}$ and the concrete composite structure is subjected to a thermal decrease, the effects of differential shrinkage will be strengthened.

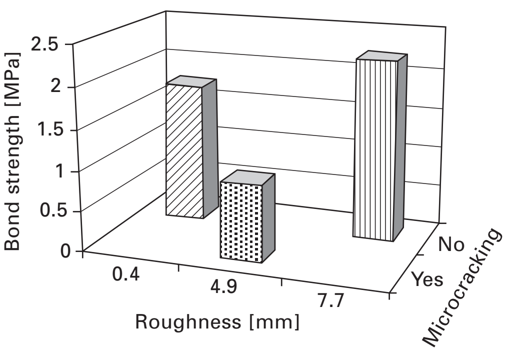
Naturally, the substrate properties cannot be changed. Consequently, the detrimental property differences between substrate and overlay, if any, have to be considered when selecting the concrete mix for the overlay. By coring and laboratory tests, the strength, modulus of elasticity, and the coefficient of thermal expansion can be determined. Subsequently, the overlay concrete mix can be designed in order to match the properties of the substrate, especially the coefficient of thermal expansion but, if desired, also the modulus of elasticity. A fundamental role of the thumb is to repair 'like with like', i.e. one material with a similar one.

8.4.3 Microcracks

The surface of the remaining concrete needs to be free from microcracking. Otherwise, the top layer of the remaining concrete will constitute a zone of weakness. The amount of microcracking is governed by the selected method of concrete removal. In principle, mechanical methods (jackhammers) are likely to introduce microcracking, whereas water-jetting has been shown to be a less damaging – but efficient – method (Silfwerbrand, 1990). The difference is significant (Fig. 8.3). Laboratory tests show that the bond strength was around 2 MPa in the case of no microcracking (water-jetting).



8.3 Measured microcrack length (left column) and number of cracks (right column) of substrate surfaces treated by jack hammers weighing 7, 14, and 21 kg, respectively, and high-pressure water jet according to Courard *et al.* (2005).



8.4 Micro-cracking has a major importance on bond strength. Sandblasting (left) and water-jetting (right) do not cause micro-cracking whereas pneumatic jackhammers (middle) does. Simultaneously, the limited influence of roughness (here measured as the double amplitude of a saw-tooth curve) is shown. (Laboratory test results from Silfwerbrand, 1987)

In the case where the surface contained microcracking, i.e. the surface treated with pneumatic jackhammers, bond strength values obtained were only half as high (Fig. 8.4). Field tests, however, have shown that the bond strength can reach satisfactory values, if mechanical removal is followed by high-pressure water cleaning. The field tests included the removal of a rutted top part of a concrete pavement by scarifying and placement of a new concrete overlay. Average bond strength of 2.3 MPa was obtained (Silfwerbrand and Petersson, 1993). Talbot *et al*. (1994) and Carter *et al*. (2002) found that sandblasting subsequent to the use of heavy mechanical methods could remove the damaged concrete and provide a sound interface. Wells *et al*. (1999) and Warner *et al*. (1998) achieved good bond strength on surfaces that were sandblasted without prior roughening.

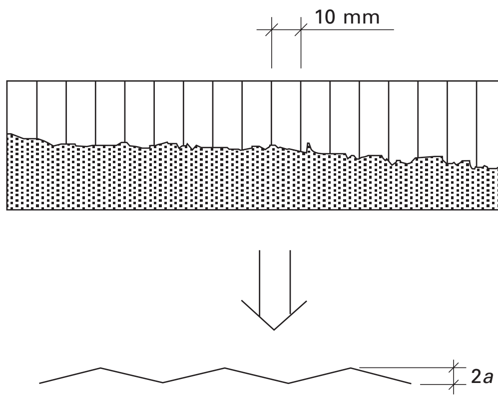
8.4.4 Laitance

Laitance is a layer of weak and non-durable material containing cement and fines from the aggregates, brought through bleed water to the top of the concrete. If the laitance is not removed prior to overlay placement, it will lower the bond markedly. If the new overlay replaces old concrete, the laitance will be removed together with the deteriorated concrete. However, if the overlay is placed on unaltered parts, the removal of the laitance must not be forgotten. Usually, sandblasting will be sufficient to remove the laitance.

8.4.5 Surface roughness

The roughness of the substrate surface depends to a large extent on the method of substrate surface preparation. Mechanical methods of concrete removal normally leave the substrate surface much rougher than blast methods. The magnitude of surface roughness for concrete repairs is commonly measured in mm. There are several methods of measuring the surface roughness. The most widespread test method is probably the sand area method (Kaufmann, 1971), in which sand of known volume is spread over the concrete surface to form a circle until all sand has settled in the surface cavities. The roughness is defined as the average thickness of the 'cylinder'. A simple surface roughness method using a saw-tooth curve (Fig. 8.5) was introduced by Silfwerbrand (1987).

The surface roughness used to be considered to have a major influence on the bond between old and new concrete. Bond tests have, however, shown that surface roughness only has a minor influence on the bond (Fig. 8.4). In a test series, bond to rough, water-jetted surfaces was compared with bond to smooth, sandblasted surfaces (Silfwerbrand, 1990). The average bond strength was approximately equal, but interface failures were more frequent on the sandblasted surface. The conclusion is that there might be a threshold value. If the surface roughness is higher than the threshold value, further improvement of the roughness does not seem to enhance bond strength. According to these tests, this threshold value ought to be close to the surface



8.5 The surface profile is transferred to a saw-tooth curve with double amplitude 2*a*. The distance between consecutive measuring points is 10 mm.

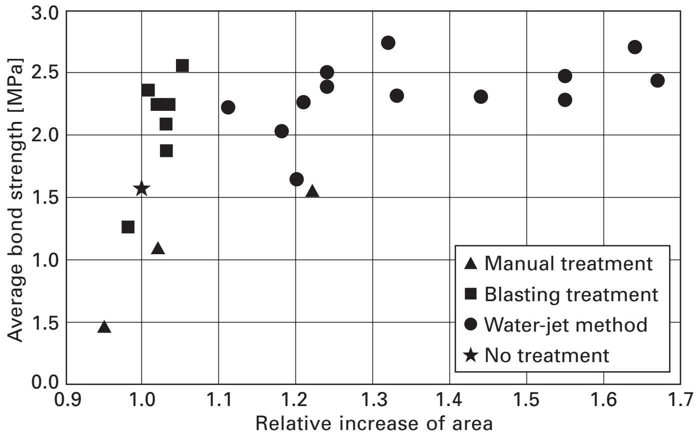
roughness of typical sandblasted surfaces. The above cited beam tests (Section 8.4.1, Silfwerbrand, 1984) indicate that the threshold value corresponds to a surface roughness that at least exceeds that of steel grinding.

The existence of a threshold value is evidenced by tests by Takuwa *et al.* (2000). They studied the influence of surface preparation of the substrate concrete and defined 'increase of area' as a surface roughness parameter. They compared manual treatment, shot blasting treatment, and water-jetting with no treatment (Fig. 8.6).

Mainz and Zilch (1998) achieved high bond strengths on water-jetted surfaces with a roughness of >1 mm. Similarly, Tschegg *et al.* (2000) compared roughness of 1.75 mm to 0.65 mm on water-jetted surfaces and found better bond characteristics for the rougher interface.

8.4.6 Cleanliness

The single most important factor influencing bond is the surface cleanliness. A surface which is contaminated at the time of overlay placement will produce poor bond characteristics. In the first Swedish bridges repaired with water-jetting and bonded overlays in 1984 and 1985, coring showed poor bond at several locations. In most cases, insufficient cleaning was the reason for this. Loose particles were found in the interface between old and new concrete (Silfwerbrand, 1990). In order to achieve cleanliness, the surface needs to be cleaned twice, particularly if it is water-jetted. The first cleaning has to be done shortly after water-jetting to prevent loose concrete particles, such as exposed unhydrated cement surfaces, from bonding to the (moist) surface.

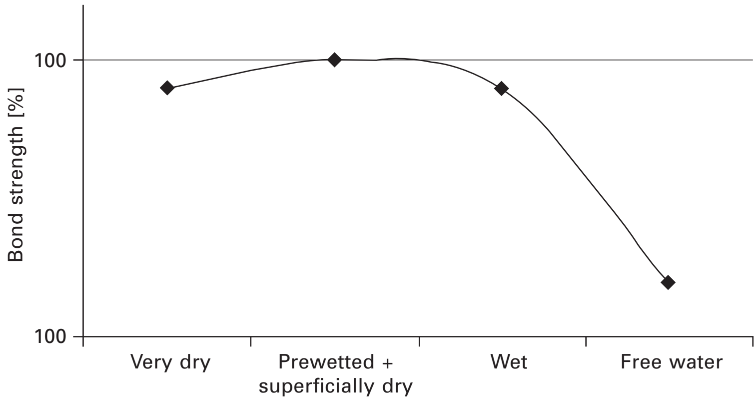


8.6 The relationship between the increase of surface roughness – described as increase of substrate area in percent – and bond strength (Takuwa *et al.*, 2000). One line in two perpendicular directions of the treated surface was measured every 0.1 mm using a laser device. The area increase was determined by transferring the obtained zigzag lines. Note that an area increase also was measured for the non-treated surface (indicated with a star).

The second cleaning has to be carried out shortly before overlay placement to make sure that the surface is free from sand, oil, dust, or other particles from the environment or the construction works. The best methods are hosing down with high-pressure water and the use of vacuum cleaners.

8.4.7 Prewetting

The influence of surface moisture on the bond between old and new concrete has been investigated in many studies. Since the effect of surface moisture seems to be fairly small, it is difficult to distinguish it from other factors that are seldom completely constant in practical tests. Theoretically, the influence of surface moisture on bond strength is assumed to have some form of optimum at a certain level and decrease when the moisture increases to free water (Fig. 8.7). A too dry surface may absorb water from the fresh concrete overlay giving risk of a heterogeneous, porous zone close to the interface. If the surface is too wet, an overlay zone with a high water–cement ratio will develop close to the interface, which will lead to a local reduction in overlay strength. Free water at the surface may destroy the bond completely. Zhu (1991, 1992) has found experimental signs of optimal moisture, but the moisture influence on the bond was so small that it was difficult to distinguish between moisture influence and scatter of test results. Li *et al.* (1999) measured the bond strength of repaired specimens after freeze–thaw cycles



8.7 Essential relationship between moisture condition of the substrate at time of overlay placement and bond strength.

and found that different repair materials correspond to different optimum interface moisture conditions at the time of overlay placement.

According to the Swedish Road Administration (2006), the substrate surface ought to be moistened but superficially dry at the moment of overlay concrete placement. This condition may in temperate climates be obtained by maintaining the substrate surface wet during 48 hours and letting it dry during the last 12 hours (the last night) prior to the concrete placement.

8.4.8 Bonding agents

Bonding agents, e.g. Portland cement grout, latex-modified Portland cement grout, and epoxy resins, are sometimes used to improve bond (ACI Committee 546, 1980). However, bonding agents cannot compensate for bad substrate surface preparation and may act as a bond breaker when used inappropriately (Pigeon and Saucier, 1992; Schrader *et al.*, 1992). The chapter author's opinion is that bonding agents should normally be avoided. The use of bonding agents leads to two interfaces and thus to the creation of two possible planes of weakness instead of one. Besides, a grout often has a high water–cement (W/C) ratio leading to a low strength and the risk of a cohesive failure within the bonding agent itself. On the other hand, grouts may have an ability to assimilate loose particles on an insufficiently cleaned surface. This assimilation may increase the bond strength for this specific case. Bonding agents may improve bond strength for certain materials, especially stiff repair mortars that cannot properly fill open pores and cavities. Of course, if any kind of commercial repair system is used, the user has to follow the manufacturer's instructions, i.e. if a bonding agent or a primer is included in the system, it has to be used. Otherwise, the achievement of the putatively good result cannot be anticipated.

8.4.9 Overlay properties

The overlay properties are predominantly dependent on the type and quality of the overlay. There are several alternatives:

- Portland cement mortar and concrete;
- silica fume mortar and concrete;
- polymer-modified mortar and concrete;
- polymer concrete;
- steel fibre-reinforced concrete;
- slurry-infiltrated fibre-reinforced concrete (SIFCON);
- preplaced-aggregate concrete.

Of course, the selection of the overlay material influences the material properties of the overlay and, hence, also the bond. However, this chapter is focused on bond, whereas Chapter 7 is devoted to repair materials, including those for bonded overlays. Consequently, this section is limited to a general discussion on the influence of fresh and hardened material properties on the bond.

Fresh material properties

In order to fill open cavities and voids on the substrate concrete surface, the workability and the compaction (see Section 8.4.11) of the overlay have great importance. Concrete of conventional fluidity and workability is generally sufficient in this respect. Small repair patches are commonly carried out with premixed, relatively stiff mortars, which are applied with a trowel. Self-compacting (self-consolidating) materials (with high workability) are expected to fill the depressions of the substrate surface, open cavities, and voids to a high percentage and, hence, improve the potential of obtaining good bond strength.

Strength

When evaluating bond strength, the substrate characteristics, the interface characteristics, and the overlay characteristics all contribute. Of course, the properties of the hardened concrete overlay also influence strength. Of those, the tensile strength has the most significant influence. It also affects crack development and, therefore, the formation of boundary conditions that may support the initiation of debonding. Delatte *et al.* (2000b) found that an increase in early age concrete strength increased both tensile and shear bond strength significantly.

Pigeon and Saucier (1992) state that the interface between old and new concrete is very similar to bond between aggregates and cement paste.

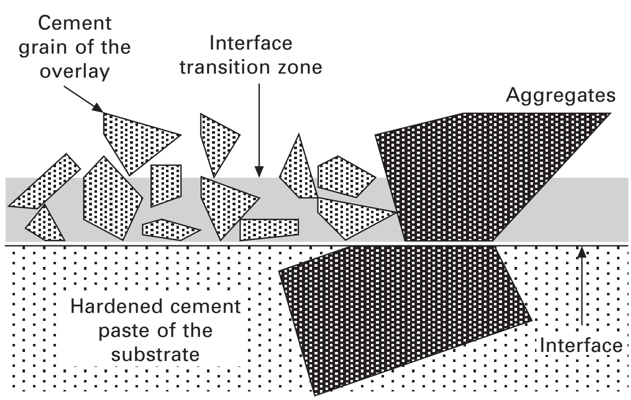
According to their research, a wall effect exists between overlay and substrate, resulting in a transition zone that creates a layer of weakness (Fig. 8.8).

Van Mier (1977) has summarized existing knowledge on interfaces between aggregates and cement matrix. The bond mechanisms between aggregate and cement paste depend largely on the porosity of the aggregate. Generally, a thin layer of CH forms at the physical boundary between aggregate and cement matrix, followed by a relatively open layer containing oriented CH crystals, ettringite, and CSH. This so-called contact or transition layer has a high porosity. Van Mier explains this high porosity with absorption of mixing water at the surface of aggregate particles, which increases the effective w/c ratio. According to his research, fracture surfaces generally exist not directly at the physical boundary between aggregate and matrix but rather slightly remote from the interface in the porous transition zone.

Beushausen (2005) found that 'interface' failure in shear bond tests commonly occurs inside the overlay very close to the interface, which was observed both on short-term specimens and on specimens tested after more than two years. This supports the theory that an interfacial transition zone exists, as illustrated in Fig. 8.8, representing a zone of weakness. On sound and well-prepared substrates, overlay strength is therefore one of the decisive factors for bond strength and bond durability.

Other overlay properties

Overlay permeability may influence bond durability, for example, very impermeable overlays result in stresses at the interface when moisture from the substrate cannot migrate through the overlay (Schrader *et al.*, 1992). The addition of polymers to cementitious repair mortars was found to result in better bond characteristics on specimens subjected to extensive temperature



8.8 Transition zone between substrate and overlay, according to Pigeon and Saucier (1992).

cycles (Atzeni *et al*., 1993). Chen *et al*. (1995) measured a significant increase in shear bond strength with the addition of short carbon fibres to repair mortar. They attributed the effect to the decrease in drying shrinkage and the resulting decrease in interface stress. Granju (1996) states that fibres enhance bond durability through the control of crack development.

Geometrical properties of the overlay

Repair patch dimensions, such as area and thickness, can affect bond durability due to the influence on stresses resulting from differential movement between the substrate and the overlay. In general, large repairs tend to crack more easily than smaller areas. According to Banthia and Bindiganavile (2001), thin repairs are more likely to debond than thicker repairs. In contrast, Laurence *et al*. (2000) have concluded from their measurements that bond strength was not influenced by the repair thickness. Equal bond strength does not necessarily mean that the likelihood of a bond failure is equal. The failure likelihood is dependent on both bond strength and bond stress, and the thickness is likely to influence the bond stress, e.g. by thickness-dependent shrinkage stresses.

8.4.10 Concrete placement

The concrete placement of the overlay does not differ and does not have to differ from concrete placement of other horizontal structures. The concrete placement should be carried out with the same care as for other concrete structures. Handling techniques that lead to a poor result, e.g. too high rate of placing and too high dumping heights, should be avoided in order to prevent segregation that, in the case of the bonded concrete overlay, may lead to a significantly reduced bond strength.

8.4.11 Concrete compaction

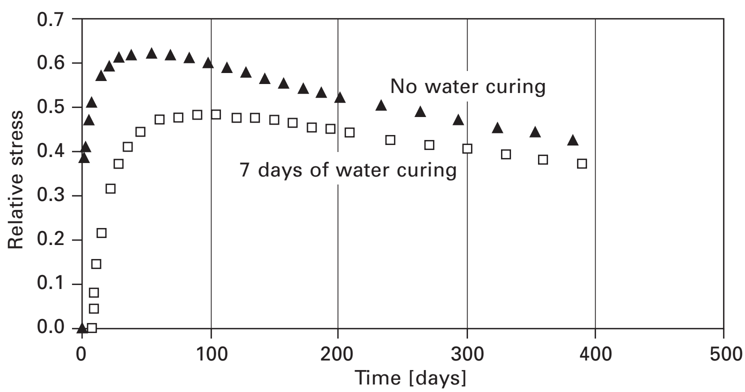
Compaction is important to obtain a dense and homogeneous overlay as well as a good and uniform bond. Compaction is especially important in overlays on rough surfaces to prevent the development of air pockets in the valleys of the surface texture. Air pockets were found in some cores taken from Swedish repairs in the early 1980s (Silfwerbrand, 1990). The Swedish Road Administration (2006) recommends the use of vibration pokers and vibration platforms.

Self-compacting concrete (SCC)

During the 1990s, self-compacting concrete (or self-consolidating concrete, which it is called in North America) was introduced on the international market. It does not need any conventional compaction and improves both productivity and the working environment. SCC has been considered to have superior bond properties compared with normally vibrated concrete. The hypothesis is supported by its filling ability and early tests on the bond between SCC and reinforcement bars. A literature review (Silfwerbrand, 2007) showed, however, that the superiority of SCC to normal concrete is not guaranteed either for concrete-to-steel or concrete-to-concrete bond. The literature review also shows that there is a lack of rigorous laboratory and field studies evaluating concrete-to-concrete bond and comparing SCC with normal concrete. Since numerous factors influence the bond strength, it is very difficult to distinguish the effect of the compaction method (i.e. SCC or vibration) from all the others. The understanding of the mechanisms of bond ought to be improved, for example, by microstructural analyses. Finally, it may be stated that the results found in the literature, although somewhat disappointing, show pull-off strength values of similar magnitude as obtained for successful repairs.

8.4.12 Curing

Early-age shrinkage may result in overlay cracking. Cracks may initiate debonding due to the formation of boundary conditions (free edges). Curing is therefore one of the most important factors in reducing early-age overlay and interface stresses. It prevents moisture loss and thereby reduces early-age shrinkage, leading to higher tensile strength at the onset of shrinkage (Fig. 8.9). Simultaneously, other advantages are gained: reduced risk of plastic cracking, higher strength, improved durability, and better wear resistance.

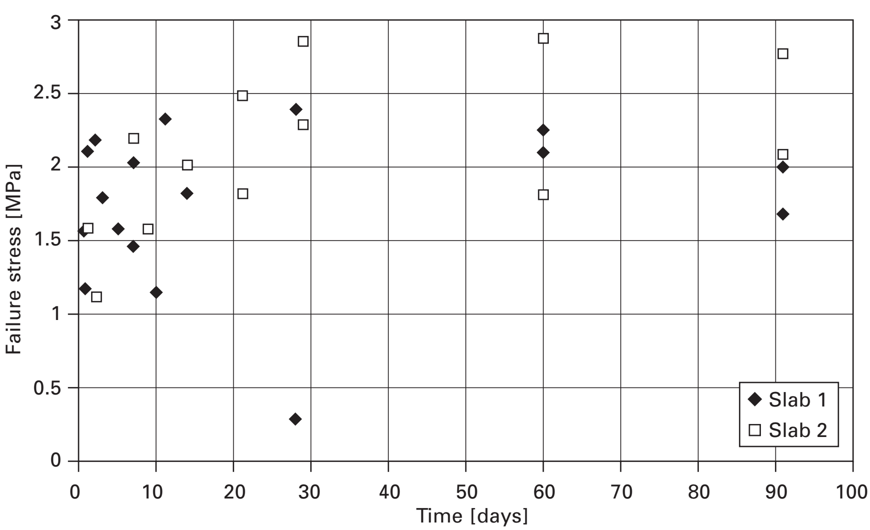


8.9 Relationship between relative stress (equals ratio between maximum tensile stress and available tensile strength) and time after casting. (Computational result according to Silfwerbrand, 1987)

Paulsson and Silfwerbrand (1998) recommend a minimum of five days water curing on bridge deck overlays. Exposure to direct sunlight was found to have a detrimental effect on shear bond strength even under proper curing conditions, which included wet burlap and plastic (Delatte *et al*., 2000b). Li *et al*. (1999) tested different repair materials and found that, for some materials, wet curing and, for others, dry curing results in better bond strength. Schrader (1992), in contrast, states that curing mainly affects the surface of a concrete repair, but has little influence on the material or bond properties at a depth of more than 25 mm.

8.4.13 Time

The development of early-age bond strength is important for the structure's ability to withstand interface stresses induced by differential movement between substrate and overlay. For pavement and bridge deck overlays, high early bond strength is usually required due to traffic and live loads. According to Delatte *et al*. (2000a, b), bond strength develops rapidly after placement, similar to concrete compressive strength development. In their studies they suggest a concrete maturity approach, which characterizes bond strength development in relation to the concrete's rate of hydration rather than its age. Laboratory tests carried out by Silfwerbrand (1992) have shown that the bond strength development is rapid, more rapid than both compressive and splitting tensile strength (Fig. 8.10). At an age of 24 hours, the obtained

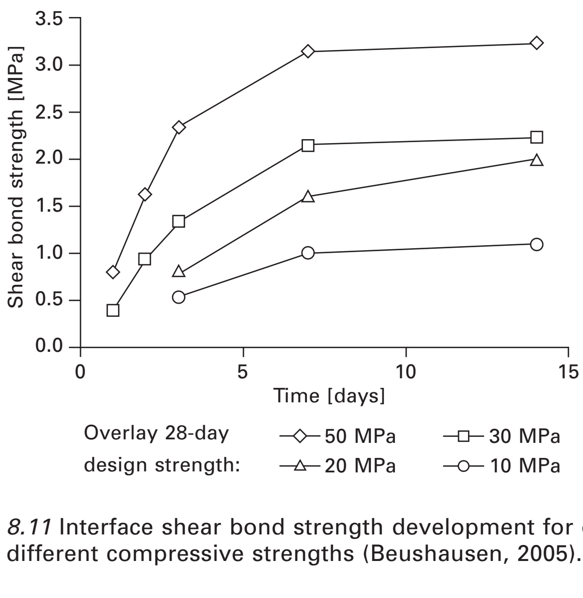


8.10 Tensile bond strength development according to Swedish test results (Silfwerbrand, 1992).

bond strength exceeded 1 MPa. The bond strength development was found to be faster than the development of compressive strength of the concrete overlay. The tests were carried out indoors. At the time of testing, the old concrete had a compressive cube strength of 54 MPa. The overlay had a 28-day compressive cube strength of 63 MPa. The surface of the old concrete was water-jetted prior to overlay placement.

Beushausen (2005) measured the development of bond strength with time, using a modified version of the direct shear test proposed by FIP (1978) (compare Section 8.4.5). Test specimen size was 150 × 150 × 50mm with both substrate and overlay having a depth of 75mm. The interface was sandblasted with an average roughness of 0.7mm. All specimens were fully water cured at 23 °C. Different overlay strengths were used (28-day design strengths 10, 20, 30, and 50 MPa), the substrate compressive strength being 48 MPa. Shear bond strength development was closely related to the development of compressive overlay strength, with a relatively constant ratio between the two of approximately 0.10–0.12 over the whole test period for all specimens. Shear bond strength development is shown in Fig. 8.11. The test results represent the mean value of four to six specimens after exclusion of outliers.

For fully bonded overlays it appears that the rate of bond strength development conservatively can be related to that of overlay compressive strength. This facilitates the design of interface shear strength for structural overlays.



8.11 Interface shear bond strength development for overlays of different compressive strengths (Beushausen, 2005).

8.4.14 Early traffic

Normally, traffic-induced vibrations do not harm the development of bond between old and fresh concrete. Independent researchers have even found that continuous and limited vibrations may increase both overlay and bond strength; see, e.g., Silfwerbrand (1992). However, heavy vibrations starting a few hours after overlay placement should be avoided. Manning (1981) stated that the best way of preventing heavy vibrations is to maintain a smooth riding surface and a smooth transition at the expansion joints of the bridge. Based on Silfwerbrand's literature survey and tests (1992) and investigations by Ansell (2003), Ansell and Silfwerbrand (2003) have proposed the vibration limitations shown in Table 8.2.

8.4.15 Fatigue

Usually, concrete structures exposed to fatigue are designed by replacing the design strength values with reduced ones. The reduction is usually considered proportional to the logarithm of the number of load repetitions. The relative reduction expressed, e.g. as a percentage of the 28-day strength, does not seem to differ among different strength properties. Tepfers (1979) and Tepfers and Kutti (1979) conducted fatigue tests on cubes both in compression and splitting tension and found similar behaviour. His results are also valid for flexure and flexural concrete strength (see, e.g. Söderqvist, 2006).

It is likely too that the bond strength would show fatigue behaviour similar to that as compressive, tensile, and fatigue strength, but the question is if the bond in a real composite structure is subjected to high-cycle fatigue? Bond strength is usually tested in pure tension by pull-off tests. It is very difficult to find a loading case that causes repeated tension in the interface between overlay and substrate. The shear stress might vary when an overlaid concrete bridge deck or concrete pavement is trafficked, but usually the magnitude of

Table 8.2 Recommended maximum allowed peak particle vibration velocities (Ansell and Silfwerbrand, 2003)

	Concrete age	Max. allowed PPV
Fresh concrete	0–3 hours	100 mm/s[a]
Young concrete	3–12 hours	35 mm/s[b]
Early-age concrete	12–24 hours	50 mm/s
Almost hardened	1–2 days	100 mm/s
concrete	2–7 days	175 mm/s
Hardened concrete	7–10 days	225 mm/s
	> 10 days	300 mm/s

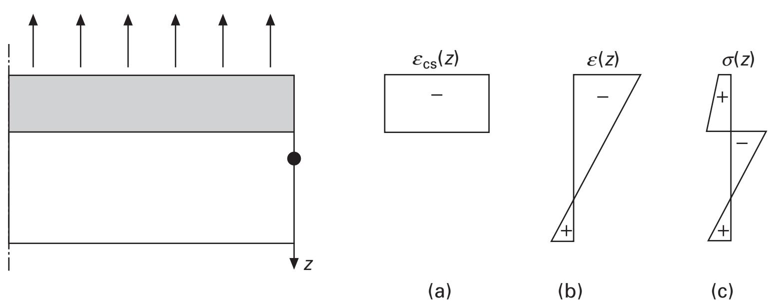
[a] Revibration may increase concrete strength.
[b] The velocity of heavy traffic must be limited.

the shear variation is very small. In order to obtain failure within a limited testing time, fatigue tests are normally conducted at much higher stress levels. However, even at high stress level, the bond has not been shown to constitute a plane of weakness providing that the static bond strength is good (Silfwerbrand, 1984) and that the interfacial plane does not coincide with the reinforcement plane (Silfwerbrand, 1989). Finally, it may be added that pull-off tests conducted on overlaid concrete bridge decks subjected to ten years of intensive truck traffic have not revealed any bond strength reduction (see Section 8.5).

8.4.16 Differential shrinkage

Differential shrinkage, i.e. the shrinkage difference between overlay and substrate (Fig. 8.12), has been investigated at least since the 1950s (e.g. Evans and Parker, 1955). It leads primarily to tensile stresses in the overlay and compressive stresses in the base or substrate. However, if the thickness of the overlay constitutes a substantial part of the thickness of the composite structure, both parts may be subjected to both tension and compression at different height levels.

Various models to compute stresses due to differential shrinkage exist, but there is still no consensus. The simple models are based on linear elasticity and Bernoulli's principle of plane section remaining plane (Evans and Parker, 1955; Birkeland, 1960; Silfwerbrand, 1986). By using these models, only normal stresses that are constant along the composite beam can be computed. The shear at the interface will be zero along the beam but reach infinite values at the edges. This evident shortcoming has been eliminated in more sophisticated theories (e.g. Jonasson, 1977; FIP, 1978; Silfwerbrand, 1986). By using these theories, finite shear stresses may be computed. They still have their maximum at the edges, but diminish successively to zero along a distance that approximately equals three times the overlay thickness.



8.12 Sketch showing the principle of shrinkage: free shrinkage (a), strain (b) and stress (c) diagrams in an overlaid concrete structure subjected to differential shrinkage.

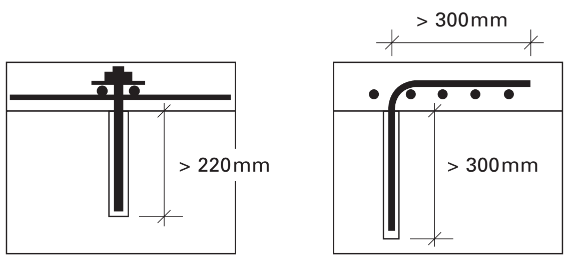
Indirect evidence of the shear stress concentrations at edges is given by field observations of debonding starting at edges of, or vertical cracks through, the overlay. However, despite both laboratory and field measurements through pull-off tests at various distances from the overlay edge, it has not been possible to verify any bond strength reduction due to differential shrinkage shear stresses. The distance had no significant importance on the bond strength. A possible hypothesis is that the coring that is necessary for the pull-off test releases the shear stresses when the drill passes the interface. Creep will also lead to vanishing stresses due to differential shrinkage. In conclusion, there is no evidence that differential shrinkage has any decisive influence on the bond strength in cases where debonding does not occur.

8.4.17 Reinforcement

In cases of poor or uncertain bond or in areas with specific demands (e.g. high shear or tensile stresses, serious consequences in the case of failure), it might be necessary to strengthen the shear and tension capacity by installing some kind of reinforcement crossing the interface. It is mandatory that the reinforcement be sufficiently anchored in both the remaining concrete and in the overlay. Two Swedish solutions are given in Fig. 8.13. It is important to note that the reinforcement units do not work until the bond has broken. The reason is that the reinforcement unit has to be strained before it carries more than a negligible part of the load. This is similar to ordinary reinforcement that mainly carries load after concrete cracking.

8.4.18 Other factors

The most important factors have been listed in Fig. 8.1 and discussed in Sections 8.4.2–8.4.16. Some additional factors are discussed here. However,



(a) (b)

8.13 Example of dowel bars crossing the interface. (a) nut, plate, and stud M20 (diameter = 20mm) in injected hole with diameter Φ = 30mm; (b) bent reinforcement bar with diameter Φ = 16mm; in injected hole with diameter Φ = 25mm.

despite what single researchers may have reported, it is the chapter author's opinion that these factors have only a negligible impact on the bond between substrate and overlay.

Carbonation of the substrate

Through, for example, water-jetting, the old weathered and carbonated concrete surface can be removed, exposing a fresh and uncontaminated surface. However, long periods of time between substrate removal and overlay placement may result in new carbonation of the exposed surface, especially in dry environments. Test results by Gulyas *et al*. (1995) show that carbonation may decrease the bond strength significantly. Block and Porth (1989), in contrast, found that substrate carbonation does not affect pull-off bond strength.

Substrate temperature

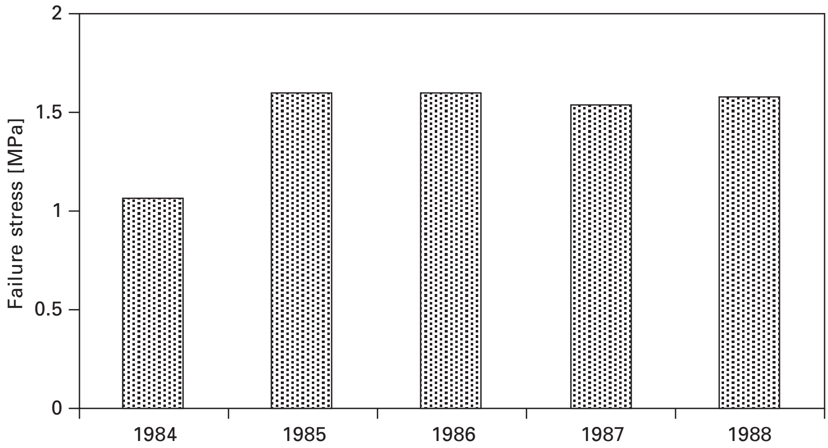
The substrate temperature at the time of overlay placing was found to have a significant effect on shear bond strength development. Cold substrate (4°C) results in lower initial bond strength but higher long-term bond strength, compared with substrates of higher temperature (21 or 38 °C) (Delatte *et al*., 2000a).

State of stress

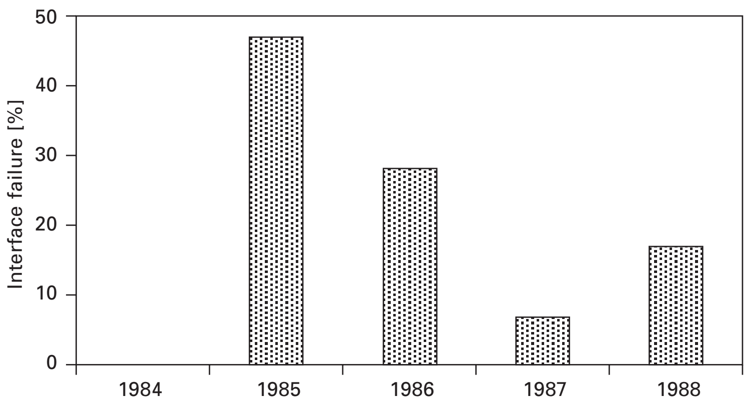
Within the bridge, the bond may theoretically be different in zones with positive (sagging) moment and negative (hogging) moment, respectively. In zones with positive moment, the overlay is compressed. In zones with negative moment, the overlay is subjected to tensile stresses that might increase the risk of vertical cracking and maybe also the risk of cracking along the interface plane. However, there is no evidence of such a relation. Silfwerbrand and Paulsson (1998) tested the bond strength in both zones on seven concrete bridge decks but did not find any difference.

8.5 Case studies

In the mid-1980s, more than 250 Swedish concrete bridge decks were repaired by means of water-jet removal and a new-cast, bonded concrete overlay. Shortly after repair, the Department of Structural Engineering, Royal Institute of Technology (KTH), tested the bond on more than 20 bridges (Silfwerbrand, 1987, 1990). The average failure stress was above 1.5 MPa (Fig. 8.14). The bond strength might have been even higher, since only a minor fraction of the tested cores failed at the interface (Fig. 8.15). If the failure



8.14 Average failure stresses of 200 pull-off tests from 30 Swedish bridges between 1984 and 1988 (Silfwerbrand, 1990).



8.15 Percentage interface failures of 200 pull-off tests from 30 Swedish bridges between 1984 and 1988 (Silfwerbrand, 1987).

occurs elsewhere, the failure stress only gives a lower bound of the bond strength. The failure modes were analysed and a failure within the remaining old concrete bridge deck was found to be the dominant one. In many cases, the damage was so severe that it was not possible to remove all degraded concrete without perforating the concrete bridge deck. Consequently, a layer of relatively low-quality concrete had been saved to provide formwork for the new concrete overlay. The predominant causes of the occurring interface failures were found to be poor cleaning and insufficient overlay compaction. Since the importance of careful cleaning and overlay compaction has been recognized by the contractors working with bridge repairs, the average failure stress has increased and the interface failure percentage has decreased (Figs 8.14 and 8.15).

Differential movement between substrate and overlay, such as differential shrinkage and thermal expansion and contraction, is commonly considered to be the main factor affecting long-term bond properties. It ought to be emphasized that high initial bond strength does not necessarily lead to bond durability. Proper substrate preparation, material selection, and curing procedures should, however, normally result in good long-term bond properties. Repaired beams and columns with well-bonded overlays were shown to have structural capacities similar to monolithic members (Silfwerbrand, 1984; Souza and Silva Appleton, 2003). Carter *et al.* (2002) state that well-designed bridge deck overlays can be expected to provide more than 30 years of service life if they are placed and cured correctly. Talbot *et al.* (1994) investigated the influence of different interface textures and concluded that smooth surfaces as well as sandblasted surfaces experienced a significant loss of bond strength with time. However, surfaces which were roughened mechanically and subsequently sandblasted had good bond durability. Debonding on vertical or overhead concrete repair patches usually leads to spalling and hence to failure of the repair. Delaminated bridge deck overlays, in contrast, may be rebonded using epoxy injection (Smithson and Whiting, 1992).

Between 1995 and 1999, a major research project was carried out at the Royal Institute of Technology (KTH) in Stockholm, Sweden, to study durable repairs (Paulsson, 1998, 1999; Paulsson and Silfwerbrand, 1998; Silfwerbrand and Paulsson, 1997; Paulsson-Tralla, 1999). Seven bridges repaired in the mid-1980s were selected. Tests on concrete strength, bond strength, and freeze–thaw resistance were carried out as well as crack surveys and measurements on chloride penetration.

In the previous section, the most important factors influencing bond have been listed. They are connected to measures taken prior to, during, and shortly after overlay placement. The long-term performance of the bond may, however, also be influenced by traffic loads and environmental factors. The following factors were considered to be most important: (i) the number of heavy vehicles, (ii) the stiffness of the bridge, in turn dependent on the structural system, (iii) the use of de-icing salts, and (iv) the number of freeze–thaw cycles. The seven bridges chosen had different traffic volumes, different structural systems, locations in different climate zones, and different needs for de-icing salts (Table 8.3). Consequently, all factors were covered in the investigation, even though not all could be studied separately.

Bond strength was measured on cores drilled from the concrete bridge decks. The cores had a nominal diameter of 100mm. At the time of repair, the pull-off tests were carried out *in situ*. In 1995, the pull-off tests were carried out in the laboratory some time after drilling. Between drilling and testing, the cores were stored in sealed plastic bags. Before testing, the cores

Table 8.3 Bridges investigated (Silfwerbrand and Paulsson, 1998)

Bridge	Structural system	Climate zone	Year of construction	Year of bridge deck repair	Average daily traffic
Bjurholm	Steel girders	5	1966	1985	1440
Mälsund	Concrete arch	2	1924	1986	210
Skellefteå	Steel girders	4	1960	1987	25 000
Södertälje	Flat slab	2	1966	1989	~10 000
Umeå	Steel girders	4	1949	1987	23 000
Vrena	Steel girders	2	1937	1987	170
Överboda	Concrete arch	2	1946	1986	7600

Note: Climate zone 2 means an annual frost amount of 300–600 day × degree of frost, zone 4: 900–1200, and zone 5: 1200–1500. 300 day × degree of frost = 30 days of –10 °C or 60 days of –5 °C.

Table 8.4 Field tests on bond at time of repair and ten years afterwards (Silfwerbrand and Paulsson, 1998)

Bridge	Number of cores tested		Failure stress [MPa]		Failure stress increase [%]
	at time of repair	in 1995	at time of repair	in 1995	
Bjurholm	9	8	1.96	1.99	2
Mälsund	6	8	1.71	2.17	27
Skellefteå	3	9	1.43	1.82	27
Södertälje	?	7	> 1.5	2.05	~ 35
Umeå	4	9	1.56	1.61	3
Vrena	8	9	1.49	1.56	5
Överboda	12	9	2.09	2.18	3

were sawn perpendicularly to the cylinder axis and steel plates were glued to the end surfaces. The cores were then placed in a testing machine with a loading rate of 500 N/s (0.064 MPa/s). Failure stresses and failure modes were registered and it was found that most failures occurred in the old concrete. All tests are described in Paulsson (1997). The results are summarized in Table 8.4.

On all bridge decks investigated, the failure stress had increased slightly during the time after repair (Table 8.4). That means that neither traffic loading nor de-icing salts and frost–thaw cycles had deteriorated the bond between old and new concrete. Obviously, long time tensile stresses in the overlay do not impair the bond. The overall conclusion is that a good and durable bond can be obtained provided all operations with concrete removal, surface cleaning and preparation, overlay placement and compaction, and curing are carried out correctly and carefully.

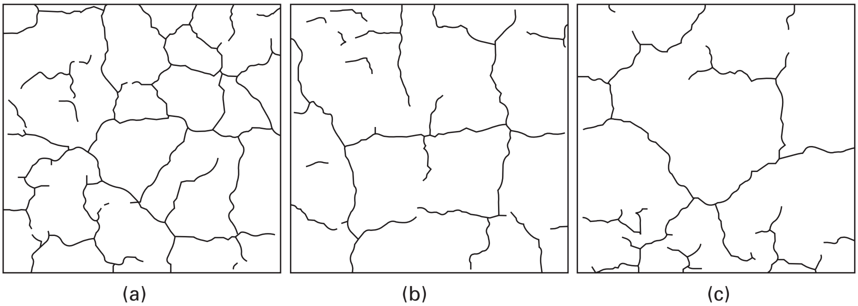
8.6 Recommendations

8.6.1 Design

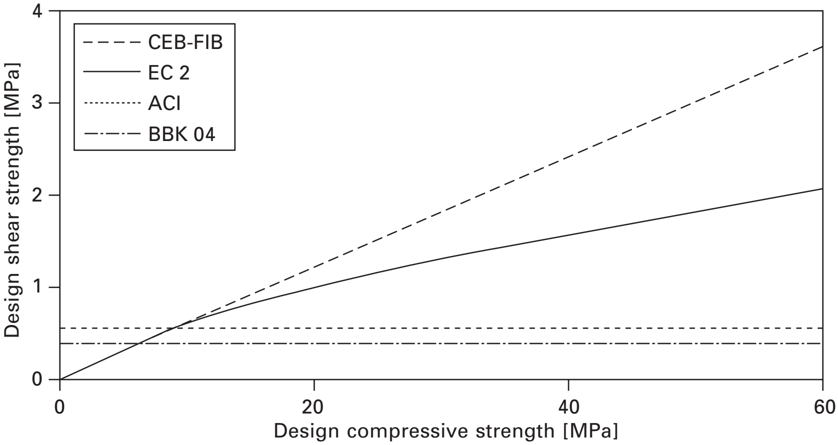
The design of bonded concrete overlays may include thickness design, design of reinforcement for crack control, and design of vertical reinforcement to improve bond strength. The overlay thickness has to be selected considering load-carrying capacity, structural stiffness, geometrical standards, and casting abilities. For sufficient bond strength (see Section 8.6.4), monolithic action between base and overlay can be considered. For such cases, an overlay thickness equalling the thickness of the removed concrete will restore both load-carrying capacity and structural stiffness. Geometrical standards deal with height levels of adjacent structural elements and sufficient overhead clearance under bridges, if any. Finally, the overlay thickness has to exceed the maximum aggregate size of the overlay concrete by at least 5mm.

Design for crack control has to follow the client's requirements or those of any local authorities or international recommendations (e.g. Eurocode 2 or ACI 318). Thin bonded concrete overlays may be cast without reinforcement since the bond to the substrate acts like reinforcement in distributing cracks. The crack spacing s would be limited to three times the overlay thickness (Fig. 8.16) and the crack width can be estimated to $w = s \cdot \varepsilon_{cs}$ where ε_{cs} is the anticipated shrinkage of the overlay. For a 70mm thick overlay with ε_{cs} = 0.6mm/m, the crack width can be estimated to 0.04mm which is usually acceptable

Most international codes accept fairly low values of the design shear stress at the interface in composite concrete structures, and also for rough and sound ones (Fig. 8.17). If the design shear stress exceeds the shear strength of the interface, vertical reinforcement is needed. Examples of reinforcement solutions are given in Section 8.4.17.



8.16 Crack spacing for three different overlay thicknesses: (a) 25mm, (b) 50mm, (c) 75mm. The side length of each specimen is 400mm. (From Laurence *et al.*, 2000)



8.17 Design shear strength as a function of design compressive strength of the concrete overlay according to ACI 318 (1999), the Swedish concrete handbook BBK 04 (2003), CEB-FIP MC 1990 (CEB, FIP, 1993), and BS EN 1992-1-1; 2004 (BSI, 2004).

8.6.2 Material selection

The main recommendation is to select a material that matches the material of the substrate as closely as possible. Since the existing structure is almost always cast in Portland cement concrete, the first choice is just Portland cement concrete. If possible, the concrete aggregate of the overlay should be selected so that it results in a concrete that has a coefficient of thermal expansion similar to that of the existing concrete. For patch repairs, ultra-thin overlays, and other specific cases, materials other than Portland cement concrete may be considered.

8.6.3 Execution

The execution of the concrete removal, cleaning operations, casting, and curing are all of uppermost importance to obtain good and durable concrete overlays with good bond strength. The most important issues may be summarized as follows:

- Determine the cause, degree, severity, and extent of the degradation of the existing concrete structure.
- Plan the repair process considering traffic closing, access roads, environmental restrictions, equipment for concrete removal, removal of asphalt or other materials, transportation of debris and waste water from the site, cleaning equipment, electrical power supply, equipment for concrete casting and curing, measures for quality control, and staff of adequate numbers and with sufficient competence.

- Select a suitable concrete removal method, preferably the water-jet technology.
- Start the concrete removal from the top (highest level) of the concrete bridge or other structure.
- Clean the exposed surface as soon as possible after concrete removal and remove the debris from the site.
- If reinforcement bars are exposed, continue the removal process far enough that the new concrete can surround them.
- Anchor loose reinforcement bars in the base concrete and replace corroded or lost reinforcement with new reinforcement.
- Clean the substrate surface shortly before concrete placement.
- Moisten the substrate surface during the 48 hours prior to concrete placement and let it dry during the last 12 hours (usually the night) in order to create a superficially dry surface.
- Cast the concrete overlay carefully.
- Compact the concrete overlay by using vibration pokers and vibrating screeds (unless a suitable self-compacting concrete is used).
- Cure the concrete overlay properly, start as soon as possible, and continue for at least five days, if possible seven or more.
- Do not expose the new-cast concrete to higher vibration levels than those recommended in Section 8.4.14.
- Conduct pull-off tests if prescribed by the client.

8.6.4 Quality control

The quality control may contain the following items:

- assessment of the base concrete prior to the repair process;
- assessment of the substrate surface after concrete removal and cleaning;
- assessment of the amount and anchorage of the exposed reinforcement;
- assessment of the bond strength.

The assessment of the substrate surface may consist of determining the degree of microcracking, cleanliness, and surface roughness. As stated in Section 8.4.5, there are numerous methods for determining surface roughness, despite its relative insignificance for the bond strength. In contrast, there are no standardized methods for either microcracking or cleanliness.

However, the magnitude of bond strength determines the result of the entire repair process. Consequently, it has been investigated over several decades and using several methods (Silfwerbrand and Beushausen, 2005). The most common one is the pull-off test that either can be conducted *in-situ* or in the laboratory.

The draft of prEN 1504-3 (2001) states that the tensile bond strength should equal or exceed 2.0 MPa on structural repairs and 1.5 MPa on non-structural ones. Considering *in-situ* measurements obtained and the fact that all deteriorated old concrete usually cannot be removed, these requirements may be fairly difficult to fulfil. The Swedish Road Administration (2004) with a long experience on concrete bridge repair, has used, and is still using, the following requirements:

$$m \geq f_v + 1.4 \cdot s \quad [8.1]$$

and

$$x \geq 0.8 \cdot f_v \quad [8.2]$$

where f_v is the required tensile bond strength equalling 1.0 MPa, m and s are the average and the standard deviation ($s \geq 0.36$ MPa) of the measuring values, respectively, and x is a single measured value.

8.7 Future trends and research needs

First, it has to be stated that the need for concrete repair most likely will increase in the near future. Most of the countries in the developed world experienced a construction boom during the first decades after the Second World War. Those structures are now reaching 'retirement age' and have an increased demand for maintenance and repair. In many countries, the new concrete production during a year is limited to 1% of the total stock of buildings and civil engineering structures. Due to its simplicity, the bonded concrete overlay will maintain its position as one of the most frequently used repair methods. Consequently, all developments are of great interest. The use of self-compacting (or self-consolidating) concrete is likely to increase in order to improve productivity and the working environment, and to attract young people to the construction sector, and it will also be used in bonded overlays. The increasing international interest in environment, sustainability, and carbon dioxide reduction will lead to (i) an increased desire to maintain and extend the service life of the structures that are already built, (ii) requirements for more durable concrete structures, and (iii) reducing CO_2 emissions by minimizing concrete volumes (e.g. by reducing cross-sections) and limiting the cement content (e.g. by increasing use of alternative binders). This will also influence the bonded concrete overlays of tomorrow. Finally, it is likely that specialty material, e.g. SIFCON and ultra-high performance fibre-reinforced concrete (Charron *et al.*, 2004), will be used in special cases with high demands, e.g. in severe environmental conditions.

In a key-note lecture, Silfwerbrand (2006) proposed the following research needs:

- Cleanliness prior to overlay placement is the most important factor affecting bond, but we need more knowledge on efficient and cost-effective cleaning methods and how the bond is influenced if the cleaning is not complete.
- Microcracking reduces bond, but how and to what degree does it influence bond reduction?
- Overlay compaction is important for bond development. Which are the most suitable tools for overlays with varying thickness? Can SCC be used instead?
- Curing is beneficial for many concrete properties including bond. More knowledge is desirable to quantify the beneficial relationship between curing and shrinkage crack risk reduction.
- The substrate moisture condition prior to overlay placement has some influence on the bond development. What does the relationship look like and is there an optimum moisture condition?
- Traffic vibrations usually have only minor effects on bond development, but some research reports indicate that there is a time window when vibrations may be detrimental. Does this window exist and how large is its impact?
- Most research indicates that the substrate roughness has only a minor influence on bond, but many practitioners do not agree. Who is right? Does there exist a threshold value defining the border between influence and no influence, i.e. smooth surfaces reduce bond strength but roughness above the threshold doesn't further increase bond strength?
- How high must the bond strength be in order to enable monolithic action? Is this strength dependent on the loading case and the state of stress in the repaired structure?
- Differential shrinkage has been discussed since the 1950s, but still there is no mechanical model accepted by the entire scientific society. Further model development and subsequent model verification by field tests are needed.
- Is debonding a real research issue? Does it exist in reality if the prerequisite for obtaining good bond is fulfilled? Despite this, debonding is an interesting scientific topic. More research is needed for improved understanding.
- The interest in durability issues, e.g. long-term bond strength development, compatibility between substrate and repair concrete, and reinforcement corrosion close to the transition zone, is increasing for environmental and economical reasons.

8.8 Sources of further information and advice

RILEM has an on-going committee dealing with bonded concrete overlays. It will soon publish both a state-of-the-art report and recommendations. Another source of good information on bonded concrete overlays is ACI Committee 546 (2006). Further information on shrinkage, creep, and differential shrinkage can be found in ACI Special Publication No. SP-246CD (2007).

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9
Repair/retrofitting of concrete structures with fiber-reinforced polymers

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Abstract: With their rapid introduction in the construction market and, specifically, in the area of repair and retrofitting deteriorated reinforced concrete structures, fiber-reinforced polymer composites have proved to offer a complementary or even alternative route to existing retrofitting methods. In this chapter, an introduction to repair/retrofitting concrete structures employing fiber-reinforced polymers is presented, followed by an overview of the issues related to the enhancement of member flexural, shear and deformation capacity. The case study of an actual application to a reinforced concrete structure is also presented.

Key words: FRP jacketing, seismic retrofitting, externally-bonded FRP, flexural/shear retrofitting, deformation capacity.

9.1 Introduction

When loss of structural integrity or serviceability is encountered in reinforced concrete structures, remedial measures may need to be taken. Environmental and economic considerations dictate that it is preferable to maintain and upgrade existing structures rather than demolishing and building new ones. The requirements for repair and retrofitting of deteriorated reinforced concrete structures, and the higher public expectation in terms of performance, may be more easily met by a new class of construction materials, the fiber-reinforced polymer (FRP) composites. After their initial introduction into the construction market in early 1990s, the use of FRP composites increased steadily as progress was made in the understanding of their performance.

In this chapter, a brief overview on repair/retrofitting of concrete structures employing FRPs is given. As all of the research aspects and design issues for repair interventions in concrete structures via FRP cannot be fully discussed within this chapter, only the basic principles are presented. Following a brief review on the basics of repair/retrofitting, the strategic decisions, materials and available techniques are summarized. A representative overview of the issues regarding retrofitting to remedy a lack of flexural or shear capacity is given in Section 9.3, while the member deformation capacity enhancement offered by FRP jacketing is presented more analytically. Finally, the case study of an actual application is presented and conclusions are drawn. The interested reader may refer to the bibliography at the end of the chapter for more detailed information.

9.2 Repair/retrofitting with externally bonded fiber-reinforced polymer (FRP) composite

An increasing proportion of the activity of construction industry is now spent on maintenance and upgrading, with important applications to the problems of durability and seismic upgrading. The most prevalent forms of poor structural performance problems are material deterioration, scaling, disintegration, cracking, environmental effects, steel corrosion, concrete delamination and spalling and poor quality of construction. Some of these are discussed more thoroughly in Chapter 10. Another source of the poor performance stems from non-conforming detailing and outdated design criteria stipulated in building codes of earlier times.

Following assessment of the deteriorated structure, it is not uncommon that the extent of intervention required for structural repair is such as to trigger retrofitting as well, when both the present state and predicted future decay are taken into consideration. This step towards retrofitting is taken more easily if retrofitting may be performed with the minimum of intervention and disruption of function and the maximum of effectiveness, longevity and economy, although the cost incurred may be higher. For this reason, combined with other advantageous characteristics, the introduction of FRP materials in the area of repair/retrofitting deteriorated concrete structures has gained wide acceptance in the conservative construction industry.

Following a period of skepticism, reluctance was overcome by the encouraging results of intense research effort, drastic cost reduction, and a combination of favorable characteristics of FRP (high tensile strength, high strength to weight ratio, ease of application, resistance to corrosion, and electromagnetic neutrality). In addition, the application of such a versatile material does not require labor-intensive, time-consuming noisy procedures or expensive equipment. It can easily address issues concerning access limitations, minimization of debris production and space loss and unaltered aesthetics. On the other hand, reduced deformation capacity (absence of yielding or plastic deformation), high cost when calculated on a per-weight basis, thermal expansion properties incompatible with concrete and premature deterioration when exposed to high temperatures, are among the drawbacks of FRPs.

9.2.1 Basic design concepts

In building renovation activities the terms 'repair' and 'retrofitting' are often used indiscriminately, although the underlying concepts are different. According to CEB Bulletin 243 (*fib*, 1998) repair means the 'action taken to reinstate to an acceptable level the current functionality of a structure or its components which are defective, deteriorated, degraded or damaged in some way and without restriction upon the materials or methods employed. This action may not be intended to bring the structure or its components so treated back to their original level of functionality or durability'. On the other hand, 'retrofitting intends the action to modify the functionality or form of a structure or its components and to improve future performance. It relates particularly to the strengthening of structures against seismic loading as a means of minimizing damage during specified earthquake or to increase load-carrying capacity'. Nonetheless, as structures old enough to develop significant deterioration (steel corrosion, concrete spalling, cracking, etc.) normally also lack sufficient earthquake resistance or vertical load-bearing capacity, the need for measures against deterioration often indicates a need for retrofitting as well. Since the implementation of any repair scheme to an existing structure cannot avoid interrupting its function to some extent (perhaps even leading to the evacuation of the structure for a period of time), more often than not owners are faced with the challenge of proceeding to structural retrofitting as well as a repair.

Before proceeding to any form of repair/retrofitting intervention, an identification of the weaknesses and deficiencies to be treated by the intervention should be performed. Such identification can be based on several available assessment procedures which are the outcome of extensive research in the last two decades and have been implemented in many regulatory documents (ATC, 1997; EN 1998-3:2005; NRC, 2004, etc.). Such procedures provide answers not only regarding whether there is a need for retrofitting, but also identify the causes of potential future deficiencies (lack of strength, stiffness, deformation capacity, durability, or any combination thereof). On the basis of a thorough technical assessment, taking into account all design load situations, the objectives of repair are defined and alternative repair schemes are evaluated. The design of retrofitting measures should also ensure that, for the ultimate limit states, brittle modes of failure are suppressed and local failures excluded. Possible constraints affecting the implementation should also be investigated.

If deficiencies in a few components have been identified or if the outcome of the assessment procedure is a lack of deformation capacity in some structural elements, then the engineer may opt for modification of only these components (local intervention), which is easily achieved via FRP jacketing. As FRPs do not influence the stiffness of the structure, they are normally employed for upgrading the performance of individual members. Any existing damage must be repaired before retrofitting by injecting cracks and replacing spalled concrete. More details are provided in Chapter 6.

Even in cases where the assessment points towards global retrofitting measures (for example, the addition of shear walls for structural stiffening and global displacement reduction), it may still be necessary for some members to undergo strengthening (e.g. increase in shear resistance) to cope with the modified building response. As a design principle, regardless of any type of retrofitting selected, local interventions should not impair the safety of part or the whole structure (examples include flexure-retrofitted elements becoming shear-critical, or beam strengthening leading to plastic hinging of columns). Although strength and serviceability requirements are to be respected at all times, it is necessary that the design approach for such interventions should be conservative, owing to the limited existing knowledge of FRP systems with respect to reinforced concrete and to the requirement that some redundancy is always necessary should FRP failure occur.

9.2.2 Materials and techniques

For each application, care should be taken as to the selection of the material and application method. Geometry and member dimensions, current state of member stress, condition of substrate, *in-situ* restrictions, environmental conditions, and experience of the designer and of the personnel are all key parameters influencing the solution to be adopted and, thus, future member performance. Fiber-reinforced composites are based on continuous fibers of carbon, glass or aramid (25–35% per volume for sheets and 50–70% for strips) bound in a matrix. The matrix, used to transfer stresses among fibers and provide protection, is of epoxy, vinylester, or polyester. The composite material is applied to the substrate (concrete or masonry) via an adhesive, to ensure force transfer from the substrate to the composite material. FRP-retrofitting systems comprise three broad categories:

- **Wet-layup systems** – Sheets or fabrics without a matrix affixed to concrete via an epoxy resin that has either been applied on the member before or is used to impregnate the fibers before attaching them on concrete.
- **Pre-impregnated strips ('pre-peg')** – Prefabricated elements applied on concrete via epoxy resin.
- **Special systems** – These include automated application of the FRP jacket, FRP prestressing, special fixtures for mechanical attachment of the FRP, FRP in the form of narrow strips inserted in longitudinal slots along the member (near-surface-mounted reinforcement), systems based on the replacement of resin by inorganic binder (known as textile-reinforced mortar, TRM, fiber-reinforced cementitious matrix, FRCM, or mineral-based composites, MBC), fusion-bonded straps, *in-situ* fast curing through heating devices, vacuum FRP impregnation, etc. The above techniques are briefly presented in *fib* Bulletin 35 (*fib*, 2006).

Although application is easy, many parameters may influence the performance of FRP repair systems, such as substrate preparation, temperature, relative humidity, surface moisture, improper fiber saturation, and curing of the resin. It is thus imperative that installation procedures developed by standardizing bodies and system manufacturers be followed carefully.

9.3 Retrofitting for flexure and shear

Flexural retrofitting of reinforced concrete members can be achieved with the FRP applied to the tension zone of the member. If debonding of the material is prevented, flexural strength enhancement in the range of 10–160% has been documented, albeit at the expense of considerably decreased ductility. For the ultimate limit state in flexure, well-established procedures can be followed and the required quantity of externally-bonded FRP can be determined. The latter is also conditioned by the checks performed regarding the possible failure modes (concrete crushing, steel yielding, FRP rupture, FRP debonding, FRP peeling and combinations thereof), followed by the verification of ductility and adequacy of anchorage length. Note that any increase in flexural strength achieved is almost certainly not accompanied by an increase in shear strength as well and, thus, the retrofitted element may become shear-critical after the intervention. Earlier US ACI 440.2R-21 (2002) provisions and later Italian code CNR-DT-200 (NRC, 2004): 'Instructions for design, execution and control of strengthening interventions by means of fiber-reinforced composites' include detailed design procedures for flexure-retrofitted elements. In the former, based on internal force equilibrium and strain compatibility, the member flexural strength can be determined provided that the controlling mode of failure is identified. The resulting member capacity multiplied by the usual strength reduction factor ϕ (ACI 318-99) and by an additional strength reduction factor of 0.85 (reflecting the lower level of confidence in the FRP reinforcement as compared to the internal tensile reinforcement) is checked against the respective action effects calculated from the factored loads. At ultimate limit state, concrete is assumed to reach its crushing strain (0.003) while FRP develops either its ultimate strain at rupture or a strain level compatible with the mechanism of cover delamination or debonding:

$$\varepsilon_{\mathrm{fe}} = \frac{f_{\mathrm{fe}}}{E_{\mathrm{f}}} = \varepsilon_{\mathrm{cu}}\left(\frac{h-\chi}{\chi}\right) \leq k_{\mathrm{m}}\varepsilon_{\mathrm{fu}} \qquad [9.1]$$

with f_{fe} and E_{f} being the ultimate stress and Young's modulus for the FRP material, $\varepsilon_{\mathrm{cu}}$ the concrete crushing strain, h the height of the section, χ the depth of the neutral axis and f_{fe}, f_{fu} the actual and the rupture strain of the FRP, respectively. The last term in the above equation represents the upper limit of the strain developing in the FRP material, corresponding to early failure due to either cover delamination or FRP debonding. Bond-dependent coefficient k_m assumes values not greater than 0.90 and depends mainly on the stiffness of the laminate.

In the Italian code CNR-DT-200 (NRC, 2004) the flexural capacity of a FRP-strengthened member is given as:

$$M_{Rd} = \psi\, b\, \chi\, f_{cd}(d - \lambda x) + A_{s2}\sigma_{s2}(d - d_2) + A_f\sigma_f d_1 \qquad [9.2]$$

and should not exceed 60% of the initial capacity. In the above equation, b and d are the section width and static depth, respectively, χ is the neutral axis depth, f_{cd} is the concrete design strength, ψ and λ are dimensionless coefficients accounting for the intensity and position of compressive concrete resultant, respectively, A_{s2} is the area of the reinforcement in compression and σ_{s2} and σ_f are the stresses in the compression reinforcing steel and the – adequately confined – FRP material, respectively. As in ACI440, the capacity is assumed to develop either when concrete reaches its ultimate strain under compression or when the composite material reaches its ultimate value defined as the material tensile strain ($n_a \cdot \varepsilon_{fu}/\gamma_f$) or the design strain at debonding (f_{ffd}/E_f), whichever is attained first. Similar to the ACI Committee 440 report, factor n_a accounts for special design issues (long-term loading and environmental conditions), while γ_f, f_{ffd} and E_f are the material partial factor, tensile strength and modulus of elasticity, respectively.

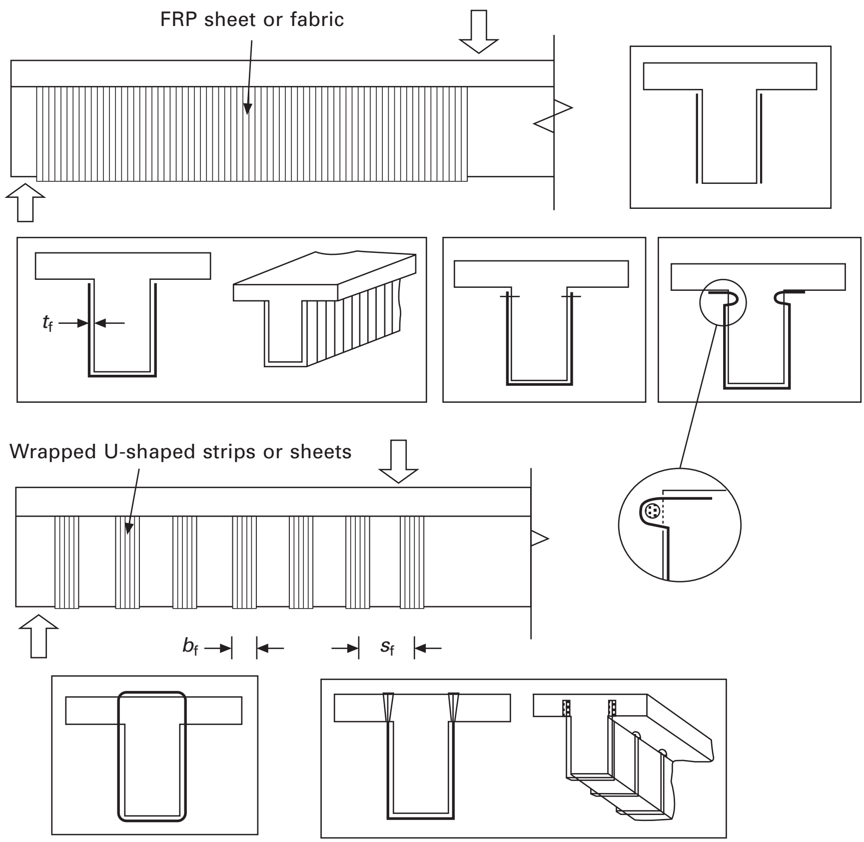
Retrofitting shear-critical reinforced concrete members with externally bonded FRP is far more frequent than flexural retrofitting. Although the effectiveness of the composite material is in this case maximized when the orientation of the fibers is parallel to the direction of principal tensile stresses, possible crack reorientation after initial concrete cracking as well as practical reasons dictate that fiber direction is usually normal to the member longitudinal axis. Several configurations are available, namely side bonding, U-jacketing or fully wrapped FRP sheets (Fig. 9.1a). Experimental studies (e.g. Priestley and Seible, 1995) have shown that, when properly designed and applied, FRP wraps increase member shear strength and ductility (Fig. 9.1b), shifting the mode of failure to flexure. The contribution of FRP to the shear resistance of the retrofitted member can be calculated from available design models (ACI 440-2R.02; EN1998-3, CEN, 2005). Additionally, it has been shown (Triantafillou and Antonopoulos, 2000) that the rigidity of the jacket, FRP failure mode and concrete tensile strength influence the strain developing in the FRP jacket, and design models have been proposed for the calculation of the effective strain corresponding at member shear failure (*fib*, Bulletin 24, 2003).

9.4 Enhancement of deformation capacity

9.4.1 Concrete confinement

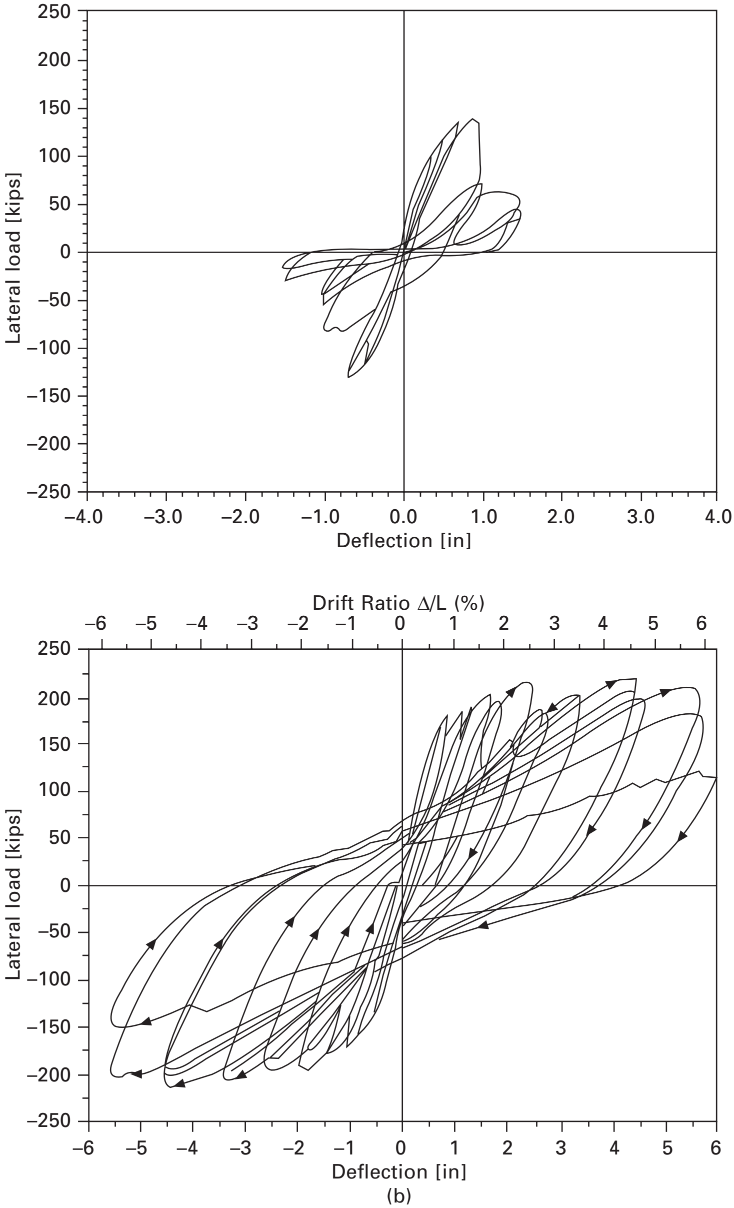
The increase in concrete confinement and the resulting benefits for member deformation capacity are probably the most important advantages of FRP jacketing in repair/retrofitting of concrete structures. A vast number of tests on both concrete specimens and reinforced concrete members has demonstrated that confinement influences many aspects of structural response (flexural and shear strength, bond strength at lap-splices and, mainly, lateral deformation capacity), including the suppression or delay of undesirable modes of failure (e.g. longitudinal bar buckling). The application of FRP material with the main fibers oriented transversely to the member longitudinal axis develops a mechanism of reaction to the transverse dilation of concrete, and provides a confining action against such restraint-sensitive materials. As a direct consequence, both the strength and member deformation capacity are increased (Fig. 9.2).

Due to the elastic-to-failure behaviour of FRP jackets, the resulting



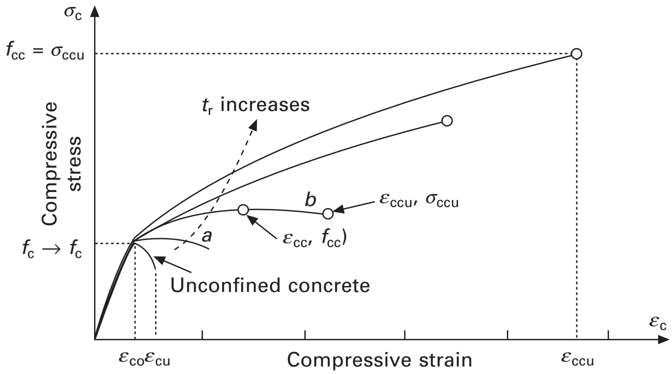
(a)

9.1 (a) Configuration of FRP sheets applied for shear retrofitting. (Image produced with permission of the International Federation for Structural Concrete) (b) Test results on beams: (top) as-built; (bottom) retrofitted in shear with FRP.



9.1 Cont'd

transverse pressure increases continuously, as compared with the constant pressure exerted by steel after yielding. The confining action is expressed as the average value of the transverse pressure in two orthogonal directions, as a result of the lateral pressure exerted by the FRP jacket and by the legs



9.2 FRP confined concrete stress–strain curve. (Image produced with permission of the International Federation for Structural Concrete).

of the stirrups parallel to the FRP fibers. The geometry of the cross-section, the fraction of concrete confined with FRP, and the orientation of the fibers of the FRP material with respect to the member longitudinal axis are the main parameters influencing the efficiency of the confining action by the FRP jacket. In members with circular cross-sections, the confining action is provided by the uniform kinematic restraint around the perimeter (passive confinement). The picture is different in square and especially rectangular sections, as the effectiveness of the FRP jacket in directions orthogonal to the member axis is a function of the size and the section aspect ratio, as well as of the radius at section corners. As confining action in rectangular sections stems from section corners and increases with the corner radius, corner chamfering is crucial in realizing the required level of confinement.

In design, i.e. in determining the FRP jacket thickness/material, the aim is to supply adequate confining action to the section so as to allow attainment of the deformation capacity associated with the prescribed level of displacement demand. Several design approaches exist: the Eurocode 8-Part 3 (CEN, 2005) guidelines, the approach by Mutsuyoshi *et al.* (1999) and the empirical expression by Tastani and Pantazopoulou (2008). For more detailed information, the interested reader is referred to *fib*, Bulletin 24 (*fib*, 2003).

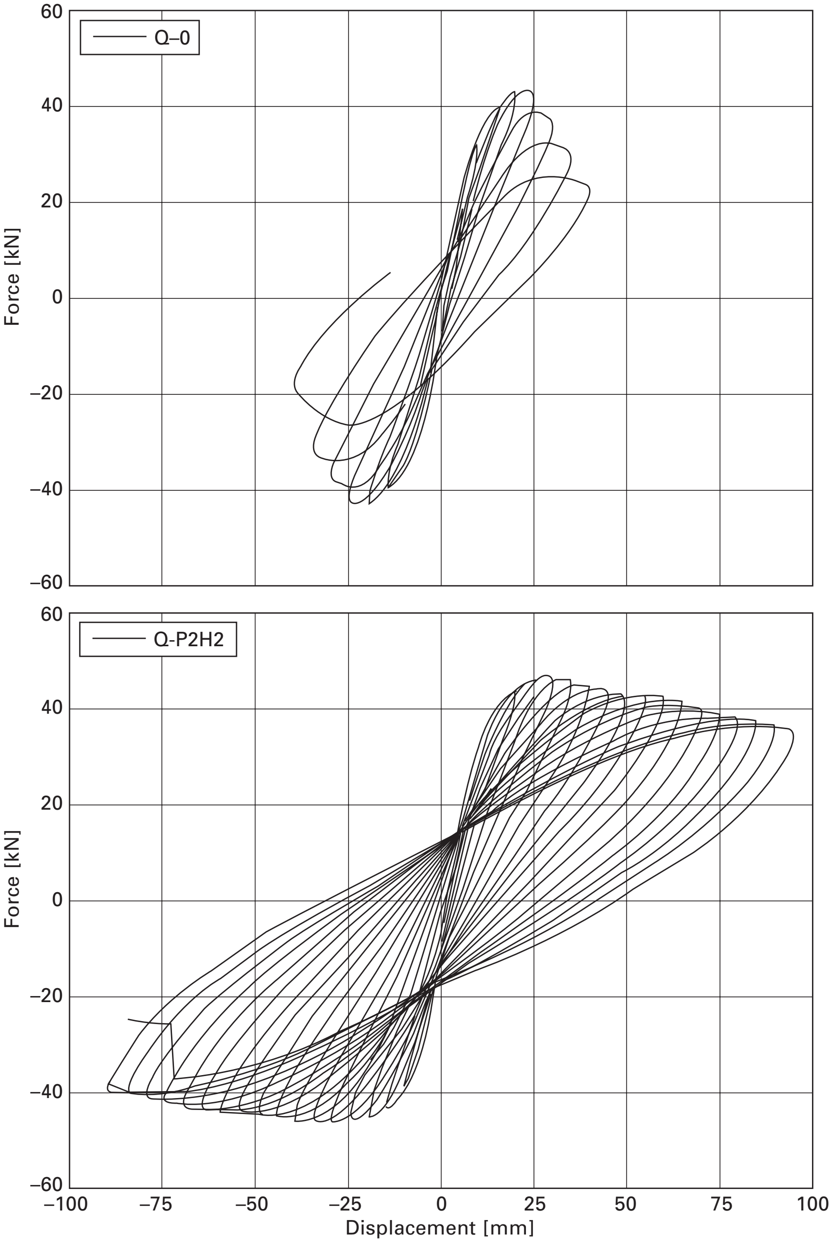
9.4.2 Bar buckling: lap-splice failure

In addition to increasing section confinement and member deformation capacity, FRP wraps are also beneficial in suppressing premature types of failure. Such cases are be related to bond problems and bar splicing at critical member regions without provision of sufficient transverse reinforcement, both being very common in European sub-standard structures built in the 1950s–60s. A number of experimental studies (e.g. Darwish, 2000; Wang and

Restrepo, 2001; Tastani and Pantazopoulou, 2004; Tastani *et al.,* 2006) have demonstrated that members with sub-standard details and, specifically, large stirrup spacing (bar slenderness ratio exceeding 10) may fail prematurely due to buckling of the longitudinal bars. By increasing the failure strain of the encased concrete, FRP wrapping yields a higher concrete strain ductility capacity and, thus, the strain ductility of the bars under compression is also increased while reducing bar 'effective' slenderness ratio (*fib* Bulletin 24, 2003; Tastani and Pantazopoulou 2008).

Although FRP jacketing cannot entirely mitigate buckling of longitudinal reinforcement, it may postpone the occurrence of bar instability until attainment of higher member deformation. This is depicted in Fig. 9.3 (Bousias *et al.*, 2007a) in which the force–displacement hysteresis loops of a sub-standard unretrofitted column (Fig. 9.3a, top) and that of an identical column retrofitted with two layers of carbon FRP (Fig. 9.3a, bottom), are presented. Although the increase in force capacity is about the same in the two columns, the post-peak strength deterioration and the energy dissipation capacity are very different. Moreover, member deformation capacity of the retrofitted column increased by a factor of about 3, achieving a drift ratio at conventional failure (20% drop in resistance with respect to peak resistance) of over 7% as compared to the 2.2% drift sustained by the unretrofitted column. Pictures of the specimens after testing showed (Fig. 9.3b) buckling of the longitudinal corner bars.

The positive contribution of externally bonded FRP in preventing lap-splice failure during the early stages of response (low deformation demands) has been documented by a considerable number of experimental studies (Saadatmanesh *et al.*, 1997; Seible *et al.*, 1997; Chang *et al.*, 2001; Haroun *et al.*, 2001; Bousias *et al.*, 2007a), despite the differences in behavior between deformed bar splicing and smooth reinforcement splicing. When smooth bars are used, the bond resistance available is due to friction at the concrete–bar interface and due to the formation of 180° end-hooks. It is shown (Fig. 9.4a) that a column of square cross-section reinforced with smooth bars spliced for a length of 15-bar diameters at the bottom of the column has very low deformation capacity. The response of this column both in terms of force and deformation is not very different from that of an identical column with a bar splicing length of 25-bar diameters (Fig. 9.4b). When both columns are retrofitted with a jacket composed of four layers of CFRP, it is shown that, for smooth bars with 180° hoops at the ends and lapping of at least 15-bar diameters, lap length does not influence the force and cyclic deformation capacity or the rate of strength degradation in the rehabilitated column (Fig. 9.6). It is noted that the benefits offered by FRP jacketing at bar lap-splicing are most evident only on the corner bars, because it is at chamfered corners that confining action is maximized. The above results have not only been verified by tests on individual columns under uniaxial cyclic deformations,



(a)

9.3 Force–deformation loops (a) and corresponding damage (b) for unretrofitted (top row) and retrofitted (bottom row) sub-standard columns.

(b)

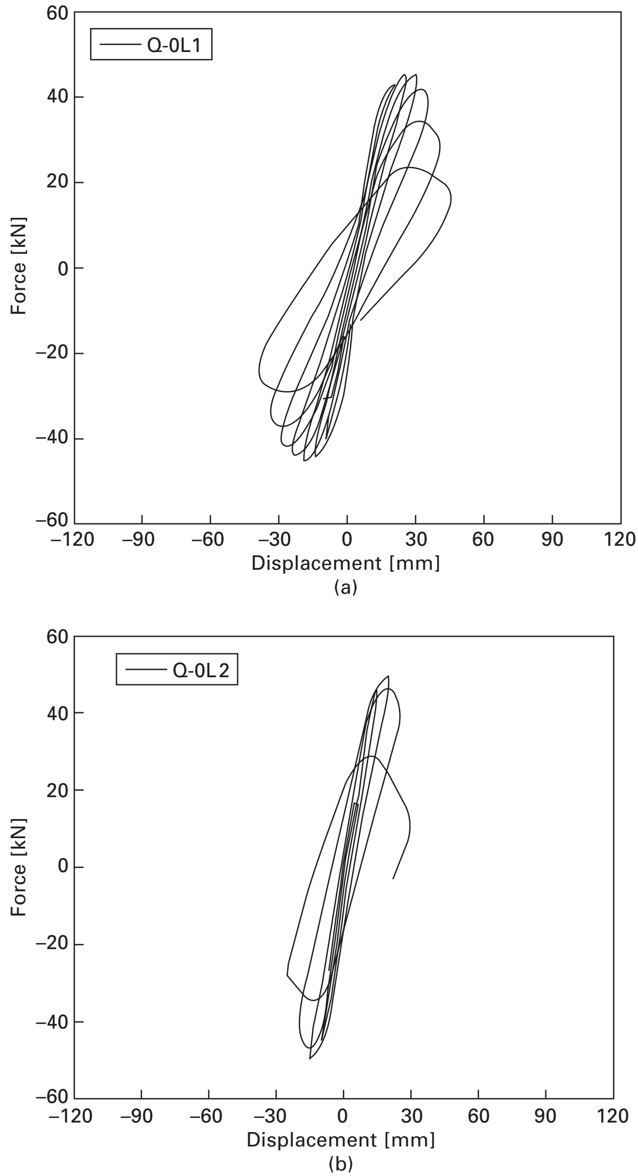
9.3 Cont'd

but also from full-scale tests on gravity-designed-only reinforced concrete structures tested under actual seismic records (Bousias *et al.*, 2007b).

In columns with straight splicing of longitudinal bars, force is transferred from one bar to the other through inclined compression struts joining the tips of bar ribs. Thus, by applying a transverse confining action via the FRP jackets, bond action in columns is improved for bar lapping lengths of at least 15-bar diameters. In fact, FRP wrapping of columns with bar lapping of 15-bar diameters may restore member strength to above 90% of that of a column with continuous bars, while in columns with longer bar splicing, strength increases well above that of the unretrofitted column (Bousias *et al.*, 2007a). It seems that the adverse effects of a lap length of more than 30-bar diameters can be almost fully removed by FRP wrapping but, by contrast, if the lapping is as short as 15-bar diameters, not all adverse effects can be removed by FRP wrapping.

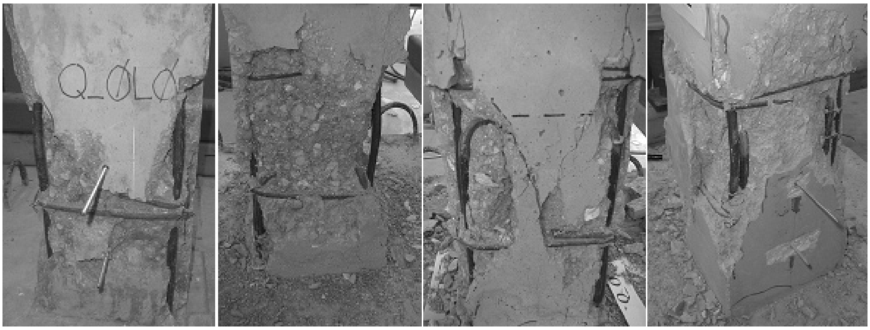
9.5 Repair/retrofitting for durability-related problems

Durability is a design criterion which is at least as important as the criteria for safety and serviceability, and implementing appropriate criteria for durability is a must. Old, sub-standard concrete structures often suffer both



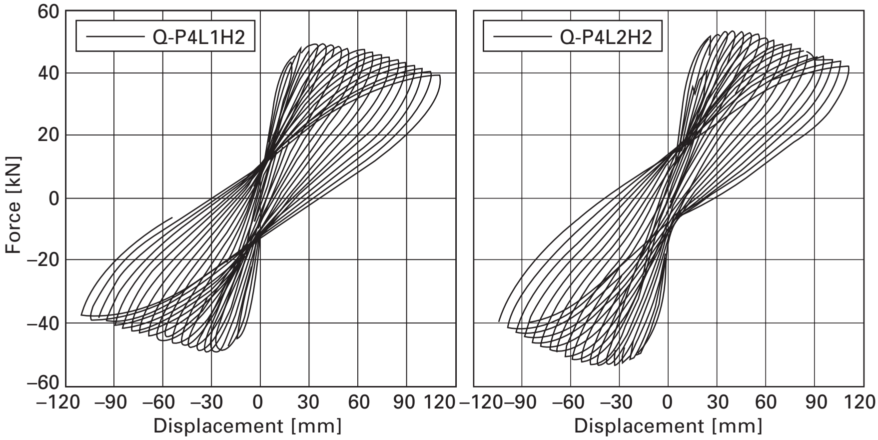
9.4 Force–deformation loops for as-built columns with smooth lap-spliced bars; lap length (a) 15-diameters and (b) 25-diameters.

from deficiencies in member strength and deformation capacity and from reinforcement corrosion. It is a known fact that rebars are well protected from corrosion when surrounded by concrete; the alkaline environment of concrete successfully protects the passive layer of rebars. Carbon dioxide



(a) (b) (c) (d)

9.5 Damage of unretrofitted specimens for (a), (b) 15-bar diameters, (c), (d) 25-bar diameters.



(a) (b)

9.6 Force–displacement response of specimen with lap length (a) 15-bar diameters and (b) 25-bar diameters.

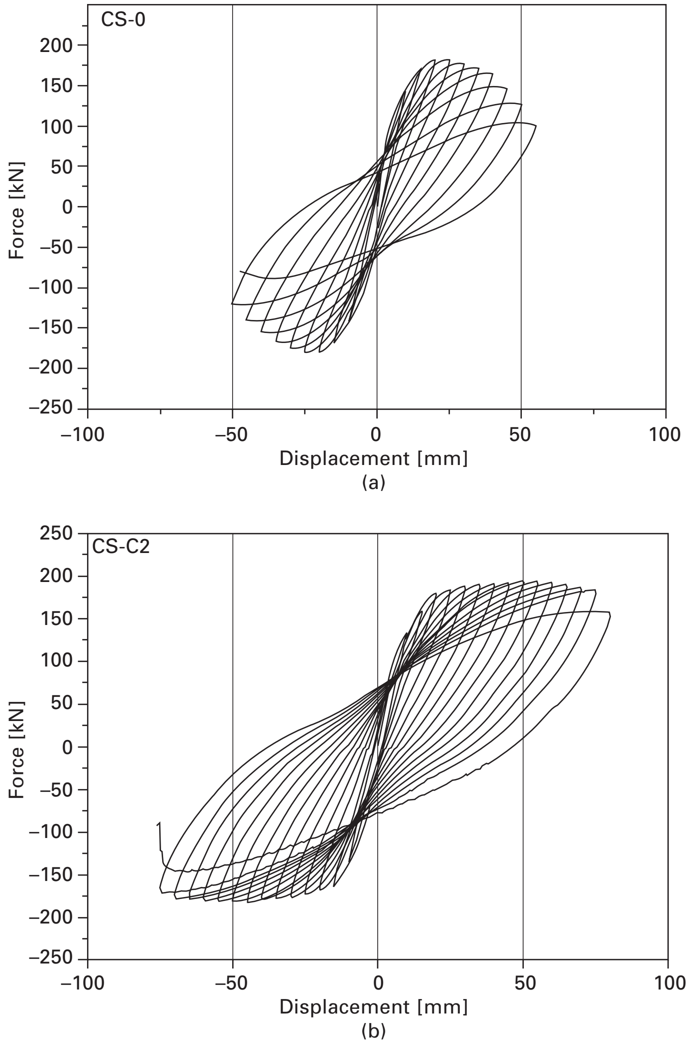
and chlorides are the two common causes of cement matrix neutralization, leading to bar corrosion and member degradation. As corrosion products occupy almost 6–8 times the volume of the parent material, tensile stresses develop in the concrete that may subsequently force the concrete to spall (especially if stirrups are sparse) and allow a higher diffusion of water and oxygen to further accelerate deterioration. As a result, corrosion reduces member strength due to steel area loss, adversely affects bond and anchorage, makes bars more susceptible to buckling and reduces steel ductility. Such degradation is exacerbated by the freeze–thaw and dry–wet cyclic exposures causing accelerated ageing of structures over time. Transverse reinforcement is more vulnerable to corrosion because it is of smaller diameter and closer to the concrete surface than longitudinal reinforcement and, thus, its contribution to confinement decreases. In Europe, the problem is considered to be aggravated by the widespread use of the more corrosion-prone tempcore S500 steel since the late 1980s.

Traditional techniques (removal of spalled concrete and repairing with mortar) are considered to be impractical and expensive, while the efficiency of electrochemical measures against rebar corrosion is not commensurate to their cost. As structures old enough to develop significant reinforcement corrosion normally lack sufficient earthquake resistance, the need for measures against the on-going corrosion often paves the way for seismic rehabilitation as well. When retrofitting against corrosion is realized through jacketing, FRP wraps have a proven record of positive contribution (Bonacci and Maalej, 2000; Soudki and Sherwood *et al*., 2003; Tastani and Pantazopoulou, 2004) and can provide confinement activated by lateral expansion of the concrete.

Tests on corrosion-damaged columns that were subsequently strengthened with two layers of carbon FRP wraps (Bousias *et al*., 2004) showed that the conventionally defined deformation capacity of columns increased by 50% (Fig. 9.7). Corrosion seems to cause reduction of the elongation capacity of reinforcing bars and early rupture, thus preventing the full use of the strong confining effect of FRP wraps. The same study showed that, had glass-FRP jackets been used instead of carbon-FRP, the performance would have been approximately the same, provided that the same extensional stiffness in the circumferential direction was provided.

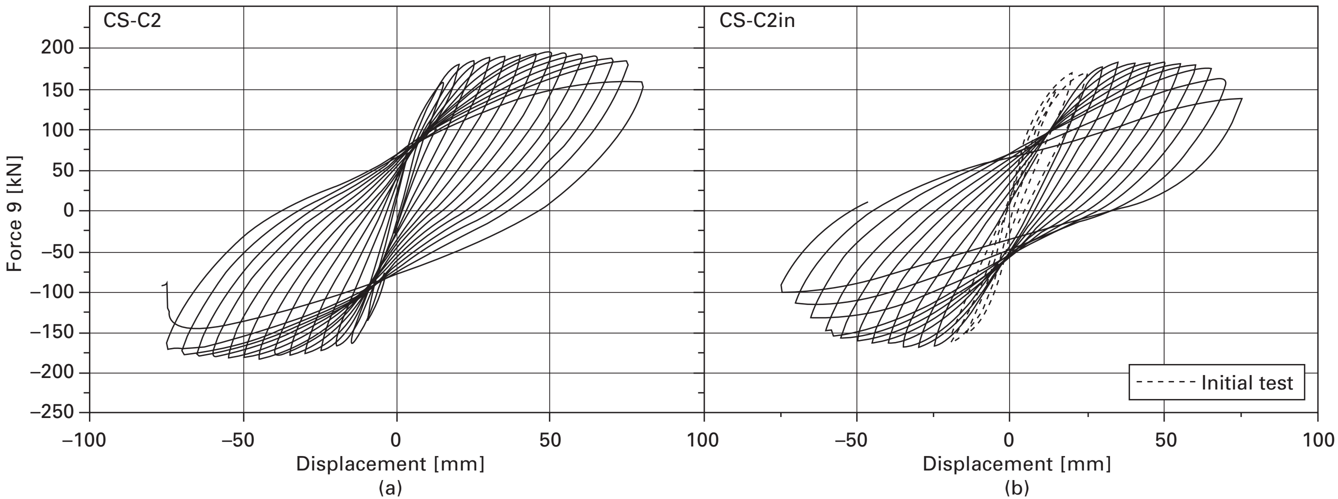
However, contradictory experimental information is available on the durability of repaired corrosion-damaged structures. Results of some studies (Berver *et al*., 2001; Pantazopoulou *et al*., 2001; Soudki and Sherwood, 2003; Tastani and Pantazopoulou, 2004) indicate that FRP reduces the rate of corrosion in reinforcing steel by creating a non-permeable barrier that minimizes the ingress of corrosion agents. Others (Emmons *et al*., 1998) maintain that, in some cases, the application of a barrier to moisture and oxygen can significantly change the corrosion mechanism in the underlying concrete; in that respect it was suggested that strengthening systems do not arrest corrosion in already contaminated concrete and that the corrosion problem should be investigated and addressed before that application of the strengthening system. Although FRP wrapping is considered as a practical means of restoring capacity lost due to corrosion and of upgrading earthquake resistance at the same time, the long-term impact of the FRP systems needs further investigation and, specifically, the synergistic effects of continued corrosion on the performance of the repaired system. Durability of repairs is addressed in Chapter 12.

Using fiber-reinforced composites for repair entails also the treatment of previously inflicted damage to concrete. Existing damage of sufficient extent to have caused concrete cover spalling and yielding of the longitudinal reinforcement may endanger the effectiveness of retrofitting with FRP



9.7 Force–deflection loops of specimen with corroded reinforcement: (a) before retrofitting, and (b) after retrofitting.

jacketing. If FRP wrapping takes place without repair of the damage (other than the treatment of the column surface necessary for application of the FRP), then a column that is retrofitted after being damaged beyond yielding of the reinforcement exhibits more rapid strength loss and lower deformation capacity compared with a previously undamaged, retrofitted one (Fig. 9.8). This may be attributed to the fact that, if the concrete has undergone some permanent lateral expansion in the absence of the FRP jacket, when confined



9.8 Force–deflection loops of specimens with corroded reinforcement and retrofitted with two carbon FRP wraps: column (a) with initial damage; column (b) without initial damage.

afterwards with a jacket, it will reach its crushing strain with less activation of the FRP wraps and will benefit from them. It is also important to note that, in columns with rectangular sections, the difference in performance between previously damaged and undamaged columns is much larger when the response is in the column strong direction, where – due to its narrower compression zone – the column benefits most from confinement by the FRP.

9.6 Practical aspects: detailing

Whenever FRP wraps are applied to concrete members, a number of important practical issues arise, such as surface preparation, requirements for anchorage of the FRP sheets, previous damage inflicted on the member, spalled concrete cover, and corner chamfering. Since FRP retrofitting is a bond-critical application, the existence of a sound concrete substrate (of not less than 17 MPa compressive strength, according to ACI 440-2R02), is of vital importance for the effectiveness of any repair/retrofit of concrete members with FRP systems. In addition, proper preparation and surface profiling are equally important: irregularities should be filled or smoothed, and cracks larger than 0.3mm that may cause delamination of the FRP or fiber crushing should be repaired beforehand. The surface must be cleaned of all laitance, dirt, oil, and existing coatings by appropriate means. After proper surface preparation, the FRP is applied following the manufacturer's specifications.

When FRP jacketing is employed for strengthening members in shear, it is recommended that anchorage be achieved through extending and adhering the FRP sheets or strips in the compression zone either directly or, if this is not possible (e.g. in T-beams), sheets may be curved within special grooves at the top of the web (see *fib*, Bulletin 24, 2003).

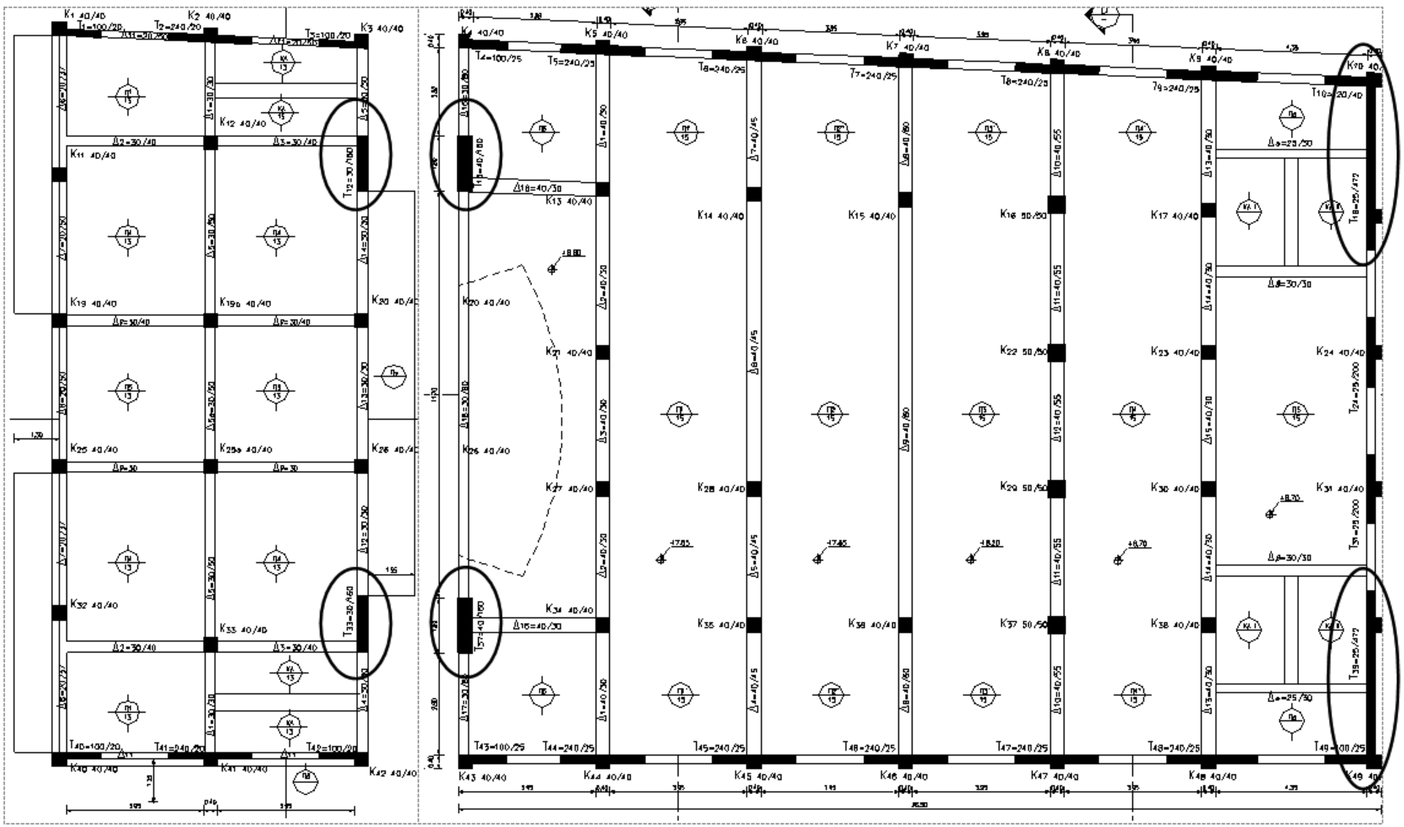
9.7 Application example

The example presented concerns a reinforced concrete building housing a theater and constructed in the highly seismic area of the island of Kephalonia (Greece) in the early 1970s (Kosmopoulos *et al.*, 2007). Visual inspection revealed extensive vertical cracking at the corners of perimeter vertical members (Fig. 9.9) due to severe reinforcement corrosion. The building, designed according to codes of the 1950s without detailing for ductility, was grossly inadequate against the requirements of current seismic design provisions. Furthermore, the strong irregularity – in plan and elevation – of the two parts of the building increases its seismic vulnerability. Because it was necessary to repair members suffering from reinforcement corrosion, it was decided to upgrade the entire building. The evaluation of the building was performed according to Part 3 of Eurocode 8 via non-linear dynamic seismic response analyses and pushover analyses. The outcome was that a low seismic action (peak ground acceleration – PGA = 0.1 g) in any of the two horizontal directions yields demands that exceeded the specified limits for Life Safety or Near Collapse in some members. The two pairs of shear walls next to the expansion joint separating the two parts of the building (Fig. 9.9b) were identified as likely to fail in shear, while in the left part, the critical walls were the two interior ones parallel to the joint. In the right part, the two exterior walls normal to the joint were proven to be shear-critical.

To raise resistance and lateral stiffness to the level required, it was decided that strengthening and stiffening the exterior U-shaped shear walls on each of the two longer sides of building were necessary. A 150mm thick shotcrete overlay was added to the external surface of the walls, converting them into rectangular walls with uniform thickness. In addition, the second and fourth

(a)

9.9 (a) Vertical cracking at corners of perimeter vertical members (b) Framing plan.



(b)

9.9 Cont'd

bays of the 5-bay frame on the left-hand side of Fig. 9.9(b) were filled in with reinforced concrete, creating two new 500mm thick shear walls that encapsulate the existing columns at their two ends, as well as any beams between these columns. The above rehabilitation measures above proved insufficient for the prevention of shear failure of the two large walls of the façade as well as of the two pairs of interior walls on either side of the expansion joint (Fig. 9.9).

Reinforced concrete jackets or overlays could not be used because of lack of access to the foundation at these points to enable connection of the jackets to the foundation, as well as the architectural restrictions regarding intervention at the façade and disturbance and the disproportionately high costs entailed in the construction of jackets or overlays of cast-in-place or sprayed concrete. Carbon-FRPs were thus used to correct the shear deficiency of the two large walls of the façade and of the two pairs of interior walls on either side of the expansion joint. Horizontal sheets of carbon FRP were bonded to the exterior face of the two large façade walls and to the surface of the accessible long sides of the two pairs of interior walls. The total thickness of CFRPs was proportioned on the basis of the mean deficit of shear strength in the corresponding wall. The anchorage of the ends of the one-sided CFRP sheets at the re-entrant corners between the web of the façade wall and the column-like square barbells protruding from its exterior surface was achieved through 100mm long spike CFRP anchors placed in 100mm deep holes filled with epoxy.

9.8 Future trends

Despite the rapid evolution and expansion of the application of externally bonded composite materials to the repair/retrofitting of concrete structures, its full potential has not yet been realized. New developments continue to appear, such as near-surface-mounted reinforcement, steel-reinforced polymers, textile-reinforced mortar, etc.

Near-surface-mounted (NSM) reinforcement is an alternative to flexural strengthening via externally bonded FRP products (plates or laminates); it basically consists of inserting bars or laminates inside surface grooves sawn on the side face of the members and using adhesive for the attachment to concrete. Although this technology offers a decreased risk of premature debonding, its application procedure is not trivial: a groove is required to be created on the side of the element and filled halfway with epoxy paste, before the FRP reinforcement is placed. Furthermore, its effectiveness depends strongly on the groove depth and bond conditions within the cover of member cross-section where it is inserted. Some of the disadvantages may be alleviated by wrapping the anchorage zones of the NSM with FRP jackets, allowing the development of higher inelastic displacement and flexural member strength and delaying failure due to debonding.

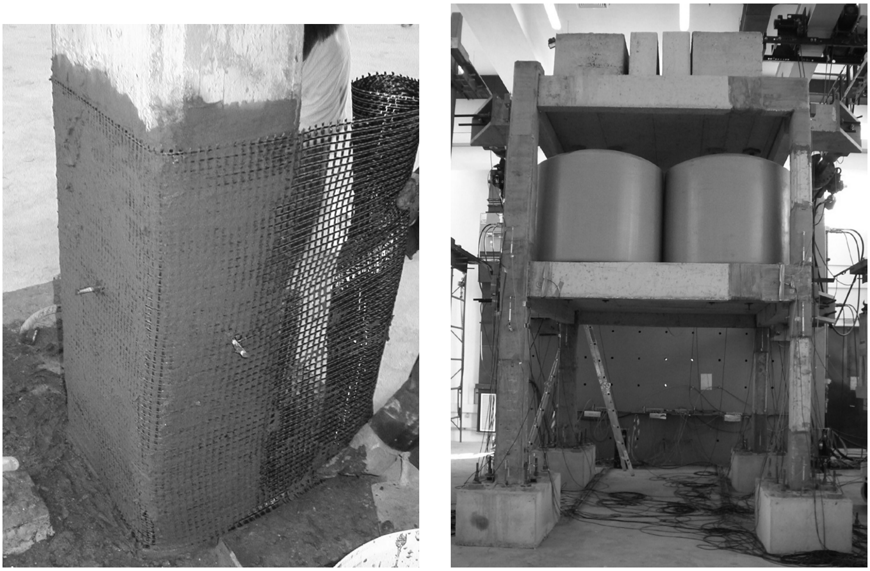
Steel-reinforced polymer (SRP) sheets, in which the glass, carbon, or aramid fibers have been substituted by high-strength steel wires, represent an alternative to CFRP jacketing for flexure (Barton *et al*., 2005). The steel cords are coated with either zinc or brass and then aligned to form a steel tape with very high strength and stiffness. The result is an economical and very tough material, which is applied with a process similar to that for FRP sheets (adhered via epoxy resin or mortar) and behaving in a similar manner. It has been shown (Prota *et al*., 2006) that SRP sheets provide strength enhancement similar to that provided by FRPs, but appear to be more efficient in terms of deformation (higher ductility capacity). Further research is needed regarding the long-term behavior (durability) and fatigue response of the systems, as well as prevention of delamination when it is applied on concrete by cementitious grout.

A major drawback of FRP repair/retrofitting systems is the organic resins (typically epoxy) used to bind the fibers. An alternative to FRP repair/retrofitting material, on which considerable research effort is being concentrated, are the so-called textile-reinforced mortars (TRMs) or fiber-reinforced cement mortars (FRCM): they comprise textiles (fabric meshes made of fiber rovings) with fibers oriented in at least two directions, impregnated with inorganic binders such as cement-based mortars (Triantafillou and Papanicolaou, 2005; Bournas *et al*., 2007). If the cementitious matrix used possesses certain characteristics (non-shrinking, workable, viscous, with sufficient tensile strength), then debonding is avoided and the interlock of the mortar–grid structure is achieved.

The effectiveness of TRM applied as jackets for repair/retrofitting reinforced concrete structures has been explored basically through testing sub-elements but also nearly full-scale structures. A two-storey, one-bay per direction reinforced concrete structure constructed in the laboratory was tested under earthquake excitation employing the pseudodynamic testing method, and subsequently repaired/retrofitted with TRM jacketing applied at the bottom and the top of sub-standard columns (Fig. 9.10). The retrofitted structure sustained three consecutive rounds of testing imposing a Eurocode 8 compatible earthquake record with peak acceleration of 0.35 g, 0.45 g and 0.55 g PGA. It was shown that the TRM-retrofitted ground storey columns sustained drift demands of about 3%, with concurrent drift in the orthogonal direction increasing from almost nil to 0.5% due to the progressive development of twist. Despite the increased deformation demands in the confined column-end regions, neither jacket rupture nor bar buckling was observed. More details on the testing program and the experimental results can be found in Bousias *et al*. (2007a).

9.9 Sources of further information and advice

There have been a large number of experimental results and analytical studies published in reports, conference proceedings, and journal papers regarding



(a) (a)

9.10 Application of TRM jacket (a) at column base and (b) retrofitted structure before testing.

FRP-retrofitted reinforced concrete elements. A list is provided at the end of this chapter. Two review articles, one by Neale (2000) and the second by Triantafillou (2001), provide the basic concepts and design guidelines for the application of FRP for retrofitting concrete structures, while in the article by Hamilton and Dolan (2000) a detailed discussion on durability issues is provided. Bulletins 24 and 35 of *fib* (*fib*, 2003, 2006) provide a very comprehensive coverage on repair/retrofitting of concrete structures with externally bonded FRP, albeit with emphasis placed on seismic applications. An early and very informative report covering the design and construction of externally bonded FRP systems is that by ACI Committee 440 report (ACI 440.2R-02, 2002 – the report is currently under revision). Finally, in the Proceedings of a recently held International Symposium on 'Fiber-Reinforced Polymer Reinforcement for Concrete Structures- 8' (Triantafillou, 2007), the interested reader may find information on the state-of-the-art in this field.

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10
Protection systems for concrete structures

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Abstract: This chapter addresses protection systems in the form of polymer overlays and moisture barrier systems. Protection systems provide a barrier between the concrete and its environment and are an integral part of their life line.

Key words: polymer overlays, surface preparation, epoxy, polyester, polyurethane.

10.1 Introduction

Concrete structures built around the world are subject to a wide range of differing conditions of use and exposure to environmental conditions including, but not limited to, erosion, impact loads, weather, and pollution. These factors, coupled with the quality of construction built into the structure, mean that the time to initial deterioration may vary widely. Protection systems are intended to provide a barrier between the concrete and its environment as well as the operational demands imposed on the structure. The barrier will extend the time to initial deterioration.

The protection, repair and maintenance of concrete structures are integral parts of their life line. Owners have choices to make when it comes to protection and repair. New concrete structures do not normally receive any type of protection. There are a wide variety of reasons offered for this decision; however, it normally comes down to economics and the wishful conclusion that concrete will not need any significant maintenance. History has shown that structures receiving a protective coating or overlay will have a longer life with lower maintenance costs than those that receive no type of protection.

The normal maintenance cycle starts with localized repairs. As owners become aware of the high cost of repair, the protection options become more appealing and their long-term benefits far outweigh their upfront costs.

This chapter addresses protection systems in the form of polymer overlays and moisture barrier systems. The implication is that these procedures are carried out after a structure has been in service and deterioration has started. The polymer overlays or the moisture barrier systems are then applied in an effort to further extend the life of the structure following the repair of the structure. These protection systems may be applied to horizontal surfaces exposed to pedestrian or vehicular traffic, chemical or mechanical erosion as well as vertical surfaces exposed to an aggressive chemical environment.

10.2 Surface preparation

Proper surface preparation is critical to the successful application of any system that is to be applied to a concrete surface. The protection systems are usually applied to horizontal surfaces; however, sealers that are applied as moisture barriers may also be applied to vertical surfaces.

Prior to the preparation for the application of a polymer overlay, all repairs required to re-establish the original condition of the bridge must be completed. That means all deteriorated concrete on the surface and all corrosion activity in the reinforcing steel must be addressed and repairs completed. Any corrosion activity not addressed will continue after the polymer overlay has been applied. This condition will shorten the life expectancy of the overlay and increase the life-cycle costs.

The surface preparation requirements will vary from a light removal of surface laitance for sealers, to the exposure of fine aggregate for coatings or polymer overlays and exposure of the coarse aggregate for premixed polymer concrete overlays. This may be accomplished by sand blasting, shot blasting or hydro blasting. In all cases, the removal of contaminants must be included in the removal process.

10.2.1 Light blast

The laitance coat of Portland cement is readily removed by a light sand blast or shot blast. It may also be removed using water blasting with an approximate pressure of 21–34 MPa (3000–5000 psi). These techniques will leave a relatively smooth surface with an open pore structure in the concrete and expose the top surface of the fine aggregate. This procedure will commonly remove deleterious materials on the surface.

10.2.2 Moderate blast

For the application of polymer overlays or coatings, the objective when specifying a moderate blast is two-fold. The first is to remove any deleterious materials that are on/in the concrete surface and the second is to ensure that the surface is rough enough to ensure a good bond between the surface applied material and the substrate. This will be achieved by exposing the fine aggregate. The degree of exposure should be consistent with the recommendations of the material supplier.

10.2.3 Heavy blast

For polymer concrete overlays that are premixed systems, a surface with a greater amplitude will be recommended. This will mean the exposure of coarse aggregate as well as the fine aggregate. The specified amplitude should be consistent with the recommendations of the material supplier.

10.2.4 Chemical preparation

The use of muriatic acid has been recommended by suppliers over the years. The acid will consume the cement paste, subsequently exposing a fraction of the fine aggregate. The acid will not remove some deleterious materials, such as oil, that have penetrated the concrete surface. The use of muriatic acid thus should be avoided unless there is no other acceptable solution that may be applied. The reason for avoiding muriatic acid is that there is a high probability of residue acid being retained within the pore structure of the concrete that will continue to erode the cement paste and undermine the adhesive properties of the overlay system at the bond line after the coating has been applied. In addition, any oil that remains will also act as a bond breaker between the coating and the substrate.

10.2.5 Final clean-up

When sand blasting or shot blasting are used for the preparation, all residue particulate matter remaining on the surface must be removed. This material will appear as a coating of dust. The surface should be thoroughly washed using high-pressure water of 7–17 MPa (1000–2500 psi). This procedure will remove the loose particulates and leave a surface free of materials that would otherwise impede the penetration of liquid sealers or the bond of coatings or polymer overlays.

10.3 Polymer overlays

10.3.1 Introduction

Polymer overlays are primarily applied to bridge decks, parking garage decks and structural floor systems exposed to aggressive environments. They are systems designed to provide protection against applied loads and erosion from use as well as to protect the concrete structure against attack from its environment. While concrete is subject to attack from its environment, it has the ability to resist the elements that it is exposed to for a long time before repairs will be required. The polymer overlay systems may be applied to new structures or existing structures. When applied to an existing structure the application will normally follow repair of any localized areas exhibiting deterioration.

Polymer overlays provide a dual purpose. First, they provide an impervious layer that will act as a moisture barrier to the penetration of aggressive chemicals or moisture that may be laden with deleterious materials that might be harmful to the concrete or any of its constituents or embedded materials. Second, they are a modified version of a mortar or concrete that provide a protective layer capable of resisting abrasion from traffic, either pedestrian or vehicular.

The polymer overlay systems can be applied to new structures as a means of reducing the long-term maintenance costs as well as extending the life expectancy of the structure. The concrete in a new structure is sound and free of any materials that would attack the concrete or any embedded materials such as the reinforcing steel. When applied to a new structure, the initial deterioration of the concrete system will be delayed for an extended period of time.

By delaying the application of a protection system on a newly built structure, the upfront costs have been reduced. There may be many good reasons for this decision; however, the reality is that the deterioration of the concrete system will be initiated at a much earlier age and extensive repairs will be needed much sooner. Consequently, the life time costs of this structure will be higher.

Owners will more commonly apply a protection system to a structure following its first major repair and rehabilitation. (This is synonymous with the philosophy of 'locking the barn door after the horse has been stolen'.) The application of a protective barrier will both extend the time to the next major repair and extend the life expectancy of the structure.

10.3.2 Selection of polymer overlay systems

There are three primary systems customarily used. Although there may be a wide variety of chemical modifications to the systems that are available, this chapter will deal with the basic definitions, i.e. polyurethane, epoxy and polyester. The manufacturers of these systems will modify the chemical composition to address specific needs, conditions or uses. Polyurethanes are the most commonly used systems for the protection of parking garages with epoxy systems increasing in use. Epoxy and polyester systems are more commonly used on bridge decks.

10.3.3 Polyurethane overlay systems

Polyurethanes provide an impervious membrane that is treated with sand to create a skid-resistant surface. Polyurethanes are primarily used on parking decks. These systems are designed to prevent the ingress of chemicals that would accelerate the deterioration of the concrete deck. Chemicals that would attack the concrete and/or embedded reinforcing steel, such as salts, may be applied directly, by the owner, or indirectly from autos and/or the atmosphere. The degree to which polyurethane deck coating systems are capable of withstanding abuse due to abrasion will vary depending upon the type of abuse and the applied thickness of the system.

Conditions of use

Since the primary use is to protect the concrete in parking garage structures from deterioration, there are generally two areas of consideration, i.e. the parking stalls and the driving lanes. Those geographic areas that are subjected to freezing and thawing, usually north of the 40th parallel, commonly have salts applied to melt any snow or ice. Other areas of consideration are the coastal regions that will be subjected to salt water. The polyurethane system will also protect against the ingress of aggressive airborne chemicals that may attack the concrete matrix leading to the deterioration of the concrete.

The parking stalls receive much less physical abuse than the driving lanes. They are subjected to very slow vehicular motion, with the greatest amount of abuse being derived from the turning of the wheels as the driver enters or departs from the parking stall. The salt-laden snow and ice that accumulates in the wheel wells will commonly melt during the stationary period and deposit the salt-laden water to the surface around the vehicle wheels. The polyurethane is intended to prevent the ingress of the salt into the concrete, where it will help to increase the rate of corrosion of any embedded reinforcing steel.

The conditions of abuse will increase if studded snow tires are used on vehicles. The studded snow tires have a higher propensity to abrade the polyurethane, consequently tearing the coating and exposing the concrete to attack from the salts or airborne chemicals.

The driving lanes, however, are subjected to erosion due to abrasion from the vehicle wheels. The rate of abrasion will vary between the driving lane and the turning area. The turning areas receive a much higher rate of deterioration. Again, that rate is increased if studded snow tires are used.

Polyurethane overlay application

Prior to any application of a polyurethane coating system, the surface must be properly prepared. The objective is to remove any deleterious materials and the weak layer of laitance while providing a surface with the fine aggregate exposed. This may be accomplished by a variety of different techniques;

however, shot blast is the most preferred solution since it is the least expensive and is environmentally friendly as well as being the most effective.

The polyurethane coating system is applied in multiple layers. The number of layers will be dictated by the level of abrasion to which the system will be subjected. The minimum coating system consists of (i) prime coat, (ii) base coat, (iii) wear coat, (iv) finish coat for a total thickness of 1.5mm (60 mils). The wear coat will have had a fine aggregate broadcast into the layer at a rate of 50–75 kg/m^2 (10–15 lbs per sq. ft.). Areas subjected to heavy abrasion may have one or more additional wear coats added. This basic system will have approximately 1mm (40 mils) dry thickness. The inclusion of the aggregate will increase the total dry thickness of the completed system. For each additional wear course there will be approximately an additional 0.3mm (12 mils) of dry film thickness.

A typical system will be applied with one wear coat for light vehicular traffic areas including parking stalls and two wear coats in heavily trafficked driving and turn lanes. Additional wear coats may be applied to resist areas that will be subjected to aggressive abrasion conditions such as turn lanes in high traffic areas and studded snow tires.

10.3.4 Epoxy overlay systems

The two areas in which the epoxy overlays are most commonly applied are bridge decks and parking garage decks. The epoxy overlay systems produce an impervious membrane integrated with fine aggregate to create a barrier against the intrusion of chemicals into the concrete as well as a wear course to protect and extend the life of a concrete surface. These coating systems may be applied with a single layer application or multiple-layer applications. Premixed epoxy concrete/mortar systems may also be used on bridge decks.

The single application system may be used on either bridge decks or parking garage decks. The difference between the two applications is the size and gradation of the aggregates that are applied. The bridge deck applications use a larger aggregate than is customarily used on parking garage decks. The multiple application system, like the single application system, may be used on bridge decks or parking garage decks. The increased thickness will provide a more durable protective coating capable of withstanding a very abrasive environment.

Conditions of use

The primary purpose for the application of an epoxy overlay system is to protect the concrete and reinforcing steel against attack by aggressive chemicals and abrasion. The two structural systems in which epoxy overlays are commonly used are bridge decks and parking structure decks.

The bridge deck coating system may be single or multiple layers. The single layer system will commonly use a coarser aggregate than the multiple layer system. Although a premixed epoxy mortar/concrete system is an option, I am not aware of any applications in North America.

For parking garage decks, the single or multiple applications are appropriate. As with the polyurethane coating, the levels of wear and abuse will vary depending upon the location of the application. Parking stalls receive the least amount of wear, driving lanes have an increased level of abrasion and the turning lanes receive the highest degree of abuse. Epoxy overlays have a greater ability to resist abrasion than the polyurethane system; consequently, a single application of an epoxy overlay is usually sufficient.

The conditions of abuse will be more severe in geographic regions that permit the use of studded snow tires. Although the use of studded snow tires is diminishing in favor of all-weather tires, they are still in use. In these geographic regions, multiple layers will be a good investment. The multiple layers in parking garages, when applied, are usually restricted to drive and turn lanes.

Epoxy overlay application

Prior to the application of an epoxy overlay system, the surface must be properly prepared to assure the integrity and longevity of the applied system. This includes the removal of all deleterious materials that are on and/or in the concrete at and below the surface. This may be achieved by sand blasting, scarifying or shot blast. The final surface condition should be free of any deleterious materials and reveal the fine aggregate and coarse aggregate surface. Immediately prior to application of the first coat of epoxy, the surface should be cleaned and free of all loose particulates.

The epoxy overlay systems are customarily applied in single or multiple layers. The first coat of epoxy should be applied uniformly at a rate consistent with the manufacturer's recommendations. The specified aggregate will then be immediately broadcast into the wet coat of epoxy until there are no visible wet spots. Following the recommended curing period, the excess aggregate shall be removed from the surface. The finished thickness will be dictated by the size of the aggregate broadcast into the epoxy.

If additional layers are specified, then the second layer of epoxy will be applied immediately following the removal of the excess aggregate from the prior epoxy layer application. The coating application will be as specified by the epoxy manufacturer. The aggregate will then be broadcast into the wet epoxy coating until there are no wet spots. Following the specified cure period, the excess aggregate will then be removed consistent with the procedure following the first application. The final thickness will be dictated by the layers of aggregate broadcast into the wet epoxy coating.

10.3.5 Polyester overlay systems

Polyester overlay systems are used on bridge decks. Their purpose is to provide protection as well as a wear course, and their popularity has been increasing gradually. These systems are used primarily in the western USA. The polyester overlay system produces an impervious membrane integrated with fine aggregate to create a barrier against the intrusion of chemicals into the concrete as well as a wear course to protect and extend the life of a concrete surface.

Conditions of use

The primary purpose for the application of a polyester overlay system is to protect the concrete and reinforcing steel against attack by aggressive chemicals and abrasion. The primary structural systems in which polyester overlays are commonly used are bridge decks.

The bridge deck coating system is a premixed system that may be placed in one or two layers. The single layer system will provide a layer 9mm (3/8 in.) in thickness. A second 9mm (3/8 in.) thick layer may be applied on bridge decks that will be subjected to heavy volumes of traffic for a total thickness of 19mm (3/4 in.).

The conditions of abuse will be more severe in geographic regions that permit the use of studded snow tires. Although the use of studded snow tires is diminishing in favor of all-weather tires, they are still in use. In these geographic regions, multiple layers will be a good investment.

Polyester overlay application

Prior to the application of a polyester overlay system, the surface must be dry and properly prepared to ensure the integrity and longevity of the applied system. This includes the removal of all deleterious materials that are on and/or in the concrete at and below the surface. This may be achieved by sand blasting, scarifying or shot blast. The final surface condition should be free of any deleterious materials and reveal the fine aggregate. Immediately prior to application of the polyester overlay system, the surface should be cleaned and free of all loose particulates. The finished surface should expose the sand particles and the coarse aggregate surfaces.

The polyester system comprises a primer, high molecular weight methacrylate (HMWM), and a polyester concrete consisting of a two-component polyester binder and a graded aggregate. The prime coat shall be applied, at a rate as recommended by the manufacturer, immediately before the placement of the polyester concrete.

The polyester binder is premixed. The polyester concrete, polyester binder plus aggregate, is then customarily mixed in a $0.25 m^3$ (nine cubic foot) or smaller concrete mixer. Immediately following the application of the prime coat, the polyester concrete mix will be placed using a vibrating screed that is set on preset rails. This process will provide a uniform thickness of the polyester concrete mix overlay. The polyester concrete thickness is normally applied in a 19 or 32mm (⅜ in. or 1 ¼ in.) thickness over the entire bridge deck surface.

10.4 Case studies

10.4.1 Polymer overlays

Polymer overlays were first introduced in the 1950s. The first systems were coal tar epoxy systems in which a layer of coal tar epoxy was laid down and sand broadcast into the wet epoxy. These initial attempts did not perform satisfactorily but were a harbinger of what was to come. Following the initial epoxy overlays, polyurethane and polyester overlays followed in the 1960s.

10.4.2 Polyurethane overlays

The first polyurethane overlay system was applied in the early 1970s to parking structure decks following the failure of attempts made utilizing neoprene systems that could not meet the durability expectations. These systems were designed and intended to provide a protective barrier that would eliminate water penetration from one level to another, reduce maintenance costs and extend the life of the parking structure. The original installations were in the Midwest. These installations enjoyed an acceptable level of success. Acceptance of the urethane overlays gradually expanded their sphere of influence around North America and subsequently around the world. In today's market they are the most commonly used overlay system on parking garage decks throughout the world.

The polyurethane system as we know it today has evolved over the years and demonstrated its viability as a polymer overlay that provides an effective barrier to moisture penetration. The resulting system extends the life of the concrete parking structure decks by acting as a moisture barrier and a wear coat. The specified thicknesses vary around the world from 1.5–3mm (60–120 mil) thickness for the total applied system.

10.4.3 Epoxy overlays

The first polymer overlays were coal tar epoxy systems that were applied to a bridge deck in the mid 1950s. This system was a layer of coal tar epoxy applied to a concrete substrate and had sand broadcast into the wet epoxy coating. The system did not stand up well to the abrasive forces that it was subjected to nor did it do a very good job of preventing moisture from penetrating the concrete substrate.

Subsequent attempts with epoxy systems that were modified gradually evolved, developing epoxies that were resistant to UV exposure; flexible enough to accommodate the differential movement of the concrete substrate and the epoxy overlay system; as well as being capable of resisting the abrasive forces that the epoxy overlay was subjected to. The epoxy systems in use today are single and multiple layer systems that are capable of providing a serviceable system for in excess of 20 years. These systems provide protection against the intrusion of salt-laden solutions that contribute to the accelerated corrosion of the reinforcing steel. The corrosion leads to the premature deterioration of the structural concrete bridge or parking garage deck. These systems also provide abrasion-resistant overlays that are capable of resisting the abrasive forces of tire chains commonly used on trucks and autos in the geographical regions of the northern latitudes.

The level of success as measured by the service life of the system used on a bridge deck is usually related to the abrasion-resistant capability of the aggregates that are used as well as the gradation of the aggregates. The basic system is achieved by seeding a layer of aggregates into a wet coating of epoxy applied to the structural concrete substrate or a previous epoxy overlay application. The aggregate type and size is usually determined by the traffic that the overlay will be exposed to. Heavily traveled bridges, particularly in the northern latitudes, will use multiple layers more frequently than bridges in the southern latitudes.

10.4.4 Polyester overlays

Polyester overlays, like urethanes and epoxy systems, evolved gradually to the system that is in use today. Polyesters started out as a 'broom and seed' process similar to urethane and epoxy systems in the early 1960s. The first use of the current system was in 1983 in California. The polyester overlay system has been developed in close cooperation with the California Department of Transportation. The early success experienced with the use of polyester provided the impetus for California to continue with their use. The wide variety of climatic conditions in California made the use of polyesters very desirable due to their flexibility and curing properties.

The current polyester system is a premixed system as opposed to laying down a coat of polyester and broadcasting aggregate into the wet coat. The system can be laid down in a thickness of 9mm or 19mm (3/8 in. or 3/4 in.) depending upon the geographical location and the amount of traffic expected to be carried by a bridge. Bridges in the mountainous regions of

California will commonly use an overlay thickness of 19mm (3/4 in.) to have the capability of withstanding chained tires required on snow and ice covered highways.

The use of polyester overlays is increasing in use around the country from California to New York state. This system is used exclusively on heavily travelled bridge decks. The overlay provides protection against the intrusion of salt-laden solutions that accelerate the corrosion of the reinforcing steel in the bridge deck. The original research projects carried out in both California and Virginia have shown that the polyester overlays will perform successfully for more than 20 years.

10.5 Recommendations

The uses of polymer overlays have been on the increase. Their success in any application requires a substrate that is in good repair. A polymer overlay will not cure a deteriorating deck that has not been adequately repaired. Deterioration will continue below the applied overlay. The on-going deterioration will shorten the service life of a polymer overlay thereby increasing the life cycle cost.

On parking garage decks, urethanes have been the system of choice. One of the major limitations in the use of the urethane systems has been the abuse the top deck has suffered due to snow removal blades tearing the coating, thereby permitting the ingress of salt-laden water to penetrate the exposed concrete substrate. Although there are ways to accommodate the use of snow removal blades to avoid this problem, they are rarely incorporated or required by the owner. Polyurethane systems may be applied in multiple layers to increase their abrasion resistance but not their ability to withstand snow removal blades. The quality control procedures required for the application of polyurethane systems are primarily related to guaranteeing that the specified rate of application is adhered to.

Epoxy overlays are used on both structural parking decks and bridges. These systems have the flexibility of single or multiple layers depending upon the traffic. At the present time, there are several manufacturers of epoxy systems recommending their products for use in epoxy overlays. There are differences in the performance capabilities of these systems, and it is the responsibility of the design engineer to specify the performance requirements of the system that is deemed acceptable for their application. Quality control testing is imperative with epoxy systems because they are all two-component systems that require thorough mixing.

Polyester overlays are currently being used only on bridge decks. They are more forgiving in their application than epoxy systems since thorough mixing is not imperative to achieve their full performance capability. Quality control is still an important aspect of the placement of a polyester overlay since the mixed polymer system is then blended with the aggregates before the mixed system is placed on the concrete deck. Furthermore, the polyester system must be applied within a narrow timeframe following the application of the HMWM prime coat.

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11 Patching of deteriorated concrete structures

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Abstract: Patching of concrete is necessary to restore a concrete element to its original condition. Concrete is subject to erosion, impact or corrosion that results in localized deterioration of concrete. Of the three, corrosion is by far the most common with failure due to impact in second place.

Key words: patching, corrosion, surface preparation, polymer, preplaced aggregate, shotcrete.

11.1 Introduction

The patching of concrete is the end product necessary to restore a concrete element to its original condition following some form of action that disrupts its ability to transfer loads consistent with the original design. Concrete, in an external or aggressive environment, is more subject to erosion, impact or corrosion that results in localized deterioration of concrete. Concrete in interior environments is not normally subject to corrosion and much less likely to suffer damage due to an impact load.

The conditions most commonly requiring patching are erosion due to abrasion, corrosion of the reinforcing steel or impact loads. Of the three, corrosion is by far the most common with failure due to impact in second place. Erosion is much less of a problem and requires a more specialized level of repair. This chapter will deal with the repair of concrete due to corrosion. Repair techniques required for repair of concrete in dams is a specialized area outside the scope of this book.

Concrete will deteriorate in localized areas that are likely the direct result of corrosion of reinforcing steel. The reinforcing steel occurs in the upper half of concrete in those zones that are in tension. The quantity of reinforcing steel will vary depending upon its relationship to columns and/or column capitals in slabs and beams. Reinforcing steel, when placed initially in concrete, is in a passive environment due to the alkalinity of the concrete environment. An alkaline environment is not conducive to the electrical conductivity necessary for corrosion to occur. Corrosion is commonly localized thereby requiring removal and replacement of concrete in small areas that result in the ‘patching’ of concrete. This process results in a localized repair intended to re-establish the original continuity of the concrete component or element.

The most common failure due to impact is on girders of bridges. They are commonly impacted by semitrailer trucks and trucks carrying other vehicles, such as construction equipment. Vehicles will sometimes impact concrete walls or other elements inside or outside of buildings. Any resulting damage will require repair or replacement. When repair is deemed the engineering solution, it will most likely require patching.

The concrete structural systems that incur erosion are bridge decks and spillways and stilling basins in dams. On bridge decks, erosion is the result of vehicle tires continuously passing over the concrete surface in the same lines over and over again for extended periods of time. Erosion of spillways on dams is the byproduct of particulates, sand and gravel, carried in the water flowing down the spillway and/or implosion resulting from the rapid flow of large quantities of water down the spillway. The floor of the stilling basin structure at the foot of a spillway on a dam may experience erosion from sand and aggregate or construction materials left in the spillway following the construction operation.

11.2 Condition survey

A condition survey will provide the owner with a detailed assessment of a structure's current condition. Within that context, the intent is to provide the owner with the necessary tools to make a decision whether to defer maintenance, execute a maintenance program or remove and replace the structure or individual structural elements that may need extensive repair.

There are a broad range of conditions that will cause a concrete structure and/or its elements to deteriorate. The deterioration will be visually apparent or unseen within the structural elements. Visual deterioration is most commonly caused by erosion of the concrete surface. The erosion process may be due to physical abrasion or chemical erosion caused by the deterioration of the cement paste within the concrete matrix due to exposure to an aggressive chemical element.

Internal deterioration is most commonly caused by corrosion of the reinforcing steel. Deterioration due to corrosion will provide visual indications of its occurrence such as cracking and spalling at the concrete surface. This phenomenon will not define the limits of the active corrosion. It will only indicate the limits of excessive corrosion that have produced sufficient expansion of the reinforcing steel to exceed the tensile strength of the concrete. Active corrosion occurs in the area around the visual deterioration and must be identified by testing. Testing will define the limits of the active corrosion and consequently the limits of repair that will be required.

Other forms of internal deterioration are alkali–silica reaction and ettringite reaction. Each of these phenomena is revealed visually by map cracking at the surface. This is the result of the formation of a gel around the coarse aggregate due to a chemical reaction between the aggregate and the cement phase of the concrete. The gel formation around the aggregate expands, trying to displace concrete, and thereby creates a tensile stress that ultimately exceeds the tensile strength of the concrete.

The types and means of testing, both destructive and non-destructive, as well as the causes of deterioration are addressed elsewhere in this book.

11.3 Surface preparation

The types of deterioration that have been defined in this chapter include deterioration that extends throughout a concrete element, such as alkali–silica reaction and ettringite reaction. Both erosion and corrosion may be confined to localized areas or extend across the entire surface area of a concrete element. Surface preparation will address repairs that require a localized solution. Spalling due to impact loads is another form of damage that requires a localized repair.

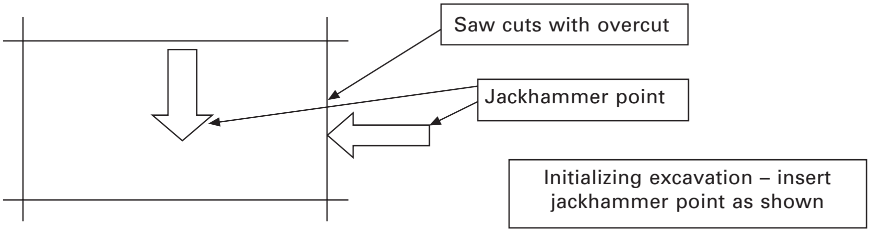
The means by which a contractor will address the excavation process will be determined by the required condition and presentation of the finished repair. Normal repairs that are strictly functional in nature are defined by a saw cut, then excavated and repaired. Repairs that will have a visual affect on the finished product will be done with greater concern to detail than a repair that is in an industrial environment.

Localized repairs are normally required because of erosion or corrosion. Corrosion is the most common cause of localized repairs. The excavation process must proceed with care. When corrosion is the issue to be addressed, then the excavation process may start in the center of the deteriorating area. This will be visually defined by either a spall and/or cracking over the corroding reinforcing steel. Eroded areas are also defined visually. The concave area can be easily identified. These areas may be delineated by saw cuts and excavation would then follow.

11.3.1 Typical patching excavation

A typical area to be patched will be defined by saw cuts that delineate the area to be excavated (Fig. 11.1). Once the area has been defined by the saw cuts, excavation of the concrete within the area can be executed. The excavation process may start in the center of the deteriorated area and work outward toward the saw cut. If the excavation process starts at the saw cut, there will be chipping of the concrete along the saw cut along the outer edge.

Following the excavation of the concrete, the vertical surfaces of the patch area created by the saw cut should be sand blasted. This is necessary because the saw cutting operation will polish the vertical surfaces consequently making it difficult for the patching material to bond. To ensure a positive



11.1 Typical area to be patched following standard saw cutting procedure.

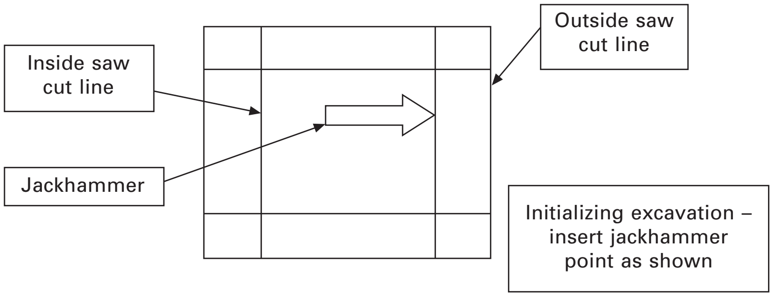
bond between the parent concrete and the patching material, the vertical surface must be roughened. The interior surface of the area to be patched must be thoroughly cleaned to ensure a positive bond throughout the area. All loose particles must be completely removed. This may be accomplished by compressed air. All moisture from the compressed air must be filtered out, because moisture may create an environment that will prohibit bond of the patching material to the prepared surface.

11.3.2 Architectural considerations

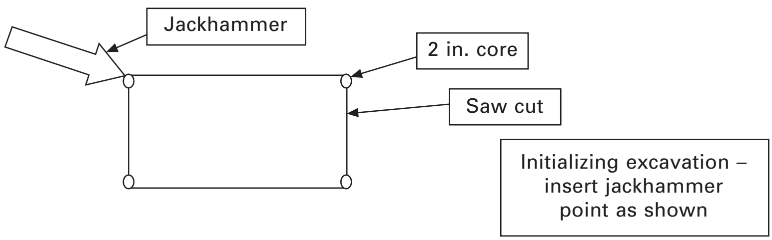
A repair that will impact the visual appearance of the concrete surface must not leave any mechanical signs such as overcutting the boundary lines when delineating the area with a saw cut. There are two ways in which this condition can be avoided.

Double saw cut

When delineating the area to be excavated, the saw cuts will intersect at the corners. To ensure that the excavation will be full depth at the corners, the saw cut must extend beyond the perpendicular line that it intersects. The overcut is unsightly and provides an access point for the start of deterioration. To eliminate the overcut, a second saw cut must be completed outside the first saw cut a distance equal to the leg of the triangle formed when using the line of the overcut as the hypotenuse. When the area inside the saw cuts is excavated, the corner within the excavated area will create an arch from the top surface to the bottom of the excavation. Further, by using an internal saw cut the laborer will not produce unsightly chips along the saw cut line common when using only one saw cut. The surface area of the excavation will not have been violated beyond the limits of the edge of the excavation either by saw cut overrun or chipping along the outside saw cut. This procedure is illustrated in Fig. 11.2.



11.2 Delineating the area to be patched using a double saw cut to eliminate overcut and edge chipping for architectural considerations.



11.3 Delineating the area to be patched by drilling four holes (cores) at the corners and saw cutting into the holes for architectural considerations.

Cored corners

Coring the inside corners of an area to be patched (Fig. 11.3) will eliminate the need for overcutting by the saw blade. This procedure will allow a uniform depth of the excavation throughout the patch area. The initial depth must be no greater than the depth of the top of the reinforcing steel.

Corrosion of the reinforcing steel is generally localized and may be repaired by excavating concrete that has been directly affected by its impact. Once the extent of corrosion has been identified, the contractor has two choices to execute the excavation process. The most common procedure is to delineate the affected area with lines. A saw cut along the lines is then executed and the concrete within the saw cut and around the reinforcing steel is then excavated.

The second excavation procedure will execute the final saw cuts following the initial excavation. The excavation would be started over the corroded reinforcing steel and continued until all the steel that exhibits active corrosion has been exposed. The saw cuts would then be executed along the specified boundaries beyond termination of the active corrosion. This procedure will minimize any requirement to delineate the area to be patched by saw cutting a second time. Subsequent saw cutting will be required if cracking occurs over the sound reinforcing steel that is impacted by the jackhammer point during the excavation process.

Excavation of areas exhibiting corrosion will require the concrete around the reinforcing steel to be removed. A clearance of 19mm (3/4 in.) below the reinforcing steel will be sufficient to allow the patching concrete to flow completely around the reinforcing steel. Full encapsulation will help to reduce shrinkage and re-establish stress flow through the repair area.

11.4 Repair materials

There are a broad range of materials on the market that have been designed to fill the void created by the excavated cavity. They include normal Portland cement concrete, polymer-modified Portland cement concrete and polymer concrete.

11.4.1 Conventional Portland cement mortar/concrete

Conventional mortar/concrete may include a broad range of different cements. These systems may also include fly ash or silica fume in addition to proprietary admixtures. The cements may have the capability of normal to early strength gain. The benefit of using a repair material that replicates the existing concrete is the fact that there will be very similar mechanical properties. The repaired area then has a better chance of reacting to its environment as well as the stress flow consistent with the concrete around it. There are also some very distinct economic benefits.

The limitation to using conventional Portland cement mortar/concrete as a patching material is that, due to the small quantities, the material must be proportioned and mixed on site. To ensure consistency in this process, experienced laborers are essential for success. With conventional mortar/concrete, there is a higher probability of developing an anodic ring around the patch where corrosion has been the primary cause for the repair. This condition is more prevalent in geographic climates with a high average humidity.

11.4.2 Latex-modified Portland cement mortar/concrete

There are several latex polymers that are available and added to conventional mortar/concrete. The most common latex additives for mortar/concrete are styrene butadiene, acrylic, vinyl acetate ethylene, styrene acrylic and epoxy. The addition of a latex, while enhancing the concrete, changes some of the mechanical properties. These systems may be proportioned and mixed on site or are premixed proprietary materials that may or may not have aggregate added and are mixed on site.

The addition of the latex provides some distinct benefits to the repair material. It has a higher degree of impermeability than conventional Portland cement concrete and generally has a different coefficient of thermal expansion. The addition of a latex to a proprietary conventional mortar/concrete mix is one of the ingredients that allows the end user to repair very thin layers. The latex-modified patching materials may be used in repairs varying in thickness from 6–300mm (1/4–12 in.). The w/cm ratio for these materials must be lower than conventional mortar/concrete. The w/cm ratio for a latex modifier is normally between 0.30 and 0.40 including the water in the latex. The w/cm ratio for an emulsified epoxy modifier is 0.25–0.35. When proprietary mixes are being utilized, the contractor should follow the recommendations of the material supplier for mixing and placing.

11.4.3 Magnesium ammonium-phosphate cement mortar/concrete

This material is available as a proprietary product for repair. It is available in two formulations, summer and winter. It is a unique material that can be placed with or without the addition of aggregate and from very thin (6mm, 1/4 in.) to quite thick (300mm, 12 in.). It is a rapid-setting material that produces a high exothermic reaction and can be put in service within hours of application. The addition of coarse aggregate to the mix helps to reduce the maximum temperature generated by the exothermic reaction. These materials have a very rapid strength gain and are desirable for repairs requiring a quick turnaround.

11.4.4 Polymer mortar/concrete

Polymers typically used in polymer mortar/concrete are epoxy, polyester, furan and vinyl-ester. Furan and vinyl-ester are specialty polymers commonly used in areas in which the concrete deterioration is due to a highly aggressive chemical environment. Additional information on these polymers may be obtained from their manufacturers. The two most commonly used polymers for polymer concrete are epoxy and polyester. This chapter will deal with epoxy and polyester only and their uses as patching materials.

Epoxy mortar/concrete

The epoxy is a two-component organic polymer. Epoxy mortars and concrete are a blend of epoxy and sand or epoxy and sand plus coarse aggregate. The epoxy system with or without aggregate will provide an impermeable environment and barrier that will protect the reinforcing steel against corrosion.

The epoxy mortar/concrete has little or no shrinkage. During the curing process the exothermic reaction will create a temperature rise that will cause the mass to expand. The presence of fine and coarse aggregate will minimize the temperature rise and subsequent expansion. As the mass cools, there will be shrinkage that will return the mass to its original size. In short, the mass will expand during the exothermic reaction and contract during the cooling period.

Epoxy mortar/concrete can be designed to repair cavities from 3–38mm (1/8–1 ½ in.). The aggregate size will be dictated by the depth of the void being repaired. The epoxy/aggregate ratio, for fine or coarse aggregate, can be optimized by utilizing a gradation that maximizes the amount of aggregate.

Sections greater than 38mm (1½ in.) deep can be repaired with an epoxy concrete. However, to achieve an epoxy concrete that will have a coefficient of thermal expansion comparable to Portland cement concrete, a placement procedure known as preplaced aggregate must be utilized.

Polyester mortar/concrete

The polyester is a two-component thermosetting polymer. Polyester mortar is a blend of polyester and fine aggregates. These aggregates may be a single size or graded. To create a polyester concrete, a well-graded coarse aggregate is incorporated in the mix. The maximum aggregate size will be dictated by the thickness of the patch and/or the clearance below the reinforcing steel.

The polyester mortar/concrete has very little shrinkage. The primary shrinkage occurs as the mass cools following the expansion caused by the heat rise due to the exothermic reaction. The presence of fine and coarse aggregate will help to minimize shrinkage. The exothermic reaction created by the thermosetting polyester is relatively low. Polyesters will gain strength rapidly when placed at ambient temperatures between 4 and 32 °C (40 and 90 °F).

Polyesters can be placed in lifts up to 300mm (12 in.) thick. From the view point of material capability, this would expand the usefulness of polyester concrete. The limitation that would restrict repairs greater than 38mm (1½ in.) thickness would be the economics of the repair. Repairs greater than 38mm (1½ in.) thick can normally be readily achieved with conventional Portland cement concrete or latex-modified Portland cement concrete.

11.5 Material placement

The placement of repair materials can be achieved in several different ways. The primary methods used are hand placement, pumping, shotcrete or preplaced aggregate. The method chosen may be dictated by a variety of conditions, both physical and economic.

11.5.1 Hand placement

Most patching materials are placed using the hand placement technique. One of the primary reasons is that most areas requiring repair and subsequent patching are on the horizontal surfaces of concrete structures that are subjected to physical and chemical abuse. These repairs simply require the mixing of the material and placement by filling the cavity by hand. Once all the material is in place, the exposed surface can then be finished and the material cured in a manner consistent with the manufacturer's recommendations. All of the materials listed under the material section can be hand placed.

All of the materials in the material section may be modified to allow for hand placement on vertical or overhead surfaces. The adhesive properties are unknown, since there are no test procedures that provide for placement to vertical or overhead conditions. There appears to be sufficient adhesion so that the repair material, once in place, stays in place, upon curing. Materials designed for this use are all proprietary, since the required modifiers are not normally known in the engineering and construction communities.

11.5.2 Pumping

Pumping is an effective means for transferring either mortar or concrete. It is a convenient method for material placement in large repairs, particularly in the repair of vertical elements. Pumping is also an efficient means by which to access repair areas that are otherwise difficult to reach.

Concrete elements that lend themselves to repair by pumping are column capitals, the addition of shear walls, replacement of bridge decks or other structural elements. These elements plus any repairs that must be formed are candidates for pumping. Pumps from hand pumps for small easily accessible repairs to large truck-mounted pumps can be used for large and/or distant repairs. Pumps can be used to transport any of the repair materials. They are, however, more commonly used for cementitious materials than for polymers.

11.5.3 Shotcrete

Shotcrete is a system by which mortar or concrete can be placed pneumatically. The shotcrete process has been around since 1910, starting out as a proprietary process known as 'gunite'. Although the term shotcrete was first used in the 1930s and officially adopted by the American Concrete Institute in the 1950s, the term 'gunite' is still heard occasionally.

Shotcrete is applied either as a wet mix or dry mix with water added at the nozzle. The wet mix process has the capability of placing either mortar or concrete. For concrete, the material will normally be delivered to the site by ready-mix truck or mixed at the site in a concrete mobile mixer. The coarse aggregate is normally limited to 9mm (3/8 in.) and at a reduced quantity compared to conventional Portland cement concrete.

The dry mix process uses a mortar mix of cement and fine aggregate with the water added at the nozzle as the cement/sand mix is pneumatically shot from the nozzle. The cement/sand mixes used for concrete repair are commonly proprietary mixes that are bagged and delivered to the job site. The mix is deposited into a hopper from which it is transferred by pump to the nozzle. The material is projected pneumatically at the nozzle and water is introduced into the mix from a water ring. This process is an effective way to repair small or medium size cavities on vertical surfaces or overhead.

An alternative process in which the mix is blended at the site provides the contractor with the capability of prewetting the fine aggregate before it is blended with the cement. Additional water is added at the nozzle.

The wet mix process lends itself to the construction of large concrete elements. With many buildings requiring upgrades to meet the structural requirements in seismic areas, shear walls are a principal element. The shotcrete process is frequently a more economical solution than forming and placing concrete.

The freshly placed shotcrete is ready to be finished immediately. The finishing process can provide a smooth finish, and the finished surface will resemble a formed finish in that it will be smooth but will likely have a wavy surface. Finishing vertical or overhead surfaces is more difficult than finishing a floor surface.

11.5.4 Preplaced aggregate

Preplaced aggregate defines in its name the basic principle of the process. An aggregate that would be similar to the coarse aggregate minus the fines is placed in a form. The size and gradation would be dictated by the size of the cavity and the filler material used to complete the recipe for the concrete. The point to point contact of the coarse aggregate creates a concrete with slightly different mechanical properties when compared with conventional concrete.

The aggregate phase placed in the form is the coarse aggregate. The fine aggregate, if required, would be included in the grout phase that would be pumped into the form as the final addition for the production of the concrete. The fine aggregate would normally conform to ASTM C33 for conventional concrete. A modified version with a smaller large particle would be used where the maximum size of the coarse aggregate would be smaller than 9mm (3/8 in.).

Preplaced aggregate concrete replicating conventional concrete exhibits much less shrinkage than concrete that is premixed. This is one of the benefits derived from point to point contact of the coarse aggregate particles. The resinous concrete utilizing epoxy or polyester will have a coefficient of thermal expansion comparable to conventional concrete. This also is the benefit of having point to point contact of the coarse aggregate particles.

The benefits attained with point to point contact of the coarse aggregate particles make the preplaced aggregate process more desirable in structural repairs. The absence of perimeter shrinkage allows the stress flow through the repair consistent with the stress flow around the repair without any deformation required to achieve contact between the parent concrete and the patch. The point to point contact in a resinous concrete allows the polymer concrete to react to thermal changes in a manner consistent with the parent concrete.

To maximize the structural capability of a preplaced aggregate repair, the latent stresses in the structural member should be removed. If a repair is to be in the compression zone of a structural element, all the compressive forces should be removed. The same is true when the repair is to be in the tensile zone of a structural element. By doing this, the repair will react to subsequent applied loads in a manner consistent with the stress flow in the original concrete of the structural element.

Conventional concrete

The conventional concrete created through the preplaced aggregate process includes the fine aggregate in the cementitious grout. The grout is then pumped into the forms containing the coarse preplaced aggregate and fills the voids. In addition to cement, the grout may contain fly ash or silica fume. It will also consist of air-entraining admixtures and fluidifiers to modify the plastic properties of the grout and control the bleed water that otherwise might collect at the bottom of the individual coarse aggregate particles and on top of the formed mass.

Latex-modified Portland cement concrete

The procedure for the placement of a latex-modified Portland cement concrete is the same as for a conventional concrete. A latex may be substituted for the air-entraining agent and the fluidifiers that are beneficial to conventional concrete as well as part of the required water.

Polymer concrete

The polymer, an epoxy or polyester, is a resinous material which will be pumped into the formed volume without the addition of a fine aggregate. The primary purpose for utilizing a resinous concrete is when a rapid strength gain requirement is imposed on the project that can only be met with an epoxy or polyester. The exothermic reaction developed by the resin has not proven to be problematic. The heat sink capability of the coarse aggregate is sufficient to minimize the temperature rise during the cure process. The temperature rise is sufficient to ensure the strength development of the resinous material and subsequently the resinous concrete.

The coarse aggregate utilized in these repairs must be dry. Any moisture has the potential of changing the properties of the resinous concrete. To define the level of dryness or the allowable moisture, the resin manufacturer should be consulted.

There are some unique requirements for the placement of preplaced aggregate with resinous materials. For epoxy and polyester, the forms should be precoated with an epoxy for an epoxy concrete and with a polyester for a polyester concrete. The objective is to prevent the resin from bonding to the form during the grouting operation. Neither resin will bond to itself once cured. In addition, the concrete surface to which the polyester concrete is to be bonded must be primed with a high molecular weight methacrylate (HMWM).

11.6 Case studies

Patching of concrete has been around since the invention of Portland cement. The original repairs were executed with a process known as drypack. Drypack consisted of cement, sand and a very small amount of water. The water content was just sufficient to moisten the cement and sand. There was no excess moisture. The object was to create a mix that had very little shrinkage. If there was just enough water to hydrate the cement, then there would be no excess water to evaporate and therefore there would be no shrinkage. The process was effective but very labor-intensive to install. Although this procedure is still carried on today, it is only in very isolated circumstances.

With the development of epoxies and their use in concrete, they were used to perform thin repairs because they did not shrink. Manufacturers started by working with contractors to mix a two-component epoxy with sand. The volume of sand would be dictated by the gradation and the roundness of the sand particles. Silica sands were readily available and could be purchased over the counter at most construction supply stores. Since they were generally only one size and quite angular it was difficult to add more than two parts sand by volume to one part epoxy. Material suppliers started to premix sand and epoxy and marketed them as patching materials. Many contractors found local sands that were well-graded and rounded that could be mixed with the epoxy in much larger volumes, and they produced a material that was easier to place and finish. Some manufacturers started selling well-graded sands in bags with their epoxy that would be used as a patching material.

As latexes were introduced to the market, they went through an evolutionary process. The latex was sold and added to a grout on the job site that was intended to provide a patching material that would repair thin patches. They started out exhibiting a high degree of shrinkage, which was proven to be detrimental to the durability of the repair. Over the years, the material suppliers have improved the properties of the latex-modified grouts and they are widely marketed successfully. The latex-modified mortars and concretes are used widely and successfully in a broad range of repair applications.

The repair industry has grown as the uses and abuses of concrete have expanded. One of the benefits that concrete has over its competitive materials is its repairability. No other material is as easily repaired as concrete, thereby ensuring a long and durable life expectancy.

11.7 Recommendations

Patching of localized areas of concrete deterioration has proven to be an effective means to increase the life expectancy of a structural element. To maximize the benefits of repairs that incorporate patching that is the result of corrosion, the reinforcing steel that has been encountered must have the concrete around it completely removed. The removal process must have a clearance below the reinforcing steel of approximately 19mm (3/4 in.). This is necessary to ensure that the patching material will completely encompass the reinforcing steel.

One of the potential results of the patching process can be the creation of an anodic ring around the patch. This problem is created by the electrical potential difference between the parent concrete and the patching material. The electrical conductivity between the patching material and the parent concrete can be reduced by reducing the conductivity of the patching material. The inclusion of silica fume or a latex modifier will reduce the conductivity of the patching material. The inclusion of a corrosion-inhibiting admixture has also shown promise in reducing the impact of the anodic ring. The use of an epoxy or polyester mortar/concrete can eliminate electrical conductivity between the patch and the parent concrete. Another means by which the anodic ring has been eliminated has been with the introduction of a sacrificial anode attached to the reinforcing steel within the patched cavity.

The reinforcing steel in areas that are being patched due to corrosion should be carefully evaluated. When the loss of the reinforcing steel's cross-sectional area is greater than 15%, that area should be supplemented with additional reinforcing steel. The supplemental bars should have a length sufficient to fully develop the embedment length on each side of the affected length.

In projects in which all corrosion-induced areas of deterioration have been addressed and the repair executed, the long-term benefits may be enhanced by protecting the deck against the increased levels of chemicals, usually chlorides; these can be prevented from penetrating the concrete surface using a sealer, polymer overlay or a latex-modified concrete overlay. These additional protective barriers act as a deterrent to the penetration of additional chlorides into the concrete. The procedure can reduce the life cycle cost of the repaired elements.

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12
Durability of repaired concrete structures

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Abstract: The main structural/material requirements for the design of durable repairs to concrete structures are reviewed. A strategy for durability assessment of repaired concrete structures is also outlined, including examples of various instrumentation and data analysis techniques that are found particularly useful for field research. Three selected case studies are presented, each featuring some of the key methods used to assess the in-service performance and durability of repaired concrete structures, along with the main findings obtained in these field studies. The chapter concludes by providing recommendations to achieve durable repaired concrete structures.

Key words: durability, concrete repair, compatibility of properties, case studies, field performance monitoring.

12.1 Introduction

Concrete is the primary construction material of many structural systems, such as highway bridges, high-rise buildings, parking structures, and dams. Today, many concrete structures that have been exposed to aggressive environments suffer from durability problems and fail to fulfil their design life requirements. The problem is particularly serious in reinforced concrete structures where concrete cracking and corrosion of the reinforcing steel can impair their safety. Concrete cracking and chloride-induced corrosion of the steel reinforcement are the two most common causes of deterioration of concrete structures, especially those made with high-strength concrete. The repair cost of corrosion-damaged structures in North America and most European countries constitutes a large portion of their infrastructure expenditures. The limited knowledge on the field performance of damaged structures and the lack of systematic and reliable approaches for their inspection, maintenance and repair contribute to the increase of their life-cycle costs, and result in the loss of serviceability, functionality and safety.

A Federal Highway Administration (FHWA) report on corrosion protection of concrete bridges estimated that the total cost to eliminate the backlog of deficient concrete bridges in the USA ranged between \$78 billion and \$112 billion, depending on the time required to carry out the task (Virmani and Clemena, 1998). The prohibitive costs needed to upgrade highway bridges require the development of innovative decision support tools for bridge owners and engineers. Such tools will enable them to assess the conditions of their structures, predict future performance and allocate limited funds in order to better manage the maintenance of their structures and achieve adequate reliability and minimum life-cycle costs. The repair of concrete structures is a complex problem due to various factors. Their performance is difficult to predict, owing to the complexities associated with assessing material characteristics, environmental effects, damage initiation and accumulation, loss of structural strength, failure mechanisms and their impact on serviceability and safety (Lounis, 2007). Most of the damage mechanisms are not fully understood, such as the processes of corrosion initiation and more specifically damage accumulation. Other difficulties arise in detecting damage early in the life of the structure, and assessing the field performance of repaired concrete structures. Furthermore, there is a need for the implementation of a systematic approach for inspection and repair of concrete structures in order to ensure their safety and durability.

This chapter begins by setting the background information (Section 12.1) and addressing the main structural/material requirements for the design of durable repairs to concrete structures (Section 12.2). Section 12.3 presents a strategy for the assessment of durability of repaired concrete structures, and outlines various instrumentation and data analysis techniques that are found particularly useful in field research. Three selected case studies are presented in Section 12.4, each featuring some of the methods used to assess the in-service performance and durability of repaired concrete structures, along with the main findings obtained in these field studies. This chapter will conclude by providing recommendations to achieve durable repaired concrete structures (Section 12.5), as well as sources of further information (Section 12.6), and references (Section 12.7).

12.2 Designing concrete repairs for durability

Despite the widespread and expanding need for concrete repair, the limited comprehensive data and lack of suitable guidelines leave designers with considerable uncertainty as to how to proceed with the design and execution of durable concrete repairs. The severe climatic conditions in North America or northern Europe exacerbate the difficulty in providing such durable repairs.

12.2.1 Types of concrete repairs and repair failures

Although patching is widely used in the repair of concrete structures, there is little differentiation shown between the types of materials used in non-structural or structural repairs. Concrete patching materials are used in two different ways: (i) non-structural repairs, which improve surface appearance, reduce permeability, protect reinforcement or improve abrasion resistance

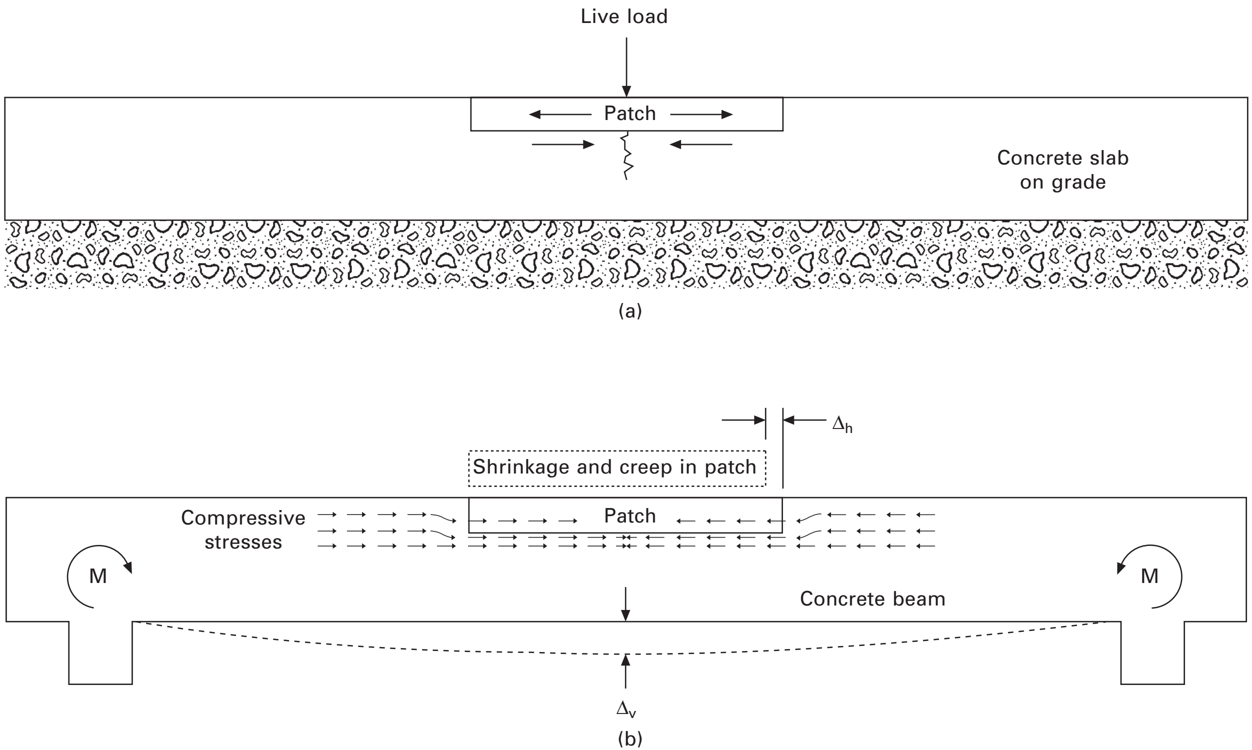
(Fig. 12.1a); and (ii) structural repairs, which restore the design load-bearing capacity of a damaged member or improve the load-bearing capacity of an under-designed member (Fig. 12.1b). Significant, or even excessive, stresses can develop in non-structural repairs just as easily as in structural repairs. A proper repair design requires the correct combination of properties and dimensions of the bonded materials in order to ensure that, for non-structural repairs, the interface bond strength is higher than the stresses at the interface and, for structural repairs, the repair should carry its design load and allow for any volume changes that may take place over time under the service conditions (Plum, 1990). Regardless of the type of repair, ensuring that the repair material is compatible with the substrate concrete is crucial, as the repaired member must behave monolithically and carry all stresses in the region of the repair without distress or deterioration (Emmons, 1994; Emmons *et al.*, 2000).

Inadequate durability of repaired concrete structures manifests itself in spalling, cracking, scaling and loss of strength. Three major modes of failure can be observed:

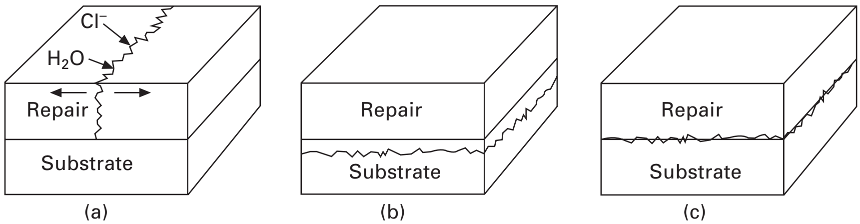
1. **Tensile cracking through the thickness of the patch** – This type of cracking can promote moisture and salt ingress (Fig. 12.2a) and is likely to occur when the tensile strength of the patch is lower than the strengths of the bond at the interface and the substrate concrete.
2. **Shearing of the substrate concrete below the interface** – When this occurs, failure is manifested as delamination of the patch with a layer of the base concrete bonded to its underside (Fig. 12.2b). This is the predominant failure mode when the shear strength of the substrate concrete is lower than the bond strength at the interface and the tensile strength of the patch.
3. **Failure of the bond between the repair material and the base concrete** – This takes place when the bond strength at the interface is lower than the strength of both the base concrete and the repair material (Fig. 12.2c).

12.2.2 Structural and exposure considerations

To achieve a durable repair, it is essential that the properties of the repair material and the substrate be properly matched. This helps ensure that the repair material can withstand the stresses that result from volume changes and load, for a given environment over a period of time, without experiencing distress and deterioration (Emmons and Vaysburd, 1994). The following parameters should be considered for the design of concrete repairs:



12.1 Typical non-structural and structural repairs: (a) non-structural; (b) structural. (Δ_h = change in horizontal deformation; Δ_v = change in vertical deformation)



12.2 Types of failure in patching systems: (a) transverse crack in repair material; (b) longitudinal crack in substrate concrete; (c) longitudinal crack at bond interface.

Size and geometry of repair patches

The performance of small patches introduced to restore durability of the member depends, for the most part, on the deformation capacity of the repair material, while the performance of large structural patches depends on both the deformation and load capacities of the repair material over the service life of the structure (Emmons and Vaysburd, 1994). For both large and small patches, high stresses can concentrate at edges and at changes in section, and may result in cracking at the bond interface and in the patch itself. Thus, a repair material with adequate tensile strength should be selected.

Presence of reinforcement in the repair

The effect of reinforcement on patch repairs is to reduce both the shear stress along the interface between the patch and the substrate, and the tensile stress in the substrate concrete. It can also provide a strong mechanical anchorage for the repair. However, reinforcing bars can introduce other problems: restraints imposed on movement and reinforcement corrosion can create large tensile stresses around the bars. For these reasons, it is important to select a repair material that bonds well to steel and has adequate tensile strength and low permeability.

Effect of section stiffness

Additional shrinkage stress can be generated in the repair material of a stiff flexural member because of the restriction of movement imposed by the relatively small amount of curvature. Moreover, differences in section stiffness along repaired beams of indeterminate structures can cause moment redistribution (generated by the shrinkage force in the repair material) resulting in greater shrinkage than in determinate structures (Yuan and Marosszeky, 1994). Therefore, where shrinkage is likely to be restrained, a repair material with low shrinkage potential is required.

In-service exposure conditions greatly affect the behaviour and performance of concrete structures, leading to different types of damage. Some of the conditions that should be considered during the design of durable concrete repairs include:

Humidity and temperature variations

Temperature changes and cycles of wetting and drying cause expansion and contraction of the concrete. These conditions may generate tensile stresses as high as the tensile capacity of the repair material and thus cause cracking and debonding of the repair material. Warm temperatures usually accelerate degradation mechanisms (including rate of corrosion); and the moisture level in concrete has a strong influence on reinforcement corrosion, as it affects carbonation, chloride penetration, electrical resistivity and oxygen level (Vennesland *et al.*, 2007).

Freeze–thaw cycles

When saturated concrete is exposed to low temperatures, the water held in the capillary pores freezes and expansion occurs. Repeated freeze–thaw cycles have a cumulative effect, causing rapid degradation of the repair material (Kropp and Hilsdorf, 1992).

Impact, sustained or cyclic loads

Impact loads can cause the concrete to spall because of the different wave-transmission rates of the several constituents of the repaired member. Sustained loads can induce additional strains in the repair material because of the differential creep between this material and the substrate concrete. Furthermore, cyclic loads can exceed the fatigue capacity of the repair material and cause its failure.

12.2.3 Material performance requirements

Some of the most important material properties to consider in the selection of a durable and compatible repair material are discussed below (Emberson and Mays, 1990). The required relationships between the properties of the repair material (R) and those of the substrate concrete (C) are identified in parentheses for each property. Achieving these requirements is a step in the right direction; however, it does not automatically guarantee durable repairs, as meticulous performance of all operations from batching to curing is also required (Neville, 2001).

Shrinkage strain (R<C)

In cement-based materials, most of the shrinkage occurs when the cement paste dries out after setting and hardening. When shrinkage is restrained (Fig. 12.1b), permanent tensile stresses develop in the repair material and may cause cracking in the material itself, or delamination at the interface (Yuan and Marosszeky, 1991). Since most repair materials are applied to an older substrate concrete that has negligible shrinkage, the repair material – which will begin to shrink soon after casting – must have very low shrinkage potential (Brill *et al.*, 1980).

Creep coefficient (R<C or R>C)

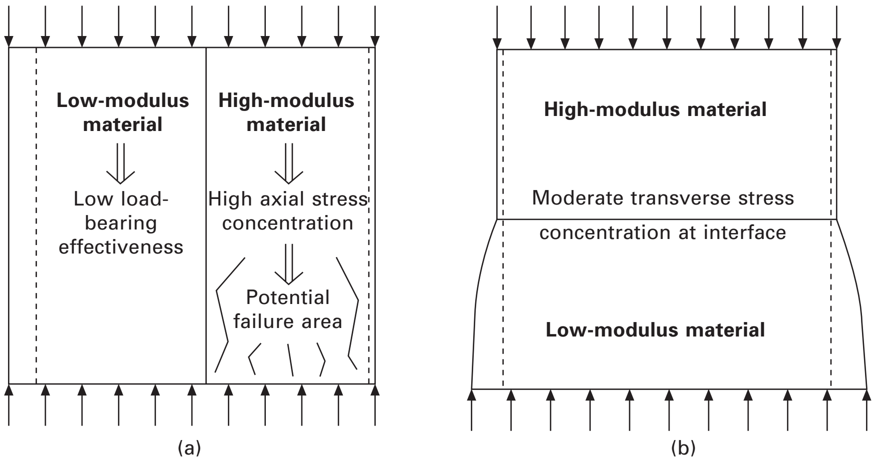
Creep is the continuous deformation of a member subjected to a sustained load. It can result in reduced load-bearing effectiveness in the repair material and in load transfer from the repair material to the substrate concrete, or to a non-structural element. In the case of structural repairs loaded in compression (Fig. 12.1b), the repair material must possess very low creep. On the other hand, in the case of repair patches loaded in direct or flexural tension, creep can be beneficial, as it can reduce the adverse effect of restrained shrinkage in the repair material (Saucier *et al.*, 1992).

Thermal expansion coefficient (R = C)

The coefficient of thermal expansion is a measure of the change of length in a material when it is subjected to a change in temperature. When two materials with different coefficients of thermal expansion are joined together and subjected to significant temperature changes, stresses are generated in the composite material. These stresses may cause failure at the interface or in the weaker material.

Modulus of elasticity (R = C)

When the external load (compressive or tensile) is applied parallel to the bond line (Fig. 12.3a), materials with different elastic moduli will transfer stresses from the 'softer' low-modulus material to the 'stiffer' high-modulus material, leading to stress concentration and failure of the high-modulus material (Hewlett and Hurley, 1985; Mailvaganam, 1992). When the external load is applied perpendicular to the bond line (Fig. 12.3b), the difference in stiffness between both materials is less problematic if the external load is compressive. However, if the perpendicularly-applied external load is tensile, different elastic moduli are likely to cause adhesion failure. The higher-modulus material imposes a severe constraint on the transverse contraction of the lower-modulus material.



12.3 Effects of mismatching concrete elastic moduli: (a) external load parallel to the interface; (b) external load perpendicular to the interface.

High concentrated stresses can then form in the lower-modulus material near the interface and initiate failure (Good, 1978).

Tensile strength (R>C)

A tensile force can be generated in a repair material by a combination of (i) external loading (impact, sustained, and cyclic), (ii) volume changes (shrinkage, creep, and temperature and humidity variations), and (iii) large differences in the properties of the repair material and the substrate concrete. When any of these forces produce a tensile stress in excess of the repair material tensile capacity, failure of the material can be expected in the form of tensile cracks, spalling, or debonding.

Fatigue performance (R>C)

Because cyclic loading causes progressive development and propagation of cracks, the fatigue strength of a material is less than its static strength. The number of loading cycles a repair material can withstand decreases rapidly as the level of stress increases. Unless the repair material is expected to experience only a negligible level of stress, it must have properties that can provide sufficient fatigue performance.

Adhesion (R>C)

Provided that an adequate match of the bonded materials exists, any improvement of the bond will increase the performance of the composite system. Repairs with bond lines in direct tension have a greater dependency on bonding than do repairs with bond lines in shear, which benefit from the aggregate interlock mechanism. The bond strength at the interface can be influenced by (i) the properties of the substrate concrete and its surface (roughness, cleanness, and curing state), (ii) the properties of the repair material, including absorption and its ability to adhere to the substrate, and (iii) environmental conditions (Saucier and Pigeon, 1991).

Porosity and electrical resistivity (R = C)

The porosity and resistivity of the patching material may also affect the durability of the patched area. When repair materials that are dense, impermeable, or highly resistive are used, there is a tendency for the repair to become isolated from adjacent undamaged areas. Consequently, there is a large porosity or chloride content differential between the patch and the substrate concrete which, in turn, may accelerate the steel corrosion in the adjoining concrete, requiring further repair around the initial patched area (Gu *et al*., 1994; Zhang and Qian, 2007).

Chemical reactivity (R<C)

The reactivity of the patching material with steel reinforcement and other embedded metals, with the aggregate in the concrete, or with specific sealers or protective coatings applied over the patch must also be considered. Patching materials with low to moderate pH provide little protection to reinforcement while highly alkaline material may attack potentially reactive aggregates in the concrete.

12.3 Durability assessment of repaired concrete structures

This section presents an approach for assessing the durability of repaired concrete structures, and outlines some instrumentation and data analysis techniques that could be used in field research.

12.3.1 Advantages and limitations of field research

Numerous laboratory studies (Emberson and Mays, 1990; Yuan and Marosszeky, 1991; Poston *et al*., 2001) have been conducted to evaluate the performance of repair systems for concrete structures; however, discrepancies between laboratory test results and actual field performance are frequently observed (Gulis *et al*., 1998; Hooton *et al*., 2006). Satisfactory field performance is a key factor in the selection of concrete repair systems for the repair of corrosion-damaged concrete structures. Field investigations of repaired concrete structures are necessary to develop guidelines for (i) adequate selection of concrete repair systems, (ii) improved repair procedures, and (iii) improved durability of repaired structures.

One approach that has proven successful in concrete durability research is to complement field research with laboratory testing and computer simulation. Although different in nature, these three components are interrelated and can complement each other if their advantages and limitations are clearly identified. One component is not sufficient in itself to allow a thorough understanding of the nature of the problem and the identification of the appropriate solution. While field research offers the ultimate test for the acceptance of a new technology, laboratory research, in a controlled environment, and computer simulations are more cost-effective and offer the necessary data that complement the fieldwork. As a result, their combined use provides a more comprehensive value-added assessment of durability. Due to the relatively high cost of conducting field research and the uncontrollable nature of most environmental parameters, not all key variables influencing the behaviour of repaired concrete structures can be assessed independently in the field. Because of that, it is important to monitor the key parameters at adequate locations in the structure in order to allow a complete analysis of the problem at hand with as few assumptions as possible. Laboratory testing of certain concrete properties is therefore necessary to characterize the concrete used in the field. Once calibrated with field and laboratory data, numerical models can be used effectively to predict the performance and durability of a repaired concrete structure under given loading and environmental conditions.

The long-term durability of a repaired concrete structure often depends on the early-age behaviour of the concrete repairs and resulting performance of the repaired structure, especially at a time when the repair concrete has not achieved full hydration and expected design strength and stiffness. Failure to ensure adequate early-age performance of concrete repairs may lead to additional deterioration of the structure, frequent maintenance activities, and shorter than expected service life. The major sources of problems at early ages are: (i) thermal cooling due to heat loss after cement hydration or to sudden decrease in ambient temperature; (ii) autogenous shrinkage of the repair concrete if the water–cement (w/cm) ratio is below 0.4; and (iii) insufficient curing due to low temperatures or reduced availability of curing water, leading to slow or incomplete strength development. All of these possible early-age problems can lead to cracking of the concrete repairs if movement is restrained by the old substrate concrete.

Sometimes, structure owners are reluctant to instrument their structures during construction, mainly due to logistic, safety, and liability concerns. In such situations, instrumenting concrete specimens made of the same field concrete, placed, cured, and exposed to the same climatic environment under which the actual structure is being constructed may be the next best option. An excellent example of such a strategy was reported by Santos *et al*. (2001). However, testing concrete specimens adjacent to a concrete structure being constructed may not provide an accurate picture of the structure behaviour. For instance, the temperature and relative humidity (RH) profiles in the specimens may not match those in the structure due to differences in geometry and scale. For this reason, the concrete of the structure and the concrete of the specimens may not have a similar maturity and hence may have different development rates of shrinkage, elasticity, creep, and strength. Moreover, differences in degrees of restraint and external loading conditions may result in dissimilar development of patterns of stresses and cracking over time.

12.3.2 Examples of field research combined with computer simulation and laboratory testing

For a complete assessment of the durability of a repaired concrete structure, different types of parameters may be obtained from laboratory testing on samples of the repair concrete and from field monitoring of the repaired structure. The following two paragraphs are examples that provide a good illustration of how laboratory testing can complement field testing for a realistic assessment of the performance of repaired concrete structures.

For a transient heat transfer analysis of the repaired concrete structure under field investigation, the analyst would require the knowledge of some key parameters, including: (i) intrinsic properties of concrete, such as thermal conductivity, specific heat, density, and cement hydration heat, which could be obtained from standard laboratory testing; and (ii) environmental parameters, such as ambient temperature, wind speed, and solar radiation, which should be obtained from field monitoring. Measurement of internal temperatures at sufficient locations in the cross-section of the repaired concrete element under consideration is an alternative to measuring the field parameters listed above. The same approach could be used for the moisture transfer analysis of drying in concrete by measuring the RH or moisture content of the concrete at several locations over time.

For a transient stress analysis, the analyst would require the determination of the time-dependent restrained shrinkage strain in the structure, in addition to temperature and moisture distributions calculated using a hygrothermal analysis or by direct field measurements. Since the restrained shrinkage strain cannot be measured directly in the field, it may be determined as the difference between the measured shrinkage strain in the structure and the free shrinkage strain measured on an unrestrained concrete specimen subjected to the same environmental conditions (Santos *et al*., 2001). To complete the stress analysis, one would need to determine the time-dependent modulus of elasticity and creep coefficient of the repair concrete. These two properties are best determined in the laboratory using combined free and restrained shrinkage testing under conditions similar to those found in the field (Cusson and Hoogeveen, 2007). To evaluate the risk of cracking of a repaired concrete element in a structure, the tensile strength of concrete cores taken from the structure, or concrete samples cured under similar conditions, needs to be measured.

12.4 Case studies

Keeping highway bridges and parking structures in good condition is crucial to economic productivity and public safety. This is a challenging problem in areas where de-icing salts are used. While de-icing salts vastly increase the safety of winter road travel, they are the main cause of deterioration of reinforced concrete structures. This section features three field projects in which the field performance and durability of repaired concrete structures exposed to severe environments were investigated.

12.4.1 Reconstructed structural concrete slabs in a parking structure

The deterioration of concrete parking structures due to shrinkage cracking and reinforcement corrosion is a widespread problem that presents an on-going challenge for their owners. The floors of parking structures are subject to chloride-laden moisture in the form of water or snow brought in by vehicles. Unlike bridges, the interior floors of parking structures are not rinsed off by rain. The exposure to chlorides may be even greater if the slabs are poorly drained. This can lead to severe corrosion of the steel reinforcement and subsequent cracking and spalling of the concrete.

Experimental program

A comprehensive retrofitting project was initiated in 2004 by Public Works and Government Services Canada (PWGSC) at the Laurier–Taché indoor parking structure in Gatineau (Québec), which had experienced severe reinforcement corrosion and concrete deterioration. As part of PWGSC's efforts to identify and assess new technologies that have the potential to improve the durability of structures in harsh environments, they have decided that a concrete (w/cm = 0.4) containing a hydrophobic pore-blocking admixture would be used for the construction of designated areas of an elevated floor (totalling 2000 m^2), in order to reduce chloride ingress in concrete. The retrofitting work consisted of the demolition of the existing structural concrete slabs and ramps (Fig. 12.4a), surface repair of existing prestressed concrete girders, and construction of new concrete slabs and ramps (Fig. 12.4b).

(a)

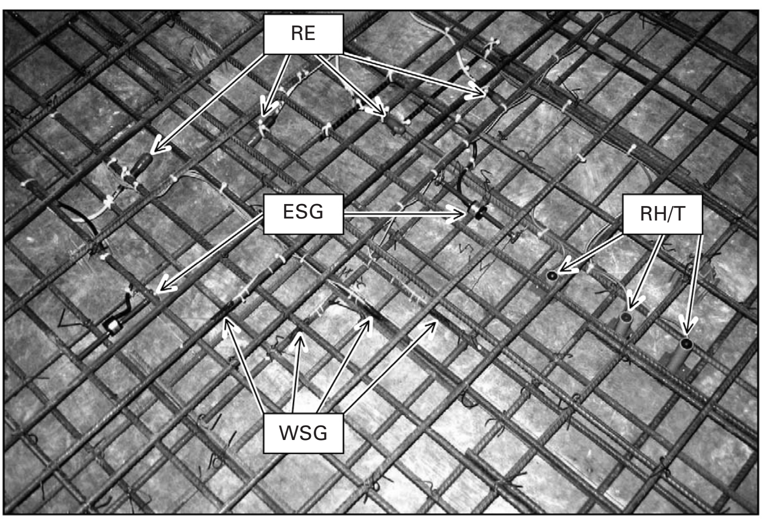
(b)

12.4 Laurier–Taché parking structure in Gatineau (Québec), Canada: (a) slab during demolition; (b) ramp after reconstruction.

These designated areas, including four slab sections and two interior ramps, were instrumented, monitored, and evaluated over an initial period of two years (Cusson *et al.*, 2007). Of the four slab sections, two were made of normal concrete (w/cm = 0.4) as references. Over 100 sensors were installed in the structure for this study. As shown in Fig. 12.5, each test section included: (i) three relative humidity and temperature (RH/T) sensors installed in the concrete at different depths (top, mid-height, and bottom) to measure the drying and temperature profiles; (ii) two embedment strain gauges (ESG) placed in concrete at mid-depth in both directions to measure drying shrinkage and detect cracks; (iii) four 10M 2m long reinforcing steel bars, each instrumented with two weldable strain gauges (WSG), attached to the top and bottom rebar layers in both directions; and (iv) four manganese dioxide reference electrodes (RE) installed at the top and bottom layers of reinforcement to monitor their corrosion potential. The data acquisition system selected for the field study consisted of a standalone micro-logger, several multiplexers, and a modem for remote communication.

Test results

After two years of structural health monitoring, temperature and relative humidity measurements indicated that the type of concrete (hydrophobic or normal) had no significant influence on the drying and thermal expansion of concrete. A typical concrete drying pattern was measured in this indoor structure, ranging from 100% RH shortly after the concrete was placed to

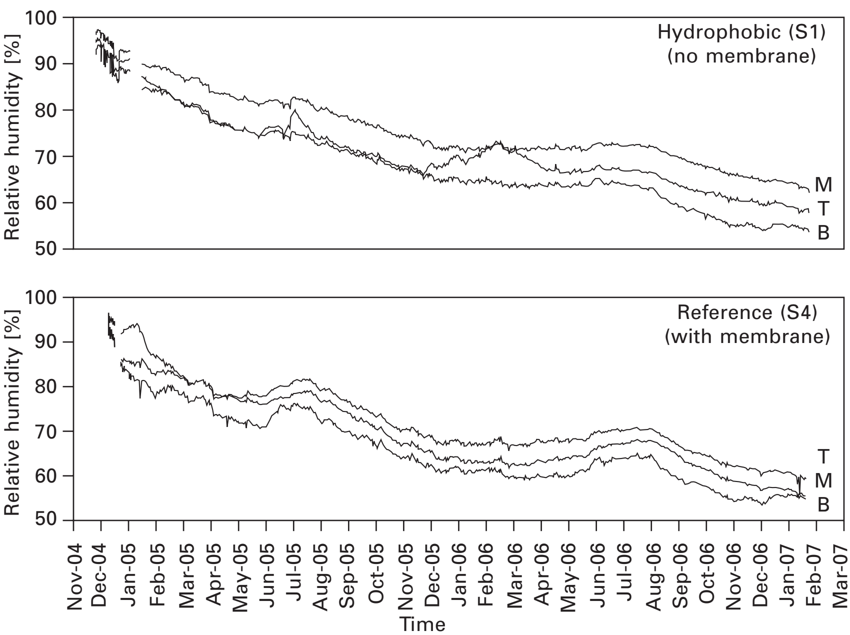


12.5 Typical instrumented section before placement of concrete.

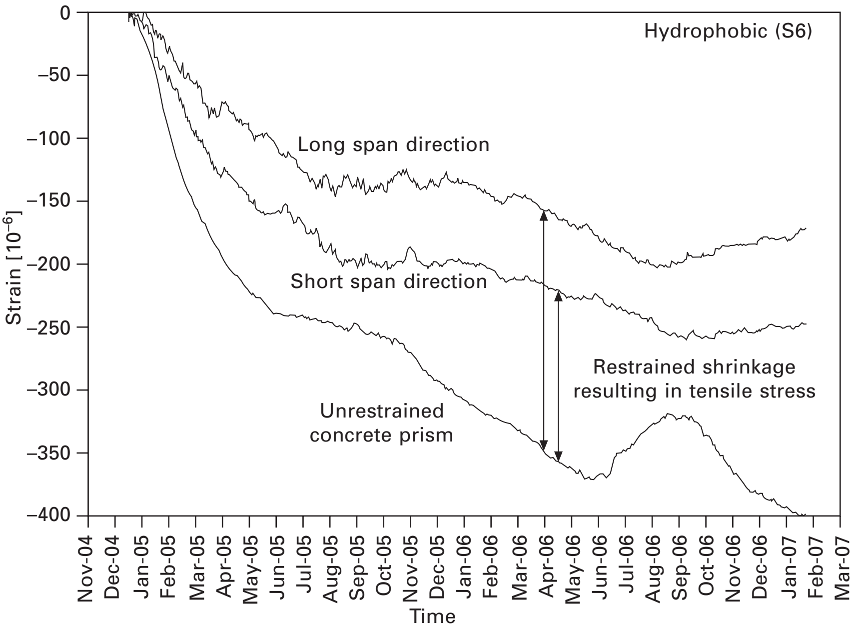
below 60% RH two years later (Fig. 12.6). Visual inspections indicated the presence of deep cracks at an early age in the six monitored concrete floor areas, including those made with hydrophobic concrete.

The strain measurements and stress calculations confirmed that this early-age cracking was due to drying shrinkage of the slabs, of which movement was restrained by the supporting prestressed concrete girders (maximum degree of restraint estimated at 50%). For example, Fig. 12.7 presents the net drying shrinkage strain measured in a hydrophobic concrete ramp in both orthogonal directions (i.e. short span and long span) and in a companion unrestrained concrete prism tested in the laboratory under conditions similar to those measured in the parking structure. It is shown that the drying shrinkage components resulting in tensile stresses were considerable even in the short term.

Flood tests confirmed that the hydrophobic concrete was not effective in preventing moisture migration through the floor cracks, even after repair with epoxy injection. The half-cell potential measurements taken on the embedded reinforcement indicated low risks of corrosion so far (after two years only). Given the extensive floor cracking and the presence of moisture and de-icing salts, long-term monitoring was recommended to track the corrosion resistance of the reinforcement, until a waterproofing membrane



12.6 Measured concrete relative humidity in concrete slabs. (B = bottom; M = middle; T = top)



12.7 Measured net drying shrinkage strain in hydrophobic concrete ramp.

is applied to prevent further ingress of moisture and chlorides to the slab reinforcement.

12.4.2 Concrete patch repairs on a bridge deck

To help address the need for a broadened knowledge base and new repair technologies, the National Research Council (NRC) and the Ministry of Transportation of Ontario (MTO) partnered in 1999 on a three-year project to field-test five proprietary commercial concrete repair systems (Cusson *et al.*, 2006). The goal was to study the effectiveness of commercial concrete repair systems in preventing corrosion of steel reinforcement and shrinkage cracking of ageing bridge decks.

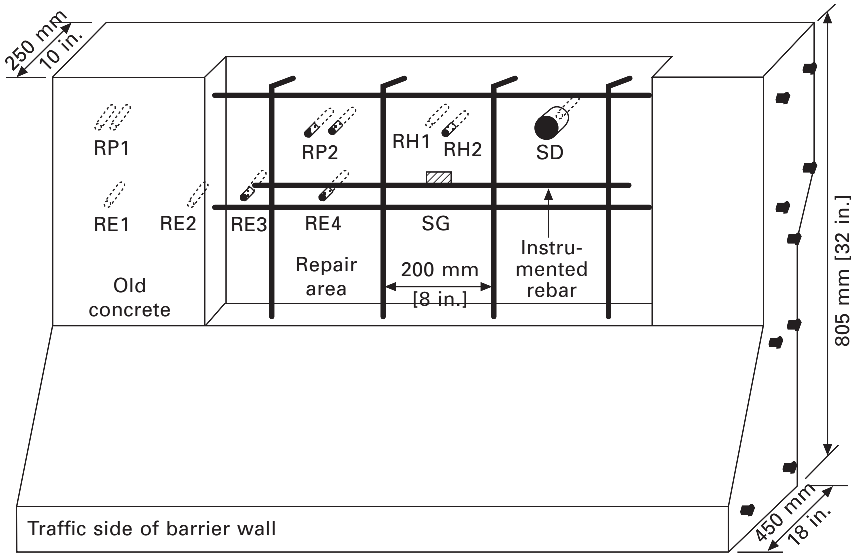
Experimental program

Testing for the project took place on an actual highway bridge, near Renfrew (Ontario), as shown in Fig. 12.8. Five proprietary commercial repair systems (including special concretes and corrosion-inhibiting admixtures) and one control system of normal concrete were used to create a series of test patches on different sections of corrosion-damaged reinforced concrete barrier walls. This type of location was chosen so that the test patches would be exposed

12.8 CNR bridge near Renfrew (Ontario), Canada, during repair and instrumentation.

to real outdoor conditions, including freeze–thaw cycles, wet–dry cycles, and de-icing salt contamination. The 28-day compressive strength of these repair concretes ranged from 25 MPa to 50 MPa.

During the patching process, embedded sensors were installed in the test sections and surrounding concrete (Fig. 12.9). In each test section, one RH sensor was inserted in the patch 50mm from the surface (i.e. at the reinforcement level), and another RH sensor was inserted in a hole drilled in the substrate concrete 50mm behind the 100mm thick patch. With these RH sensors, the aim was to assess the moisture gradient across the repaired section, which can provide information on the quality of moisture transfer between the patch and substrate, the risk of differential shrinkage, and the risk of freeze–thaw damage at the interface. Four manganese dioxide (MnO_2) RE were installed in a row 10mm behind one existing longitudinal reinforcing bar, which was located at a depth of 50mm from the surface. The aim with these RE was to detect a variation of the half-cell potential along the reinforcing bar going through both the repair and adjacent substrate, which can provide information on the formation of macro-cell corrosion in the substrate, possibly resulting in further corrosion damage (spalling or delamination) of the substrate concrete. Strain gauges (SG), previously installed on an 800mm long bar, were embedded longitudinally in the patch at a depth of 50mm from the surface; and a strain-gauge device (SD) was positioned transversally in the patch near the patch/substrate interface. The aim with these SG was to detect possible patch delamination if the strain



12.9 Location of sensors in centre of typical test section.

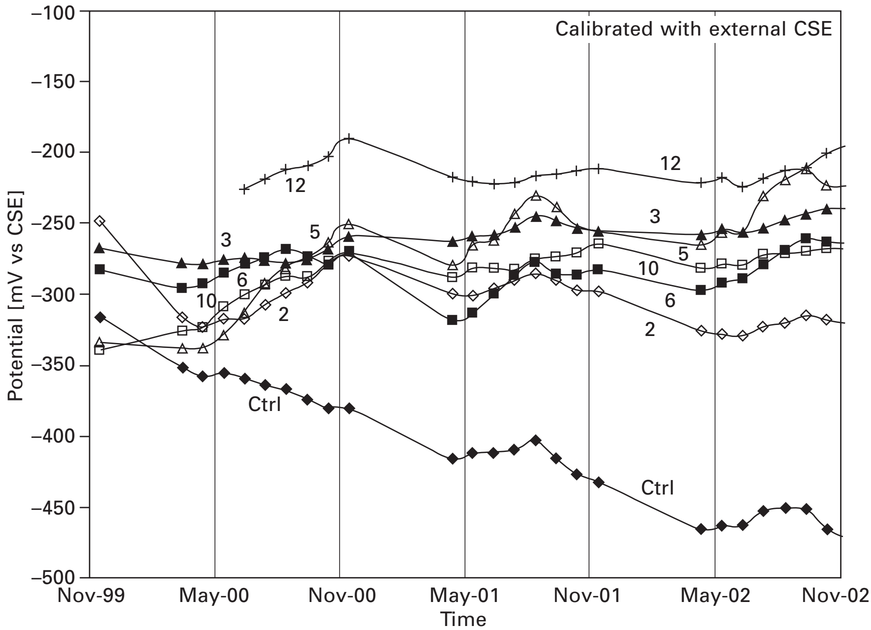
patterns did not match. Finally, one electrical resistance probe (RP) was embedded in the patch, and another one was inserted in a drilled hole in the old concrete beside the patch (Fig. 12.9). The probe tip was located approximately 50mm from the front surface of the barrier wall, which corresponded to the reinforcement depth. The aim with these probes was to obtain additional data to support and complement the data obtained from the RH sensors and RE.

The data acquisition system consisted of four data loggers equipped with a cellular modem for remote communication. The monitoring system was powered by three 12-volt lead-acid batteries, which were recharged by a set of three solar panels mounted on a 6 m high pole (Fig. 12.8) because AC power was not available at the site.

Once the patches were completed by the contractor and the sensors were in place, external monitoring using non-destructive test methods was periodically conducted. This approach provided both an inside and an outside view of the test sections for the assessment of the effectiveness of each commercial concrete repair system.

Test results

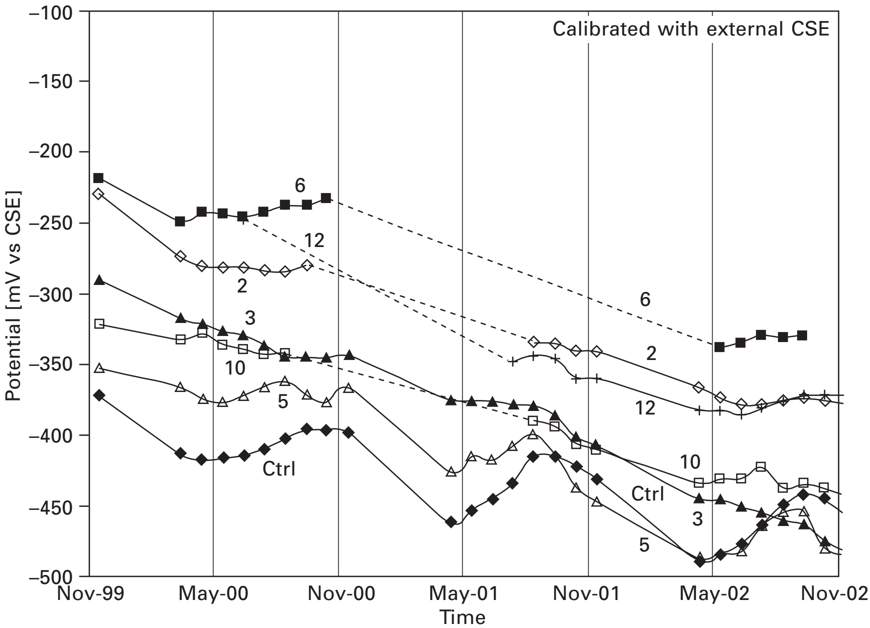
In general, the findings from both monitoring methods indicate that the commercial proprietary repair systems performed slightly better in delaying corrosion when compared to control repairs made of normal concrete. Figure 12.10 shows the monthly average of the corrosion potential of the reinforcing



12.10 Monthly average corrosion potential in concrete patches (RE4). CSE = copper sulfate electrode)

steel measured in the patch on electrode RE4 (located 400mm away from the old concrete). It is shown that the corrosion potential in the control section shifted towards more negative values, from –320 mV to –470 mV three years later, indicating an increased risk of reinforcement corrosion. In all other test sections, the corrosion potential in the patches remained practically unchanged with values between –200 mV and –350 mV, a range indicating that corrosion is uncertain according to ASTM C876. Figure 12.11 presents the monthly average of the corrosion potential of the reinforcing steel measured in the old concrete on electrode RE1 (located 400mm away from the patch). The curves show that the corrosion potential of the old concrete in all test sections shifted towards more negative values by more than 100 mV within the three-year period. This is an indication that the risk of corrosion in the substrate has increased after the repair.

Based on three years of field monitoring and supplementary laboratory test results, it was also found that the risks of microcell corrosion in the concrete patches were the lowest for the repair concretes that had the highest strengths, the highest freeze–thaw resistances and the lowest values of permeability. The risks of macrocell corrosion (anodic ring effect) in the nearby old concrete were the lowest for the repair concretes that had material properties similar to those of the old concrete, with lowest strengths and highest values of



12.11 Monthly average corrosion potential in old concrete (RE1).

permeability. The risk of microcell corrosion in the old concrete continued to increase after the repair, indicating that the concrete repair systems had little influence on the reinforcement corrosion developing in the old concrete. Shrinkage cracking, however, was observed in all patches tested in this study, including the control patching system.

Despite these results, it is believed that the delay in corrosion is enough to make the use of commercial proprietary concrete repair systems worthwhile, as long as the systems can provide for low water permeability, high electrical resistivity, and low shrinkage.

12.4.3 Reconstructed concrete barrier walls of a highway bridge

Corrosion inhibitors are considered among the most cost-effective protection techniques available for concrete structures. The mechanisms by which they protect the reinforcing steel are often complex. Some inhibitors delay corrosion by reducing the rate of the corrosion reactions, others by reducing the permeability of the concrete to chloride ions. Yet little information exists on the long-term performance of concrete structures built with corrosion-inhibiting systems.

Experimental program

A ten-year assessment of the field performance of bridge barrier walls rebuilt with reinforced concrete containing commercial corrosion-inhibiting systems was recently completed (Cusson and Qian, 2009) on a highway bridge near Laval (Québec). These barrier walls were all made of low-permeability concrete (e.g. w/cm = 0.36; cement content = 450 kg/m^3) and conventional steel reinforcement, except one that had epoxy-coated steel reinforcement.

Nine corrosion-inhibiting systems (and a control section) were investigated for ten years and included different combinations of concrete admixtures, reinforcing steel coatings, and/or concrete surface coatings/sealers. The field evaluation consisted mainly of annual corrosion surveys of (i) the main barrier wall reinforcement, protected by a 75 mm concrete cover and (ii) special rebar ladders embedded at specific locations in each barrier wall section at depths ranging from 13 mm up to 50 mm, for purposes of accelerating the corrosion evaluation.

Test results

A visual inspection of the barrier wall, carried out a few days after reconstruction, revealed closely spaced cracks running through the walls (Fig. 12.12), raising a concern for premature rebar corrosion due to accelerated

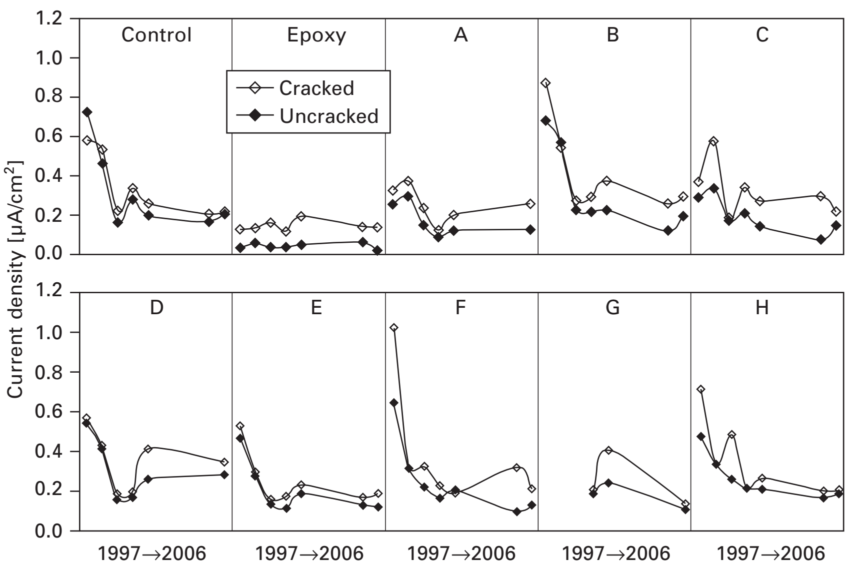
12.12 Barrier wall of Vachon bridge in Laval (Québec), Canada, after reconstruction.

moisture and salt ingress and eventual concrete spalling. Numerical modeling using field data confirmed that the early-age cracking was mainly due to uncontrolled thermal effects and autogenous shrinkage under restrained conditions, which is typical of concrete with high cement content and low w/cm ratio.

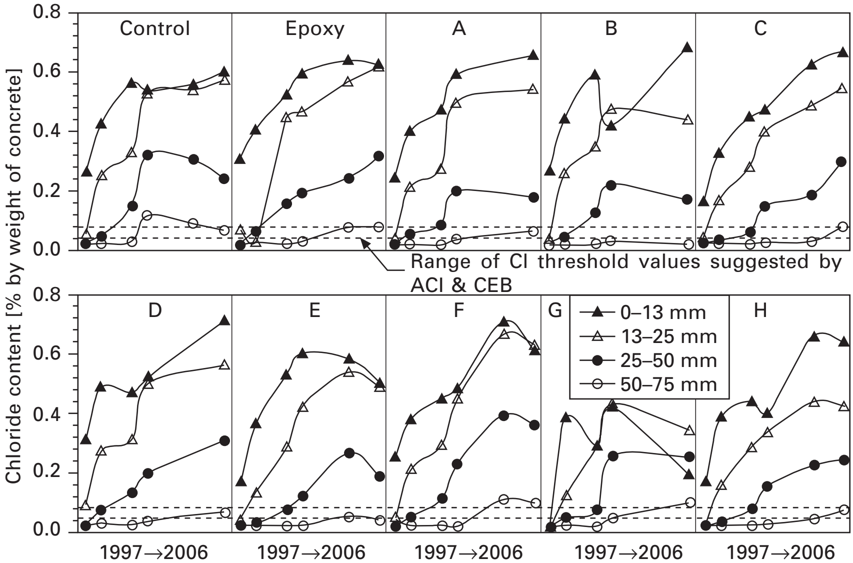
The corrosion survey of the 75 mm deep main reinforcement indicated that the risk of significant chloride-induced corrosion in all test sections, including the control section with no corrosion inhibitors, was still low after a decade of service. For example, Fig. 12.13 shows that after ten years the corrosion rates measured on the main reinforcement were still low. This is also supported by the measurements of total chloride contents (Fig. 12.14). At depths of 50–75 mm, the chloride contents remained below or near the critical chloride values in most test spans. This is an indication that corrosion of the main reinforcement may have initiated after only ten years of exposure.

Figure 12.14 also shows that, at shallow depths, the chloride contents exceeded the chloride corrosion threshold at early ages. A visual survey of the shallow rebar ladders in the barrier walls showed evidence of damage in all test spans at different degrees. For instance, the system using an inorganic concrete admixture consistently provided the best performance, showing the least amount of concrete damage (only minor cracks). The systems using organic concrete admixtures also provided good performance.

The obtained results clearly showed the importance of using low-permeability concrete and an adequate concrete cover (Mallett, 1994; Mather,



12.13 Corrosion rate measured over main reinforcement of barrier wall.



12.14 Total chloride content obtained on concrete cores from barrier wall.

2004) to provide the first line of defence in reinforced concrete structures exposed to severe environments. The corrosion-inhibiting systems provide a second line of defence in the event the concrete cover is breached because of cracking, or when critical chloride contamination reaches the steel.

12.5 Concluding remarks and recommendations

Although there were several types of construction materials, various repair procedures and different structures tested in the three field studies presented in this chapter, one major issue was common to all of them: early-age concrete cracking due to restrained shrinkage. It is evidenced that structural materials that have good performance track records in the laboratory may not necessarily perform well on real structures due to factors such as restrained thermal and drying shrinkage deformation and compatibility with the existing structure. As a recommendation for the design of concrete repairs, selecting a low-shrinkage repair material is essential to ensure durability of concrete structures. As shown in the case studies, failing to prevent early-age cracking will most likely lead to premature corrosion of the steel reinforcement and subsequent damage to the structure.

Effective and durable repairs can only be realized when a detailed diagnosis of the causes of deterioration has been made and given full consideration in the selection of materials that are both suitable for the given environment and service conditions, and compatible with the intended substrates. It is the responsibility of the design engineer to ensure that the selected repair material has these qualities so that it will last for the intended life of the repaired concrete structure.

12.6 Sources of further information and advice

ACI has an ongoing committee (201) dealing with durability of concrete, and has produced two documents titled: (i) *Guide to Durable Concrete* (2001) and (ii) *Guide for Making a Condition Survey of Concrete in Service* (1992, reapproved 1997), both of which are currently under revision. Similarly, RILEM has a committee (TC-214) looking at the effect of cracking on the durability of concrete structures. They are presently developing a state-of-the-art report focusing on integrating concrete cracking and material properties into durability calculations in order to improve the structural performance of concrete structures. RILEM TC-130 also published a report on Durability Design of Concrete Structures, which was published in *Materials and Structures* in January 2000. ISO Technical Committee TC 98 (*Bases for Design of Structures*) is currently preparing a document titled: General Principles on the Design of Structures for Durability, which mainly focuses on durability requirements, durability design, and service-life prediction of concrete structures.

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