Service-Life Prediction—State-of-the-Art Report

Reported by ACI Committee 365

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This report presents current information on the service-life prediction of new and existing concrete structures. This information is important to both the owner and the design professional. Important factors controlling the service life of concrete and methodologies for evaluating the condition of the existing concrete structures, including definitions of key physical properties, are also presented. Techniques for predicting the service life of concrete and the relationship between economics and the service life of structures are discussed. The examples provided discuss which service-life techniques are applied to concrete structures or structural components. Finally, needed developments are identified.

Keywords: construction; corrosion; design; durability; rehabilitation; repair; service life.

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CHAPTER 1—INTRODUCTION

1.1—Background

Service-life concepts for buildings and structures date back to when early builders found that certain materials and designs lasted longer than others (Davey 1961). Throughout history, service-life predictions of structures, equipment, and other components were generally qualitative and empirical. The understanding of the mechanisms and kinetics of many degradation processes of concrete has formed a basis for making quantitative predictions of the service life of structures and components made of concrete. In addition to actual or potential structural collapse, many other factors can govern the service life of a concrete structure. For example, excessive operating costs can lead to a structure's replacement. This document reports on these service-life factors, for both new and existing concrete structures and components.

The terms "durability" and "service life" are often erroneously interchanged. The distinction between the two terms is evident when their definitions, as given in ASTM E 632, are compared:

Durability is the capability of maintaining the serviceability of a product, component, assembly, or construction over a specified time. Serviceability is viewed as the capacity of the above to perform the function(s) for which they are designed and constructed.

Service life (of building component or material) is the period of time after installation (or in the case of concrete, placement) during which all the properties exceed the minimum acceptable values when routinely maintained. Three types of service life have been defined (Sommerville 1986). Technical service life is the time in service until a defined unacceptable state is reached, such as spalling of concrete, safety level below acceptable, or failure of elements. Functional service life is the time in service until the structure no longer fulfills the functional requirements or becomes obsolete due to change in functional requirements, such as the needs for increased clearance, higher axle and wheel loads, or road widening. Economic service life is the time in service until replacement of the structure (or part of it) is economically more advantageous than keeping it in service.

Service-life methodologies have application both in the design stage of a structure—where certain parameters are established, such as selection of water-cementitious materials ratios (*w/cm*), concrete cover, and admixtures—and in the operation phase where inspection and maintenance strategies can be developed in support of life-cycle cost analyses. Service-life design includes the architectural and structural design, selection and design of materials, maintenance plans, and quality assurance and quality control plans for a future structure (CEB/RILEM 1986). Based on mixture proportioning, including selection of concrete constituents, known material properties, expected service environment, structural detailing (such as concrete cover), construction methods, projected loading history, and the definition of end-of-life, the service life can be predicted and concrete with a reasonable assurance of meeting the design service life can be specified (Jubb 1992, Clifton and Knab 1989). The acceptance of advanced materials, such as high-performance concrete, can depend on life-cycle cost analyses that consider predictions of their increased service life.

Methodologies are being developed that predict the service life of existing concrete structures. To predict the service life of existing concrete structures, information is required on the present condition of concrete, rates of degradation, past and future loading, and definition of the end-of-life (Clifton 1991). Based on remaining life predictions, economic decisions can be made on whether or not a structure should be repaired, rehabilitated, or replaced.

Repair and rehabilitation are often used interchangeably. The first step of each of these processes should be to address the cause of degradation. The distinction between rehabilitation and repair is that rehabilitation includes the process of modifying a structure to a desired useful condition, whereas repair does not change the structural function.

To predict the service life of concrete structures or elements, end-of-life should be defined. For example, end-of-life can be defined as:

- Structural safety is unacceptable due to material degradation or exceeding the design load-carrying capacity;
- Severe material degradation, such as corrosion of steel reinforcement initiated when diffusing chloride ions attain the threshold corrosion concentration at the reinforcement depth;
- Maintenance requirements exceed available resource limits;
- Aesthetics become unacceptable; or
- Functional capacity of the structure is no longer sufficient for a demand, such as a football stadium with a deficient seating capacity.

Essentially all decisions concerning the definition of end-of-life are combined with human safety and economic considerations. In most cases, the condition, appearance, or capacity of a structure can be upgraded to an acceptable level; however, costs associated with the upgrade can be prohibitive. Guidance on making such decisions is included in this report.

1.2—Scope

This report begins with an overview of important factors controlling the service life of concrete, including past and current design of structures; concrete materials issues; field practices involved with placing, consolidating, and curing of concrete; and in-service stresses induced by degradation processes and mechanical loads. Methodologies used to evaluate the structural condition of concrete structures and the condition and properties of in-service concrete materials are presented. Methods are reviewed for predicting the service life of concrete, including comparative methods, use of accelerated aging (degradation) tests, application of mathematical modeling and simulation, and application of reliability and stochastic concepts. This is followed by a discussion of relationships between economics and the life of structures, such as when it is more economical to replace a structure than to repair or rehabilitate. Examples are described in which service-life techniques are applicable to concrete structures or structural components. Finally, needed developments to improve the reliability of service-life predictions are presented.

1.3—Document use

This document can assist in applying available methods and tools to predict service life of existing structures and provide actions that can be taken at the design or construction stage to increase service life of new structures.

CHAPTER 2—ENVIRONMENT, DESIGN, AND CONSTRUCTION CONSIDERATIONS

2.1—Introduction

Reinforced concrete structures have been and continue to be designed in accordance with national or international consensus codes and standards such as ACI 318, Eurocode 2, and Comité Euro International du Béton (1993). The codes are developed and based on knowledge acquired in research and testing laboratories, and supplemented by field experience. Although present design procedures for concrete are dominated by analytical determinations based on strength principles, designs are increasingly being refined to address durability requirements (for example, resistance to chloride ingress and improved freezing-and-thawing resistance). Inherent with design calculations and construction documents developed in conformance with these codes is a certain level of durability, such as requirements for concrete cover to protect embedded steel reinforcement under aggressive environmental conditions. Although the vast majority of reinforced concrete structures have met and continue to meet their functional and performance requirements, numerous examples can be cited where structures, such as pavements and bridges, have not exhibited the desired durability or service life. In addition to material selection and proportioning to meet concrete strength requirements, a conscious effort needs to be made to design and detail pavements and bridges for long-term durability (Sommerville 1986). A more holistic approach is necessary for designing concrete structures based on service-life considerations. This chapter addresses environmental and structural loading considerations, as well as their interaction, and design and construction influences on the service life of structures.

2.2—Environmental considerations

Design of reinforced concrete structures to ensure adequate durability is a complicated process. Service life depends on structural design and detailing, mixture proportioning, concrete production and placement, construction methods, and maintenance. Also, changes in use, loading, and environment are important. Because water or some other fluid is involved in almost every form of concrete degradation, concrete permeability is important.

The process of chemical and physical deterioration of concrete with time or reduction in durability is generally dependent on the presence and transport of deleterious substances through concrete,* and the magnitude, frequency, and effect of applied loads. Figure 2.1 (CEB 1992) presents the relationship between the concepts of concrete durability and performance. The figure shows that the combined transportation of heat, moisture, and chemicals, both within the concrete and in exchange with the surrounding environment, and the parameters controlling the transport mechanisms constitute the principal elements of durability. The rate, extent, and effect of fluid transport are largely dependent on the concrete pore structure (size and distribution), presence of cracks, and microclimate at the concrete surface. The primary mode of transport in uncracked concrete is through the bulk cement paste pore structure and the transition zone (interfacial region between the particles of coarse aggregate and hydrated cement paste). The physical-chemical phenomena associated with fluid movement through porous solids is controlled by the solid's permeability (penetrability). Although the coefficient of permeability of concrete depends primarily on the *w/cm* and maximum aggregate size, it is also influenced by age, consolidation, curing temperature, drying, and the addition of chemical or mineral admixtures. Concrete is generally more permeable than cement paste due to the presence of microcracks in the transition zone between the cement paste and aggregate (Mehta 1986). Table 2.1 presents chloride diffusion and permeability results obtained from the 19 mm maximum size crushed limestone aggregate mixtures presented in Table 2.2.† Additional information on the types of transport processes important with respect to the various aspects of concrete durability, such as simple diffusion, diffusion plus reaction, imbibition (capillary suction), and permeation, is available

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*Absorption is the process by which a liquid is drawn into and tends to fill permeable pores in a porous solid body; also the increase in mass of a porous solid body resulting from the penetration of a liquid into its permeable pores. Permeability is defined as the ease with which a fluid can flow through a solid. Diffusion is the movement of one medium through another.

†The results presented are for this testing method, and would be somewhat different if another testing method had been used.

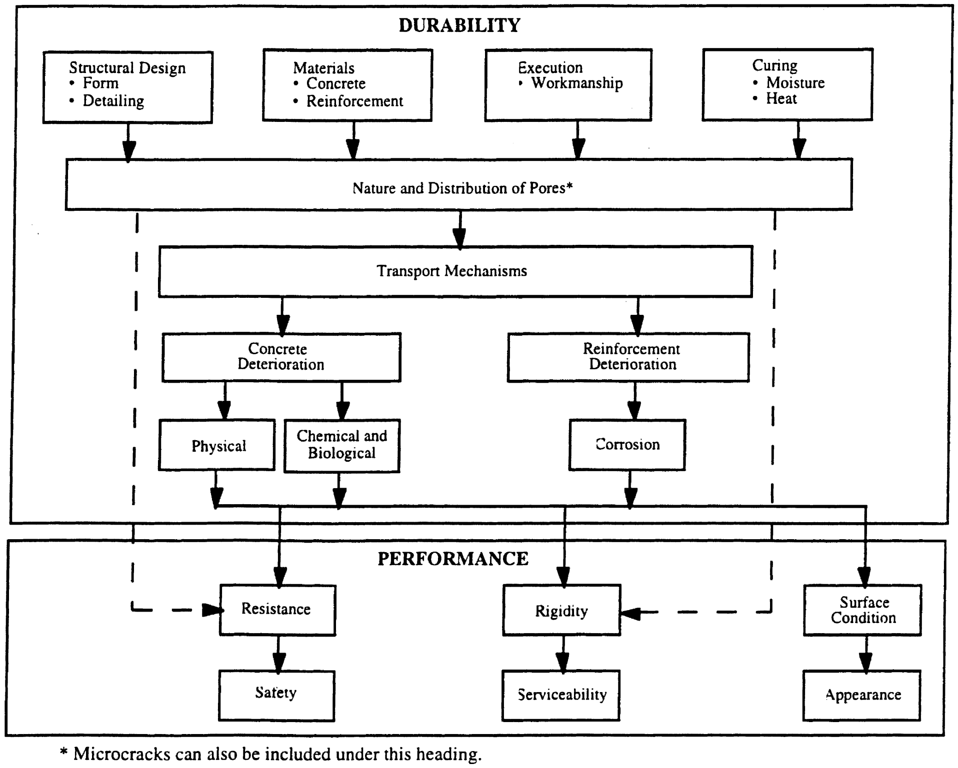

Fig. 2.1—Relationships between the concepts of concrete durability and performance (CEB 1992).

Table 2.1—Chloride transport and permeability results for selected concretes*

Mixture no.†	Cure time, days	Rapid test for permeability to Cl^-, 3% NaCl solution, total charge, Coulombs	90-day ponding, % Cl^- by weight of concrete‡	Permeability, μ Darcys§		Porosity, % by volume
				Hydraulic	Air	
1	1	44	0.013	—‖	37	8.3
	7	65	0.013	—‖	29	7.5
2	1	942	0.017	—‖	28	9.1
	7	852	0.022	—‖	33	8.8
3	1	3897	0.062	0.030	130	11.3
	7	3242	0.058	0.027	120	11.3
4	1	5703	0.103	0.560	120	12.4
	7	4315	0.076	0.200	170	12.5
5	1	5911	0.104	0.740	200	13.0
	7	4526	0.077	0.230	150	12.7
6	1	7065	0.112	4.100	270	13.0
	7	5915	0.085	0.860	150	13.0

*Whiting, 1988.
†Refer to Table 2.2 for description of mixtures.
‡Average of three samples taken at depths from 2 to 40 mm.
§To convert from μ Darcys to m^2, multiply by 9.87×10^{-7}.
‖Permeability too small to measure.

elsewhere (Lawrence 1991, Pommersheim and Clifton 1990, Kropp and Hilsdorf 1995).

Two additional factors are considered with respect to fabrication of durable concrete structures: the environmental-exposure condition and specific design recommendations pertaining to the expected form of aggressive chemical or physical attack (for example, designing the structure to prevent accumulation of water). Exposure conditions or severity are generally handled through a specification that addresses the concrete mixture (for example strength, *w/cm*, and cement content), and details (such as concrete cover), as dictated by the anticipated exposure. Summarized in the following paragraphs are descriptions of the primary chemical and physical degradation processes that can adversely impact the durability of reinforced concrete structures and guidelines for minimizing or eliminating potential consequences of

Table 2.2—Concrete mixture proportions and characteristics*

Mixture no.	Quantities, kg/m^3				Admixture(s)†	*w/cm*	Slump, cm	Air content, %
	Cement	Fine aggregate	Coarse aggregate	Water				
1	446	752	1032	132	A + B	0.258‡	119	1.6
2	446	790	1083	128	C	0.288	89	2.0
3	381	784	1075	153	D	0.401	89	2.3
4	327	794	1088	164	—	0.502	94	2.1
5	297	791	1086	178	—	0.600	107	1.8
6	245	810	1107	185	—	0.753	124	1.3

*Whiting, 1988.

†A = Microsilica fume at 59.4 kg/m^3; B = Type F high-range water reducer at 25 ml/kg; C = Type F high-range water reducer at 13 ml/kg; and D = Type A water reducer at 2 ml/kg.

‡For Mixture 1 expressed as ratio of water to total cementitious material content.

these degradation mechanisms. Combined effects where more than one of these processes can be simultaneously occurring are also briefly addressed. Available methods and strategies for prediction of the service life of a new or existing reinforced concrete structure with respect to these mechanisms are described in Chapter 4.

2.2.1 *Chemical attack*—Chemical attack involves the alteration of concrete through chemical reaction with either the cement paste, coarse aggregate, or embedded steel reinforcement. Generally, the attack occurs on the exposed surface region of the concrete (cover concrete), but with the presence of cracks or prolonged exposure, chemical attack can affect entire structural cross sections. Chemical causes of deterioration can be grouped into three categories (Mehta 1986):

1. Hydrolysis of cement paste components by soft water;
2. Cation-exchange reactions between aggressive fluids and cement paste; and
3. Reactions leading to formation of expansion product.

Results from prolonged chemical attack range from cosmetic damage to loss of structural section and monolithic behavior. Chemical attack of embedded steel reinforcement can also occur.

2.2.1.1 *Leaching*—Pure water that contains little or no calcium ions, or acidic ground water present in the form of dissolved carbon dioxide gas, carbonic acid, or bicarbonate ion, tend to hydrolyze or dissolve the alkali oxides and calcium-containing products resulting in increasing permeability. The rate of leaching is dependent on the amount of dissolved salts contained in the percolating fluid, rate of permeation of the fluid through the cement paste matrix, and temperature. The rate of leaching can be lowered by minimizing the permeation of water through the concrete (interconnected capillary cavities) by using low-permeability concretes and barriers. Factors related to the production of low-permeability concretes include low *w/cm*, adequate cement content, pozzolanic additions, and proper compaction and curing conditions. Polymeric modification can also be used to provide low permeability concretes. Similarly, attention should be given to aggregate size and gradation, thermal and drying shrinkage strains, avoiding loads that produce cracks, and designing and detailing to minimize exposure to moisture. Requirements in codes and suggested guidelines for *w/cm* are generally based on strength or exposure conditions (ACI 318, ACI 201.1R, ACI 301, ACI 350R, ACI 357R). ACI 224R provides crack-control guidelines and ACI 515.1R provides information on barrier systems for concrete.

2.2.1.2 *Delayed ettringite formation*—Structures undergoing delayed ettringite formation (DEF) can exhibit expansion and cracking. The distress often is attributed to excessive steam curing that prevents the formation or causes decomposition of ettringite that is normally formed during the early hydration of portland cement. Use of cements with high sulfate contents in which the sulfate has very low solubility can also lead to DEF. In one case where this has been reported (Mielenz et al. 1995), it was thought that the occurrence of DEF was due to the sulfate formed in the clinker of the cement being present as anhydrite and as a component of the silicate phases which are slowly soluble. Ettringite is the product of the reaction between sulfate ions, calcium aluminates, and water. If structures susceptible to DEF are later exposed to water, ettringite can reform in the paste as a massive development of needle-like crystals, causing expansive forces that result in cracking. The extent of development of DEF is dependent on the amount of sulfate available for late ettringite development in the particular concrete and on the presence of water during the service life. Elevated temperatures also increase the potential for damage due to DEF. Prevention or minimization of DEF can be accomplished by lowering the curing temperature, limiting clinker sulfate levels, avoiding excessive curing for potentially critical sulfate to aluminate ratios, preventing exposure to substantial water in service, and using proper air entrainment. Neither the mechanisms involved in DEF nor their potential consequences relative to concrete durability are completely understood. DEF leads to a degradation in concrete mechanical properties, such as compressive strength, and can promote increased permeability. A detailed review of over 300 publications dealing with DEF is available (Day 1992).

2.2.1.3 *Sulfate attack*—Sulfates present in the aggregates, soils, ground water, and seawater react with the calcium hydroxide [$Ca(OH)_2$] and the hydrated tricalcium aluminate (C_3A) to form gypsum and ettringite, respectively. These reactions can result in deleterious expansion and produce concretes with reduced strength because of decomposition and expansion of the hydrated calcium aluminates.

Increased resistance of structures to sulfate attack is provided by fabricating them using concrete that is dense, has low permeability, and incorporates sulfate-resistant cement. Because it is the C_3A that is attacked by sulfates, the concrete vulnerability can be reduced by using cements low in C_3A, such as ASTM C 150 Types II and V sulfate-resisting cements. Under extreme conditions, supersulfated slag cements such as ASTM C 595 Types VP or VS can be used. Also, improved sulfate resistance can be attained by using admixtures, such as pozzolans and blast-furnace slag. Requirements and guidelines for the use of sulfate-resistant concretes are based on exposure severity and are provided in ACI 318 and ACI 201.2R. The requirements are provided in terms of cement type, cement content, maximum *w/cm*, and minimum compressive strength, depending upon the potential for distress.

2.2.1.4 *Acid and base attack*—Acids can combine with the calcium compounds in the hydrated cement paste to form soluble materials that are readily leached from the concrete to increase porosity and permeability. The main factors determining the extent of attack are type of acid, and its concentration and pH. Protective barriers are recommended to provide resistance against acid attack.

As hydrated cement paste is an alkaline material, concrete made with chemically stable aggregates is resistant to bases. Sodium and potassium hydroxides in high concentrations (>20%), however, can cause concrete to disintegrate. ACI 515.1R provides a list of the effects of chemicals on concrete. Under mild chemical attack, a concrete with low *w/cm* (low permeability) can have suitable resistance. Because corrosive chemicals can attack concrete only in the presence of water, designs to minimize attack by bases might also incorporate protective barrier systems. Guidelines on the use of barrier systems are also provided in ACI 515.1R.

2.2.1.5 *Alkali-aggregate reactions*—Expansion and cracking leading to loss of strength, stiffness, and durability of concrete can result from chemical reactions involving alkali ions from portland cement, calcium and hydroxyl ions, and certain siliceous constituents in aggregates. Expansive reactions can also occur as a result of interaction of alkali ions and carbonate constituents. Three requirements are necessary for disintegration due to alkali-aggregate reactions: 1) presence of sufficient alkali; 2) availability of moisture; and 3) the presence of reactive silica, silicate, or carbonate aggregates. Controlling alkali-aggregate reactions at the design stage is done by avoiding deleteriously reactive aggregate materials by using preliminary petrographic examinations and by using materials with proven service histories. ASTM C 586 provides a method for assessing potential alkali reactivity of carbonate aggregates. ACI 201.2R presents a list of known deleteriously reactive aggregate materials. Additional procedures for mitigating alkali-silica reactions include pozzolans, using low-alkali cements (that is, restricting the cement alkali contents to less than 0.6% by weight sodium oxide [Na_2O] equivalent), adding lithium salts, and applying barriers to restrict or eliminate moisture. The latter procedure is generally the first step in addressing affected structures. The alkali-carbonate reaction can be controlled by keeping the alkali content of the cement low, by adding lithium salts, or by diluting the reactive aggregate with less-susceptible material.

2.2.1.6 *Steel reinforcement corrosion*—Corrosion of conventional steel reinforcement in concrete is an electrochemical process that forms either local pitting or general surface corrosion. Both water and oxygen must be present for corrosion to occur. In concrete, reinforcing steel with adequate cover should not be susceptible to corrosion because the highly alkaline conditions present within the concrete (pH>12) cause a passive iron-oxide film to form on the steel surface. Carbonation and the presence of chloride ions, however, can destroy the protective film. Corrosion of steel reinforcement also can be accelerated by the presence of stray electrical currents.

Penetrating carbon dioxide (CO_2) from the environment reduces the pH of concrete as calcium and alkali hydroxides are converted into carbonates. The penetration of CO_2 generally is a slow process, dependent on the concrete permeability, the concrete moisture content, the CO_2 content, and ambient relative humidity (RH). Carbonation can be accelerated by the presence of cracks or porosity of the concrete. Concretes that have low permeability and have been properly cured provide the greatest resistance to carbonation. Also, concrete cover over the embedded steel reinforcement can be increased to delay the onset of corrosion resulting from the effects of carbonation.

The presence of chloride ions is probably the major cause of corrosion of embedded steel reinforcement. Chloride ions are common in nature and small amounts can be unintentionally contained in the concrete mixture ingredients. Potential external sources of chlorides include those from accelerating admixtures (for example, calcium chloride), application of deicing salts, or exposure to seawater or spray. Maximum permissible chloride-ion contents, as well as minimum concrete cover requirements, are provided in codes and guides (CEB 1993, ACI 318, ACI 222R, and ACI 201.2R). Two methods are most commonly used for determination of chloride contents in concrete: acid soluble test (total chlorides), and water-soluble test. The chloride ion limits are presented in terms of type of member (prestressed or conventionally reinforced) and exposure condition (dry or moist). Because water, oxygen, and chloride ions are important factors in the corrosion of embedded steel reinforcement, concrete permeability is the key to controlling the process. Concrete mixtures should be designed to ensure low permeability by using low *w/cm*, adequate cementitous materials content, proper aggregate size and gradation, and mineral admixtures. Methods of excluding external sources of chloride ions from existing concrete, detailed in ACI 222R, include using waterproof membranes, polymer impregnation, and overlay materials. ACI 222R also notes that enhanced corrosion resistance can be provided by corrosion-resistant steels, such as stainless steel or stainless steel cladding; application of sacrificial or nonsacrificial coatings, such as fusion-bonded epoxy powder; use of chemical admixtures, such as corrosion inhibitors during the construction stage; and cathodic protection, either during the construction stage or later in life. Additional information on barriers that can be used to enhance corrosion resistance is provided in ACI 515.1R. The resistance of structures can also be increased by designing and detailing them to promote the runoff of moisture. Maintenance efforts to minimize a structure's exposure to chlorides and other aggressive chemicals should also be instituted.

2.2.1.7 *Prestressing steel corrosion*—High-strength steel, such as that used in pre- or post-tensioning systems, corrodes in the same manner as mild steel. In addition, it can degrade due to corrosion fatigue, stress corrosion cracking, and hydrogen embrittlement. Microorganisms can also cause corrosion by creating local environments conducive to the corrosion process through the intake of available food products and production of highly acidic waste products in the environment around the reinforcement. Although corrosion of prestressing steel can be either highly localized or uniform, most prestressing corrosion-related failures have been the result of localized attack resulting in pitting, stress corrosion, hydrogen embrittlement, or a combination of these. Pitting is an electrochemical process that results in local penetrations into the steel to reduce the cross section so that it is incapable of supporting its load. Stress-corrosion cracking results in the brittle fracture of a normally ductile metal or alloy under stress (tension or residual) in specific corrosive environments. Hydrogen embrittlement, frequently associated with exposure to hydrogen sulfide, occurs when hydrogen atoms enter the metal lattice and significantly reduce its ductility. Hydrogen embrittlement can also occur as a result of improper application of cathodic protection to the post-tensioning system. Due to the magnitude of the load in the post-tensioning systems, the tolerance for corrosion attack is less than for mild steel reinforcement. Corrosion protection is provided at installation by either encapsulating the post-tensioning steel with microcrystalline waxes compounded with organic corrosion inhibitors within plastic sheaths or metal conduits (unbounded tendons), or by portland cement (grouted tendons). Degradation of prestressing steel is critical because of its potential effects on monolithic behavior, tensile capacity, and ductility.

2.2.2 *Physical attack*—Physical attack generally involves the degradation of concrete due to environmental influences. It primarily manifests itself in two forms: surface wear and cracking (Mehta and Gerwick 1982). Concrete damage due to overload is not considered in this document but can lead to loss of durability because the resulting cracks can provide direct pathways for entry of deleterious chemicals (for example, exposure of steel reinforcement to chlorides).

2.2.2.1 *Salt crystallization*—Salts can produce cracks in concrete through development of crystal growth pressures that arise from causes, such as repeated crystallization due to evaporation of salt-laden water in the pores. Structures in contact with fluctuating water levels or in contact with ground water containing large quantities of dissolved salts (calcium sulfate [$CaSO_4$], sodium chloride [NaCl], sodium sulfate [Na_2SO_4]) are susceptible to this type of degradation, in addition to possible chemical attack, either directly or by reaction with cement or aggregate constituents. One approach to the problem of salt crystallization is to apply sealers or barriers to either prevent water ingress or subsequent evaporation; however, if the sealer is not properly selected and applied, it can cause the moisture content in the concrete to increase, and not prevent the occurrence of crystallization.

2.2.2.2 *Freezing-and-thawing attack*—Concrete, when in a saturated or near-saturated condition, is susceptible to damage during freezing-and-thawing cycles produced by the natural environment or industrial processes. One hypothesis is that the damage is caused by hydraulic pressure generated in the capillary cavities of the cement paste in a critically saturated condition as the water freezes. Factors controlling the resistance of concrete to freezing-and-thawing action include air entrainment (size and spacing of air voids), permeability, strength, and degree of saturation. Selection of durable aggregate materials is also important. Guidelines for production of freezing-and-thawing resistant concrete are provided in ACI 201.2R and ACI 318 in terms of total air content as a function of maximum aggregate size and exposure condition. Requirements for maximum permissible *w/cm* are also provided, based on the concrete cover and presence of aggressive agents, such as deicing chemicals. Because the degree of saturation is important, concrete structures should be designed and detailed to promote good drainage. ASTM C 666 is used to indicate the effects of variations in the properties of concrete on the resistance to internal damage due to freezing-and-thawing cycles. Ranking concrete according to resistance to freezing and thawing (critical dilatation) for defined curing and conditioning procedures can be accomplished through ASTM C 671. This test allows the user to specify the curing history of the specimen and the exposure conditions that most nearly match the expected service conditions. An estimate of the susceptibility of concrete aggregates for known or assumed field environmental conditions is provided in ASTM C 682. The effect of mixture proportioning, surface treatment, curing, or other variables on the resistance of concrete to scaling can be evaluated using ASTM C 672. These procedures are primarily for comparative purposes and are not intended to provide a quantitative measure of the length of service that can be expected from a specific type of concrete. Also, not all testing methods include criteria or suggestions for acceptance. Structures constructed without adequate air entrainment can have an increased risk for freezing-and-thawing damage.

2.2.2.3 *Abrasion, erosion, and cavitation*—Abrasion, erosion, and cavitation of concrete results in progressive loss of surface material. Abrasion generally involves dry attrition, while erosion involves a fluid containing solid particles in suspension. Cavitation causes loss of surface material through the formation of vapor bubbles and their sudden collapse. The abrasion and erosion resistance of concrete is affected primarily by the strength of the cement paste, the abrasion resistance of the fine and coarse aggregate materials, and finishing and curing. Special toppings, such as dry-shake coats of cement and iron aggregate on the concrete surface, can be used to increase abrasion resistance. If unchecked, abrasion or erosion can progress from cosmetic to structural damage over a fairly short time frame. Guidelines for development of abrasion and erosion-resistant concrete structures are provided in ACI 201.2R and ACI 210R, respectively. Concrete that resists abrasion and erosion can still suffer severe loss of surface material due to cavitation. The best way to guard against the effects of cavitation is to eliminate its cause(s).

2.2.2.4 *Thermal damage*—Elevated temperature and thermal gradients affect concrete's strength and stiffness. In addition, thermal exposure can result in cracking or, when the rate of heating is high and concrete permeability low, surface spalling can occur. Resistance of concrete to daily temperature fluctuations is provided by embedded steel reinforcement as described in ACI 318. A design-oriented approach for considering thermal loads on reinforced concrete structures is provided in ACI 349.1R. Limited information on the design of temperature-resistant concrete structures is available (ACI 216R, ACI SP-80). ACI 349 and ACI 359 generally handle elevated temperature applications by requiring special provisions, such as cooling, to limit the concrete temperature to a maximum of 65 C, except for local areas where temperatures can increase to 93 C. At that temperature, there is the potential for DEF to occur if concrete is also exposed to moisture. These codes, however, do allow higher temperatures if tests have been performed to evaluate the strength reduction, and the design capacity is computed using the reduced strength. Because the response of concrete to elevated temperature is generally the result of moisture change effects, guidelines for development of temperature-resistant reinforced concrete structures need to address factors, such as type and porosity of aggregate, permeability, moisture state, and rate of heating.

2.2.3 *Combined effects*—Degradation of concrete, particularly in its advanced stages, is seldom due to a single mechanism. The chemical and physical causes of degradation are generally so intertwined that separating the cause from the effect often becomes impossible (Mehta 1986). Limited information is available relative to the assessment of the remaining service life of concrete exposed to the combined effects of freezing-and-thawing degradation (surface scaling) and corrosion of steel reinforcement (Fagerlund et al. 1994).

2.3—Design and structural loading considerations

Designers of a new project involving concrete structures address service life by defining several critical concrete parameters. These include items such as *w/cm*, admixtures, reinforcement protection (cover or use of epoxy coating), and curing methods. The designer also verifies numerous serviceability criteria, such as deflection and crack width. Other factors to promote durability are also addressed at this stage (for example, drainage to minimize moisture accumulation and joint details).

Many of the parameters important to service life are established by ACI 318. Error, omission, or improper identification of these parameters are design deviations that can compromise construction. For example, a structure's exposure rating is either deemed severe due to vehicles carrying salted water into a parking garage, or moderate, assuming that salt water provided from other sources is marginal. Because that decision affects the ACI 318 required *w/cm*, it affects the price of the concrete. Improper selection of the exposure rating can lead to a more permeable concrete resulting in faster chloride penetration and diminished service life.

Another important design parameter is the definition of structural loads. Minimum design loads and load combinations are prescribed by legally adopted building codes (for example, ACI 318). There is a balance between selection of a design to meet minimum loading conditions and selection of a more conservative design that results in higher initial price but can provide lower life-cycle cost. The longevity of a structure designed to meet minimum loads prescribed by the building code or responsible agency can be more susceptible to degradation than the more conservative design. This is considered further in Section 2.4.

2.3.1 *Background on code development*—While AASHTO (1991) specifies a 75-year design life for highway bridges, ACI 318 makes no specific life-span requirements. Other codes, such as Eurocode, are based on a design life of 50 years, but not all environmental exposures are considered. ACI 318 addresses serviceability through strength requirements and limitations on service load conditions. Examples of service-load limitations include midspan deflections of flexural members, allowable crack widths, and maximum service level stresses in prestressed concrete. Other conditions affecting service life are applied to the concrete and the reinforcement material requirements and detailing. These include an upper limit on the concrete *w/cm*, a minimum entrained-air content depending upon exposure conditions, and concrete cover over the reinforcement. Most international design codes and guidelines have undergone similar changes in the past 30 years. For example, concretes exposed to freezing and thawing in a moist condition or to deicing chemicals, ACI 318-63 allowed a maximum *w/cm* of 0.52 and air entrainment, while ACI 318-89 allows a maximum *w/cm* of 0.45 with air entrainment. In 1963, an appendix was added to ACI 318 permitting strength design. Then in 1971, strength design was moved into the body of ACI 318, and allowable-stress design was placed into the appendix. The use of strength design provided more safety and it was possibly more cost-effective to have designs with a known, uniform factor of safety against collapse, rather than designs with a uniform, known factor of safety against exceeding an allowable stress. Realizing that design by strength limits alone could lead to some unsuitable conditions under service loads, service-load limitations listed above were adopted in ACI 318. The service-load limitations are based on engineering experience and not on any rigorous analysis of the effects of these limitations on the service life of the structure.

2.3.2 *Load and resistance factors*—Strength-design methods consider the loads (demands) applied to the structure and the resistance of the structure (capacity) to be two separate and independent conditions. The premise is that the strength of the structure should exceed the effects of the applied loads. Symbolically this can be written as

$$\text{Capacity} > \text{demand (over the desired service life).}$$

Formulation of this approach is done in two steps. First, the computed service loads are increased to account for uncertainties in the computation. Second, the strength of the structure is reduced by a resistance factor that reflects variations in material strengths and tolerances and also the effects of errors in predictive formulas and the possible consequence of failure.

The load and resistance factor calibration process deals exclusively with strength calculations. Service life, other than as affected by cover and concrete strength, generally is not a variable in the calibration process. Consequently, the selection of load and resistance factors, as currently formulated, offers no particular insight into the long-term performance of the structure. When AASHTO specifies a 75-year service life, the primary concern is fatigue effects on the reinforcement. AASHTO's service life is tied to a total number of vehicle passes. This leads to limitations on service load stresses in the reinforcement but not on the design load and resistance factors.

2.4—Interaction of structural load and environmental effects

Actions to eliminate or minimize any adverse effects resulting from environmental factors and designing structural components to withstand the loads anticipated while in service do not necessarily provide a means to predict the service life of a structure under actual field conditions (CEB 1992; Jacob 1965). The load-carrying capacity of a structure is directly related to the integrity of the main constituents during its service life. Therefore, a quantitative measure of the changes in the concrete integrity with time provide a means to estimate the service life of a structure.

Load tests on building components can be used to determine the effect of different design and construction methods and to predict the ability of the structure to withstand applied loads. The load-carrying capacity of components degraded over time due to environmental effects requires additional engineering analysis and judgment to determine their ability to withstand service loads. Often these evaluations are carried out at great expense, but they only provide short-term information and cannot adequately predict the long-term serviceability of the concrete (Kennedy 1958). Also, load tests can cause damage, such as cracking, that can lead to a reduction in durability and service life.

Many researchers have tried to quantify the environmentally induced changes by measuring the physical properties of concrete specimens after subjecting them to various combinations of load and exposure (Woods 1968; Sturrup and Clendenning 1969; Gerwick 1981). Most of the physical and mechanical properties are determined using relatively small specimens fabricated in the laboratory or sampled from structures. The properties measured reflect the condition of the specimens tested rather than the structure in the field because the test specimen and structure often are exposed to somewhat different environments. Quantifying the influence of environmental effects on the ability of the structure to resist the applied loads and to determine the rate of degradation as a result is a complex issue. The application of laboratory results to an actual structure to predict its response under a particular external influence requires engineering interpretation. The effect of external influences, such as exposure or curing conditions, on the changes in concrete properties has been reported (Neville 1991; Sturrup et al. 1987; Avram 1981; Price 1951). Guidance for prediction of change due to external influences is found in ACI 357R, ACI 209R, and ACI 215R.

As noted previously, the deleterious effects of environmentally related processes on the service life of concrete are controlled by two major factors: the presence of moisture and the transport mechanism controlling movement of moisture or aggressive agents (gas or liquid) within the concrete. The transport mechanism is controlled by the microstructure of the concrete, which in turn is a function of several other factors such as age, curing, and constituents. The microstructure comprises a network of pores and cracks in the concrete. The pore characteristics are a function of the original quality of the concrete, while cracking occurs in the concrete due to external loading as well as internal stresses. Ingress of aggressive agents is more likely to occur in the cracked region of the concrete than in an uncracked area. It is, therefore, possible that cracks occurring due to the service exposures affect the remaining service life of the concrete. Mercury-intrusion porosimetry is one method that determines pore-size distribution in concrete. Visual and microscopic techniques can determine the presence and extent of cracking in concrete.

A quantitative measurement of the concrete microstructure can be considered in terms of permeability. Models have been proposed to indicate the relationship between microstructure and permeability, however, they require validation. Most of the techniques for measuring concrete permeability are comparative and a standard test method does not exist. Attempts have been made to quantify pore-size characteristics from measurements of permeability or vice versa (Roy et al. 1992; Hooton 1986). Standard methods have also been developed for testing nonsteady-state water flow (Kropp and Hilsdorf 1995). Extensive development work is needed before such techniques can be applied to predict the remaining service life of a structure. Researchers have also proposed the development of indices for various degradation processes (Basson and Addis 1992). Periodic measurements of water, gas, chloride permeability, or depth of carbonation are means of quantifying the progressive change in the microstructure of concrete in service (Philipose et al. 1991; Ludwig 1980). This type of an approach has been used to predict the service life of dams subject to leaching of the cement paste by percolating soft water (Temper 1932). The rate of lime loss was measured to estimate the dam service life.

2.5—Construction-related considerations

Construction plans and specifications affect fabrication of reinforced concrete structures, which in turn affects service-life performance. They establish a basic performance level for the structure. Durability criteria, crack widths, concrete cover, and stress levels are established during the design phase and are reflected in the plans and specifications. Also, the construction standards and approval requirements are defined.

The ways and means of construction are the contractor's responsibility. Most often, the construction methods employed meet both the intent and the details of the plans and specifications. In some instances, however, the intent of the plans and specifications are not met, either through misunderstanding, error, neglect, or intentional misrepresentation. With the exception of intentional misrepresentation, each of these conditions can be discussed through an examination of the construction process. Service-life impairment can result during any of the four stages of construction: material procurement and qualification, initial fabrication, finishing and curing, and sequential construction. With the exception of material procurement and qualification, addressed under Section 2.3, each stage and the corresponding service life impacts are discussed as follows.

2.5.1 *Initial fabrication*—Initial fabrication is defined as all the construction up to and including placement of the concrete. This work incorporates soil/subgrade preparation and form placement; reinforcement placement; and concrete material procurement, batching, mixing, delivery, and placement.

2.5.1.1 *Soil/subgrade preparation and form placement*—Improper soil/subgrade preparation can lead to excessive or differential settlement. This can result in misalignment of components or concrete cracking. Initial preparation and placement of the formwork not only establishes the gross dimensions of the structure but also influences certain details of reinforcement and structure performance. Examples of the impact of these factors on service-life performance are summarized as follows.

Condition	Potential service-life impact
Improper soil/subgrade propagation	Structural damage such as cracking, component movement or misalignment.
Formwork too wide	Excess concrete weight, potential long-term deflection, or excessive cracking.
Formwork too narrow or shallow	Decreases structural capacity, excess deflections, or cracking.
Formwork too deep	Probably none, if structural depth increases then excess weight can be compensated by excess strength, otherwise same as too wide.
Formwork not in alignment	Excess waviness can encroach on cover, reducing bond and increasing potential for corrosion.

2.5.1.2 *Steel reinforcement placement*—Tolerances for reinforcement placement are given in ACI 318 and ACI SP-66. These documents are referenced in project specifications. Deviation from these standards can result in service-life complications such as those listed as follows.

Conditions	Potential service-life impact
Reinforcement out of specification	Cracking due to inability to support design loads.
Deficient cover	Accelerated corrosion potential, possible bond failure, reduced fire resistance.
Excessive cover	Potential reduction in capacity, increased deflection, increased crack width at surface, decreased corrosion risk.
Insufficient bar spacing	Inability to properly place concrete, leading to reduced bond, voids, increased deflection and cracking, increased corrosion risk.
Improper tendon duct placement	Improper strains due to prestress deviations.
Contaminated grout or improper use of corrosion inhibitor	Prestressing system degradation.

2.5.1.3 *Concrete batching, mixing, and delivery*—Concrete can be batched either on the project site or at a remote batch plant and transported to the site. Activities influencing the service-life performance include batching errors, improper equipment operation, or improper preparation.

Many concrete batch operations incorporate computer-controlled weight and batching equipment. Sources of error are lack of equipment calibration or incorrect mixture selection. Routine maintenance and calibration of the equipment ensures proper batching. Because plants typically have tens to hundreds of mixture proportions, batching the wrong mixture is a possibility. Errors, such as omission of air-entraining admixture, inclusion of excessive water, or low cement content, are likely to have the greatest impact on service life.

Equipment preparation is the source of more subtle effects. For example, wash water retained in the drum of a transit mix truck mixes with newly batched concrete to result in a higher *w/cm* than specified. This effect is cumulatively deleterious to service life through lower strength, increased shrinkage cracking, or higher permeability.

Ambient temperature, transit time, and admixture control are some of the factors controlling the mixture quality in the delivery process. ACI 305 and ACI 306 specify proper procedures to ensure concrete quality. Workability at the time of delivery, as measured by the slump, is also a long-term service life issue. Low slump is often increased by adding water at the site. If the total water does not exceed that specified, concrete integrity and service life will not be reduced. If the additional water increases the total available water above that specified, then the increased *w/cm* can compromise the service life.

2.5.1.4 *Concrete placement*—Proper placement, including consolidation and screeding, is important to the service life of concrete structures. Lack of proper consolidation leads to such things as low strength, increased permeability, loss of bond, and loss of shear or flexural capacity. These in turn diminish service life by accelerating the response to corrosive environments, increasing deflections, or contributing to premature failures.

2.5.2 *Finishing and curing*—Improper finishing or curing leads to premature deterioration of the concrete and reduction of service life (for example, production of a porous and abrasive cover concrete). The following summarizes common service-life issues affecting slabs and other structures:

Conditions	Potential service-life impact
Adding water during finish or reworking bleed water into surface	Dusting, scaling, blistering, or premature loss of surface, and loss of surface hardness.
Lack of proper curing	Excessive shrinkage, lower strength, cracking, or curling.
Use of calcium chloride	Degradation of embedded reinforcing steel.

A standard for curing concrete that maintains the original service-life design intent has been prepared (ACI 308R).

2.5.3 *Sequential construction*—Reinforced concrete structures are seldom completed in a single construction activity. Complementary or sequential construction can adversely affect the service life of the structure if not properly accomplished. The following two examples illustrate how this service-life impairment can occur.

2.5.3.1 *Shoring and reshoring*—In multiple-story buildings, shoring is used to support the formwork for placing concrete on the next floor. The normal practice is to remove the shoring when the form is removed and then to reshore until the concrete has gained sufficient strength to carry the construction loads. Premature form removal leads to cracking of the affected component. The cracking reduces the stiffness of the slab, increases the initial deflections and the subsequent creep deflections. Even when the concrete eventually gains its full strength, the cracked member has greater deflection than a comparable uncracked member, and can be more vulnerable to ingress of hostile environments.

2.5.3.2 *Joints*—Joints are placed in buildings and bridges to accommodate contraction and expansion of the structure due to creep, shrinkage, and temperature. Improperly designed or installed joints can lead to excessive cracking, joint failure, moisture penetration into the structure, and maintenance problems. Water passage through faulty bridge joints can result in bearings seizing up, localized bearing failures, cracking, crushing of seal materials, accelerated deterioration of the superstructure and substructure components, and unsightly staining of the substructure.

CHAPTER 3—IN-SERVICE INSPECTION, CONDITION ASSESSMENT, AND REMAINING SERVICE LIFE

3.1—Introduction

Detection and assessment of the magnitude and rate of occurrence of environmental factor-related degradation are key factors in predicting service life and in maintaining the capability of reinforced concrete structures to meet their operational requirements. It is desirable to have an evaluation methodology that, given the required data, provides the procedures for performing both a current condition assessment and certifying future performance. Such a methodology would integrate service history, material and geometry characteristics, current damage, structural analyses, and a comprehensive degradation model. For completeness, the methodology should also include the capability to evaluate the role of maintenance in extending usable life or structural reliability. Figure 3.1 presents a flow diagram of a methodology proposed as a guide in assessments of safety-related concrete structures in nuclear power plants (Naus et al. 1994). The diagram is an adaptation of a procedure proposed to evaluate the structural condition of buildings (Rewerts 1985). This chapter provides information to rate the current condition and assess remaining service life.

3.2—Evaluation of reinforced concrete aging or degradation effects

Performance of a structure is measured by the physical condition and functioning of component structural materials. Tests are conducted on reinforced concrete to assess performance of the structure as a result of (Murphy 1984):

- Noncompliance of properties with specifications;
- Inadequacies in placing, compacting, or curing of concrete;
- Damage resulting from overload, fatigue, freezing and thawing, abrasion, chemical attack, fire, explosion, or other environmental factors; or
- Concern about the capacity of the structure.

Testing is also undertaken for the verification of models, materials, and environmental parameters used for calculating the service life in the design phase. The validated or improved models are then used for optimization of the building operation and maintenance.

Prediction of the remaining service life of a concrete structure requires the accumulation of data such as depicted in Table 3.1. Verification that the structural condition is as depicted in the construction documents, such as drawings, determination of physical condition, quantification of applied loads, and examination of any degradation are important. The questions faced in predicting service life are: establishing how much data should be accumulated, the desired accuracy of the predictions, available budgets for the predictive effort, as well as subsequent levels of inspection, maintenance, and repair.

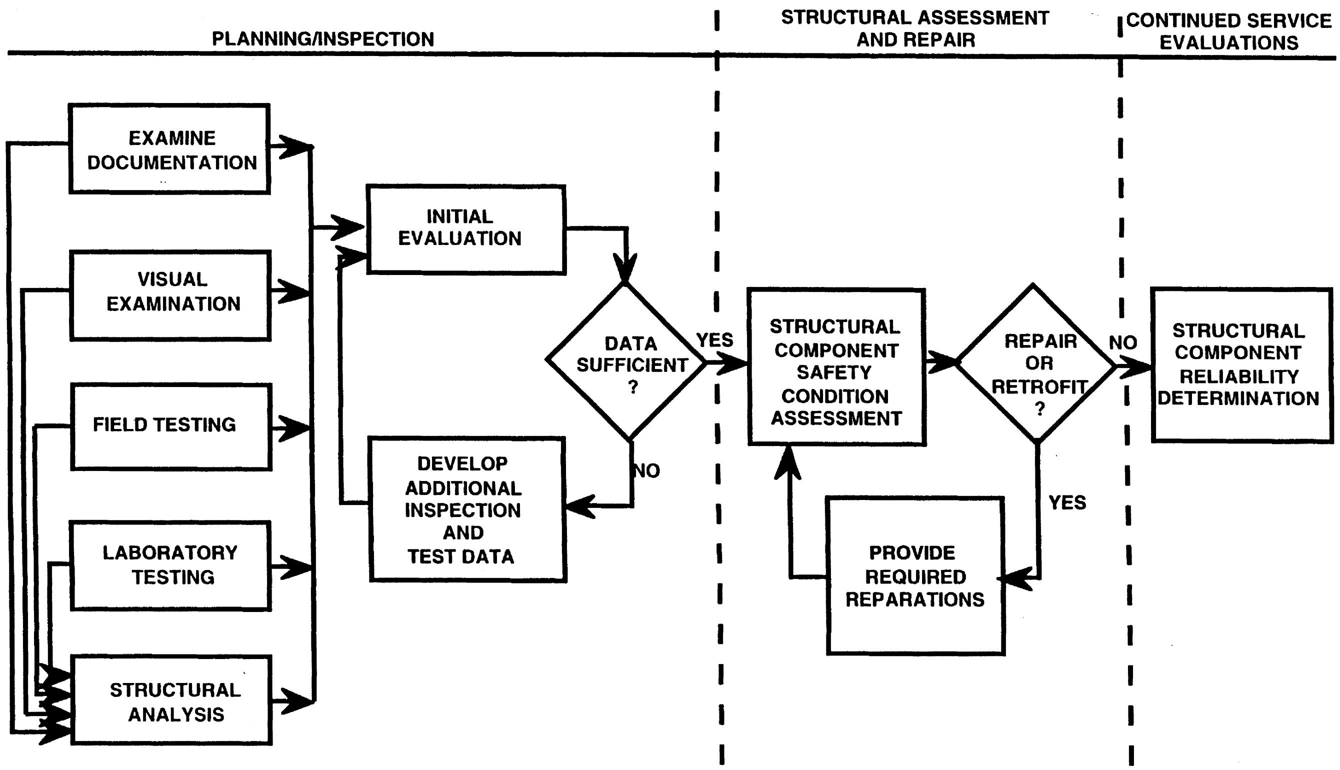

Fig. 3.1— Concrete component evaluation methodology. Source: Adaptation of a procedure presented in Rewerts 1985.

Table 3.1—Example of types of information needed for service-life assessment*

Conformance of structure to original design
Documentation review Preliminary site visit • Visual inspection for compliance with construction documents • Pachometer (covermeter) survey to locate and characterize steel reinforcement (for example, size and spacing) Preliminary analysis
Inspection for presence of degradation
Visual inspection Crack survey Delamination/spall survey Chloride survey Carbonation survey Sample removal
Laboratory testing
Petrographic studies (for example, air content, air-void distribution, unstable aggregates, types of distress, and estimation of *w/cm*) Chemical studies (for example, chemical constituents of cementitious materials, pH, presence of chemical admixtures, and characteristics of paste and aggregates) Concrete and steel reinforcement material properties (for example, strength and modulus of elasticity)
Degradation assessment
Current-versus-specified material properties Concrete absorption and permeability (relative) Concrete cover (for example, cores, or pachometer or covermeter measurements) Presence of excessive concrete crack widths, spalling, or delaminations Depth of chloride penetration and carbonation Steel reinforcement corrosion activity (for example, half-cell potential measurements, and galvanostatic pulse, four-electrode, and corrosion probes Environmental aggressivity (for example, presence of moisture, chlorides, and sulfates)
Structural reanalyses for current conditions
Reanalyses for typical dead and live loads Examination of demands from other loads (for example, seismic and wind)

*This list is not all inclusive.

Chapter 2 indicates that the ability of a reinforced concrete structure to meet its functional and performance requirements over an extended period of time is largely dependent on the durability of its components. Techniques for the detection of concrete component degradation should address the concrete, steel reinforcement, and anchorage embedments.

3.2.1 *Concrete material systems*—Primary manifestations of distress that can occur in reinforced concrete structures include cracking and delaminations (surface parallel cracking), excessive deflections, and mechanical property (strength) losses. Whether the concrete was batched using the proper constituents and mixture proportioning, or was properly

Table 3.2—Nondestructive test methods for determining material properties of hardened concrete in existing construction (ACI 228.2)

Property	Possible methods: Primary	Possible methods: Secondary	Comment
Compressive strength	Cores for compression testing (ASTM C 42 and C 39)	Penetration resistance (ASTM C 803; pullout testing drilled in)	Strength of in-place concrete; comparison of strength in different locations; and drilled-in pullout test not standardized
Relative compressive strength	Rebound number (ASTM C 805); ultrasonic pulse velocity (ASTM C 597)	—	Rebound number influenced by near surface properties; ultrasonic pulse velocity gives average result through thickness
Tensile strength	Splitting-tensile strength of core (ASTM C 496)	In-place pulloff test (ACI 503R; BS 1881; Part 207)	Assess tensile strength of concrete
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 ASTM 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)	Establishes relative susceptibility of concrete to chloride ion intrusion; assess effectiveness of chemical sealers, membranes, and overlays
Air content; cement content; and aggregate properties (scaling, alkali-aggregate reactivity, freezing-and-thawing susceptibility	Petrographic examination of concrete samples removed from structure (ASTM C 856, ASTM C 457); Cement content (ASTM C 1084)	Petrographic examination of aggregates (ASTM C 294, ASTM C 295)	Assist in determination of cause(s) of distress; degree of damage; quality of concrete when originally cast and current
Alkali-silica reactivity	Cornell/SHRP rapid test (SHRP-C-315)	—	Establish in field if observed deterioration is due to alkali-silica reactivity
Carbonation, pH	Phenolphthalein (qualitative indication); pH meter	Other pH indicators (for example, 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; ultrasonic pulse velocity; impact-echo; impulse-response	Rebound number permits demarcation of damaged concrete
Freezing-and-thawing damage	Petrography	SASW; impulse response	—
Chloride ion content	Acid-soluble (ASTM C 1152) and water-soluble (ASTM C 1218)	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 near surface concrete (15 mm)
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

placed, compacted, and cured are important because they can affect the service life of the structure. Measurement of these factors should be part of the overall evaluation process. In-place permeability tests can also be conducted on concrete to locate areas that are more susceptible to degradation.

3.2.1.1 *Nondestructive test methods*—Nondestructive test methods are used to determine hardened-concrete properties and to evaluate the condition of concrete in structures. Table 3.2 and 3.3 present nondestructive test methods for determining material properties of hardened concrete in existing construction and to determine structural properties and assess conditions of concrete, respectively (ACI 228.2R). A description of the method and principle of operation, as well as applications, for the most commonly used nondestructive test methods is provided elsewhere (ACI 228.1R, ACI 228.2R, Bungey 1996, Malhotra 1984, Malhotra and Carino 1991).

3.2.1.2 *Destructive test methods*—Visual and nondestructive testing methods are effective in identifying areas of concrete exhibiting distress but often cannot quantify the extent or nature of the distress. This is generally accomplished through removal of cores or other samples using a procedure such as provided in ASTM C 42.

When core samples are removed from areas exhibiting distress, a great deal can be learned about the cause and extent of deterioration through strength (Hindo and Bergstrom 1985) and petrographic studies (ASTM C 856). Additional uses of concrete core samples include calibration of nondestructive testing devices, conduct of chemical analyses, visual examinations, determination of steel reinforcement corrosion, and detection of the presence of voids or cracks (Munday and Dhir 1984, Bungey 1979).

3.2.1.3 *Mixture composition*—The question of whether the concrete in a structure was cast using the specified mixture composition can be answered through examination of core samples (Mather 1985). By using a point count method (ASTM C 457), the nature of the air void system (volume and spacing) can be determined by examining a polished section of the concrete under a microscope. An indication of the

Table 3.3—Nondestructive test methods to determine structural properties and assess conditions of concrete (ACI 228.2)

Property	Methods: Primary	Methods: Secondary	Comment
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); 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 corrosion
Corrosion rate	Linear polarization (SHRP-S-324 and S-330)	—	Corrosion rate of embedded steel; rate influenced by environmental conditions
Locations 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

type and relative amounts of fine and coarse aggregate, as well as the amount of cementitious matrix and cement content, can also be determined (ASTM C 856; ASTM C 85). Determination of the original *w/cm* is not covered by a standard test procedure, but the original water (volume of capillary pores originally filled with capillary and combined water) can be estimated (BS 1881, Part 6). Thin-section analysis can also indicate the type of cementitious material and the degree of hydration, as well as type and extent of degradation. A standard method also does not exist for determination of either the type or amount of chemical admixtures used in the original mixture. Determination of mixture composition becomes increasingly difficult as a structure ages, particularly if it has been subjected to leaching, chemical attack, or carbonation.

3.2.2 *Steel reinforcing material systems*—Assessments of the steel reinforcing system are primarily related to determining its presence and size, and evaluating the occurrence of corrosion. Determination of material properties such as tensile and yield strengths, and modulus of elasticity, involves the removal and testing of representative samples. Pertinent nondestructive test methods that address the steel reinforcing material system are provided in Table 3.2 and 3.3. ACI 222R provides detailed information on the mechanism of corrosion of steel in concrete and procedures for identifying the corrosion environment and active corrosion in reinforced concrete.

3.2.3 *Anchorage embedments*—Failure of anchorage embedments in concrete structures occurs as a result of either improper installation, cyclic loading, or deterioration of the concrete. Visual inspections can evaluate the general condition of the concrete near an embedment and provide a cursory examination of the anchor to check for improper embedment, weld or plate tearing, plate rotation, or plate buckling. Mechanical tests can verify that pullout and torque levels of embedments meet or exceed values required by design. Welds or other metallic components can be inspected using magnetic-particle or liquid-penetrant techniques for surface examinations, or if a volumetric examination is required, radiographic, ultrasonic, and eddy current techniques are available. ACI 355.1R, ACI SP-103, and ACI SP-130 provide additional information on anchorage to concrete.

3.3—Condition, structural, and service-life assessments

3.3.1 *Current condition*—Determining the existing performance characteristics and extent and causes of any observed distress is accomplished through a condition assessment by personnel having broad knowledge in structural engineering, concrete materials, and construction practices. Several documents are available to aid in conducting a condition assessment of reinforced concrete structures and components (ACI 201.1R; ACI 224.1R; ACI 437R; ACI 207.3R; ACI 311.4R; ACI 362R; ASTM C 823; Bresler 1977; Perenchio 1989; ASCE 11-90; Kaminetzky 1977). The condition assessment commonly uses a field survey involving visual examination and application of nondestructive and destructive testing techniques, followed by laboratory and office studies. Guidelines for conduct of surveys of existing buildings have been prepared (Perenchio 1989; ASCE 11-90). Before conducting a condition assessment, a definitive plan should be developed to optimize the information obtained. The condition assessment begins with a review of the as-built drawings and other information pertaining to the original design and construction so that information, such as accessibility and the position of embedded-steel reinforcement and plates in the concrete, are known before the site visit. Next, a detailed visual examination of the structure is conducted to document information that could result from or lead to structural distress, such as cracking, spalling, leakage, and construction defects, such as honeycombing and cold joints, in the concrete. Photographs or video recordings made during the visual examination can provide a permanent record of this information. Assistance in identifying various forms of degradation has been prepared (ACI 201.1R). After the visual survey has been completed, the need for additional surveys, such as delamination plane, corrosion, or pachometer is determined. Results of these surveys are used to select portions of the structure to be

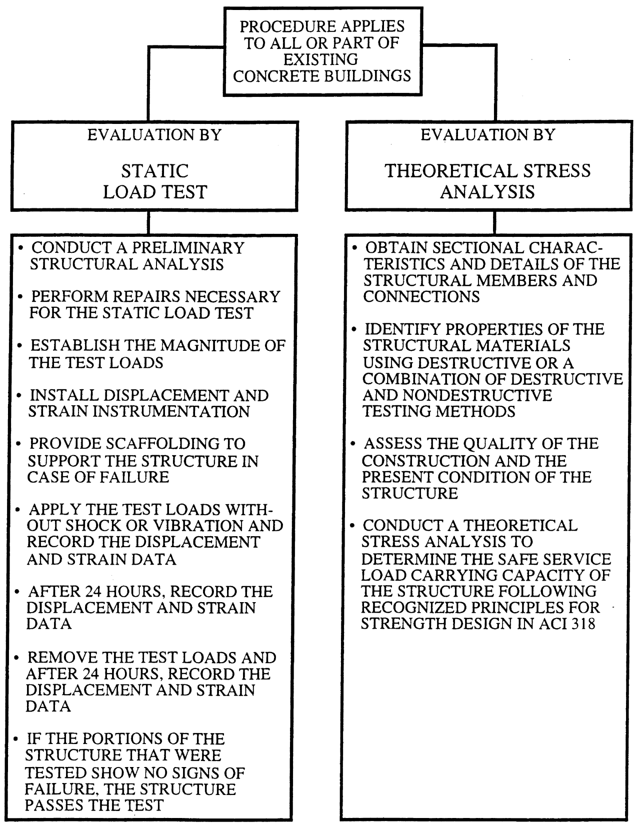

Fig. 3.2—Recommended procedure for strength evaluation of existing concrete buildings (ACI 437).

studied in greater detail. Many of the investigation techniques have been identified in the previous section. Any elements that appear to be structurally marginal, due to either unconservative design or effects of degradation, are identified and appropriate calculation checks made (refer to Section 3.3.2). A report is prepared after the field and laboratory results have been collated and studied and calculations completed.

3.3.2 *Structural assessment*—Once the critical structural components have been identified through the condition assessment, a structural assessment can be required to determine the current condition, to form the basis for estimating future performance or service life, or both. As part of the assessment it is important to note irregularities or inconsistencies in properties of materials, in design, in construction and maintenance practices, and the presence and effects of environmental factors. Although the assessment of a structure involves more than its load-carrying ability (for example, the permeability of hydraulic structures), an assessment of structural demand versus capacity is the first step. Performance requirements other than structural capacity are then addressed through supplementary tests to establish characteristics, such as leakage rate or permeability.

Procedures to evaluate the strength of existing structures have been published (ACI 437R). The recommendations developed are intended to establish the loads that can be sustained safely and serviceably by an existing building under several conditions:

- There is evidence of possible structural weakness (for example, excessive cracking or spalling);
- The building or a portion of it has undergone general or local damage (for example, environmental or earthquake effects);
- There is doubt concerning the structure's capacity; and
- Portions of a building are suspected to be deficient in design, detail, material, or construction.

Methods for strength evaluation of existing concrete structures include either an analytical assessment or a load test (Fig. 3.2).

An analytical assessment is recommended when sufficient background information is not available (for example, sectional characteristics, material properties, and construction quality), a static load test is impractical because of the test complexity or magnitude of the load required, sudden failure during a static load test can endanger the integrity of the member or the entire structure, or it is required by an authority. Some supplemental destructive or nondestructive tests described previously can be required to obtain this information. For the evaluation it is recommended that the theoretical analyses follow principles of strength design and that a structure be considered satisfactory if capacity, deformation, and other serviceability criteria satisfy the requirements and intent of the ACI 318.

Static-load tests should be utilized only when the analytical method is impractical or otherwise unsatisfactory. Situations where a static load test of a bridge or building component is recommended include those where at least one of the following cases and all of the following conditions apply (ACI 437R). Cases include incidences where structural element details are not readily available; deficiencies in details, materials, or construction are best evaluated by a load test; and the design is extremely complex with limited prior experience for a structure of this type. Conditions include: 1) results of a static load test permit a reasonable interpretation of structural adequacy; 2) principal structural elements under investigation are primarily flexural members; and 3) adjacent structure's effects can be accounted for in the evaluation of the load test results. Before conduct of a load test, some repair actions can be required and an approximate analysis should be conducted. After establishing the magnitude of the test load, the load is applied incrementally with deflections measured. The structure is considered to have passed the load test if it shows no visible evidence of failure, such as excessive cracking or spalling, and it meets requirements for deflection. In certain applications, serviceability requirements, such as allowable leakage at maximum load, can also be a criterion.

3.3.3 *Service-life assessments*—Any viable design method or assessment of service life involves a number of essential elements: a behavioral model, acceptance criteria defining satisfactory performance, loads under which these criteria should be satisfied, relevant characteristic material properties, and factors or margins of safety that take into account uncertainties in the overall system (Sommerville 1992). The selection of materials and mixture proportions, such as the maximum *w/cm*, and structural detail considerations, provides one approach used for design of durable structures. Another approach entails prediction of service life using calculations based on knowledge about the current damage,

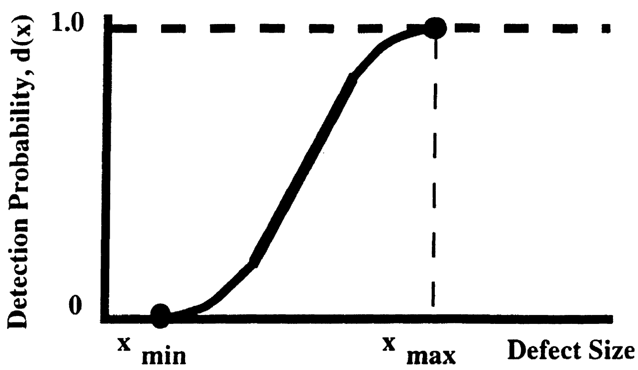

Fig. 3.3—Defect detectability function (Ellington and Mori 1992).

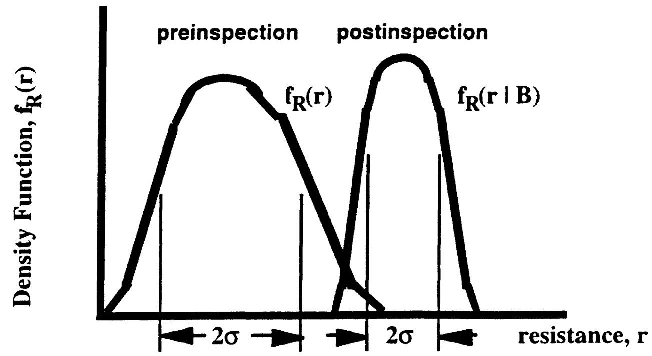

Fig. 3.4—Role of in-service inspection on strength distribution (Ellingwood and Mori 1992).

degradation mechanisms, and the rates of degradation reactions. Development of a more comprehensive approach for design of durable structures requires integration of results obtained from a large number of studies that have been conducted relative to concrete durability.

3.4—Inspection and maintenance

In-service inspection and preventive maintenance are a routine part of managing aging and degradation in many engineered facilities (House 1987). The structural integrity of civil structures, such as bridges and offshore platforms exposed to extreme climatic conditions, are routinely assessed. These assessments record performance and estimate the structure's ability to continue to meet functional and performance requirements. Also, in-service inspection and maintenance strategies can be used to predict reliability and usable life of structures.

One approach to predicting the structure's reliability or its service life under future operating conditions is through probability-based techniques involving time-dependent reliability analyses. These techniques integrate information on design requirements, material and structural degradation, damage accumulation, environmental factors, and nondestructive evaluation technology into a decision tool that provides a quantitative measure of structural reliability. The technique can also investigate the role of in-service inspection and maintenance strategies in enhancing reliability and extending usable life. In-service inspection methods can impact the structural reliability assessment in two areas, detection of defects and modifications to the frequency distribution of resistance. Several nondestructive test methods that detect the presence of a defect in a structure tend to be qualitative in nature in that they indicate the presence of a defect but may not provide quantitative data about the defect's size, precise location, and other characteristics that would be needed to determine its impact on structural performance. None of these methods can detect a given defect with certainty. The imperfect nature of these methods can be described in statistical terms. This randomness affects the calculated reliability of a component. Figure 3.3 illustrates the probability, $d(x)$, of detecting a defect of size x. Such a statistical relation exists, at least conceptually, for each of the applicable in-service inspection methods. In-service inspection methods also provide information that allow the probabilistic strength models used in reliability analyses to be revised (Viola 1983, Turkstra et al. 1988, Ciampoli 1989, Bartlett and Sexsmith 1991). The effect of in-service inspection on the distribution of resistance is illustrated in Fig. 3.4. The frequency distribution of resistance, based on prior knowledge of the materials used to fabricate the structure, construction, and standard methods of analysis, is indicated by the curve $f_R(r)$ in the figure. Scheduled maintenance and repair can cause the characteristics of the resistance to change. The effect of inspection and maintenance is illustrated by the (conditional) density $f_R(r|B)$, in which B is dependent on what is learned from the in-service inspection. The in-service inspection probably causes the mean value of the resistance distribution to increase because of basic conservatism in structural design. Quantitative data on the capabilities of in-service inspection methods are required for determining the appropriate modifications to the frequency distribution, $f_R(r)$, and to take optimum advantage of in-service inspection in the reliability analysis.

Once it has been established that a component has been subjected to environmental factors that have resulted in deterioration, the effects of these factors can be related to a condition or structural reliability assessment. Structural loads, engineering material properties, and strength-degradation mechanisms are random. The resistance, $R(t)$, of a structure and the applied loads, $S(t)$, both are stochastic functions of time. At any time, t, the margin of safety, $M(t)$, is

$$M(t) = R(t) - S(t) \qquad (3\text{-}1)$$

Making the customary assumption that R and S are statistically independent random variables, the probability of failure, $P_f(t)$, is

$$P_f(t) = P[M(t) < 0] = \int_0^{\infty} F_R(x) f_s(x) dx \qquad (3\text{-}2)$$

in which $F_R(x)$ and $f_S(x)$ are the probability distribution function of R and density function of S. Equation (3-2) provides one quantitative measure of structural reliability and performance, provided that P_f can be estimated and validated.

For service-life prediction and reliability assessment, the probability of nonfailure over some period of time, $(0,t)$, is more important than the reliability of the structure at the particular time provided by Eq. (3-2). The probability that a structure survives during interval of time (0,t), is defined by a reliability function, $L(0,t)$. If n discrete loads $S_1, S_2,..., S_n$ occur at times $t_1, t_2,...,t_n$ during (0,t), the reliability function becomes

$$L(0, t) = P[R(t_1) > S_1, \ldots, R(t_n) > S_n] \qquad (3\text{-}3)$$

If the load process is continuous rather than discrete, this expression is more complex.

The conditional probability of failure within time interval (t,t+Δt), given that the component has survived during (0,t), is defined by the hazard function

$$h(t) = -d(\ln L(0, t))/dt \qquad (3\text{-}4)$$

which is especially useful for analyzing structural failures due to aging or deterioration. For example, the probability that time to structural failure, T_f, occurs before a future maintenance operation at $t+\Delta t$, given that the structure has survived to t, can be evaluated as

$$P[T_f \le t + \Delta t | T_f > t) = 1 - \exp\left[-\int_t^{t+\Delta t} h(x)dx\right] \qquad (3\text{-}5)$$

The hazard function for pure-chance failures is constant. When structural aging occurs and strength deteriorates, h(t) charateristically increases with time as illustrated in Fig. 3.5.

Intervals of inspection and maintenance required as a condition for continuing the service of a structure also can be determined from the time-dependent reliability analysis. The updated density of R following each inspection is

$$f_R(r|B) = P[r < R \le r + dr, B]/P[B] = cKf_R(r) \qquad (3\text{-}6)$$

where $K(r)$ is denoted the likelihood function and c is a normalizing constant. The time-dependent reliability analysis then is reinitialized using the updated $f_R(r|B)$ in place of $f_R(r)$. The update causes the hazard function to be discontinuous in time and lowers the failure probability in Eq. (3-5). The effect of in-service inspection or repair on the hazard function is also illustrated in Fig. 3.5.

Uncertainties in methods of in-service inspection or repair affect the density $f_R(r|B)$. A combination of methods is usually more effective from a reliability point of view than using one method. When there are limited resources, it is most effective to select a few safety-critical elements and concentrate on them (Hookham 1991, Ellingwood and Mori 1993). Optimal intervals of inspection and repair for maintaining a desired level of reliability can be determined based on expected life-cycle cost. Preliminary investigations have found that life-cycle costs are sensitive to relative costs of inspection, maintenance, and failure. If the cost of failure is an order of magnitude larger than inspection and maintenance costs, the optimal policy is to inspect at nearly uniform intervals of time. Additional information on applying the methodology to investigate inspection or repair strategies for

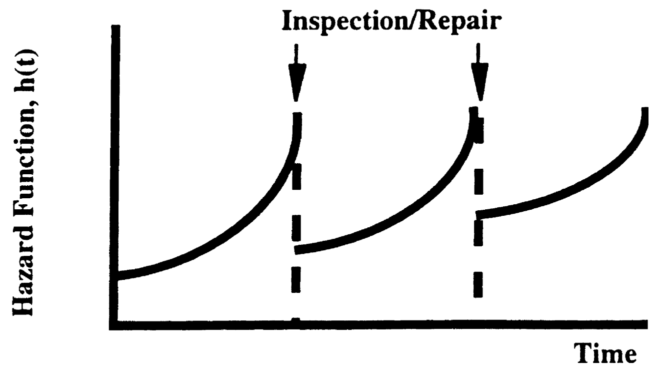

Fig. 3.5—Role of in-service inspection/repair in controlling hazard function (Ellingwood and Mori 1992).

reinforced concrete elements in flexure and shear has been reported (Mori and Ellingwood 1993, 1994b).

CHAPTER 4—METHODS FOR PREDICTING THE SERVICE LIFE OF CONCRETE

4.1—Introduction

The selection of concrete materials and mixture proportions is usually based on empirical relationships between concrete mixtures and laboratory and field performance. This approach assumes that the concrete selected supports the desired service life for the structure.

Another approach for selecting concrete involves predicting service life using calculations based on likely degradation mechanisms that manifest in the structure and the reaction rates of these mechanisms. While this approach is not often used, it can have an increasingly important role in selecting concrete because of applications that require significantly increased service lives, increased use of concrete in harsh environments, the high cost of rebuilding and maintaining the infrastructure, and the development of high-performance concretes for which a record of long-term performance is, as yet, not available. In addition, improved understanding of the factors controlling the service life of concrete contribute to the development of more durable concretes.

Many service-life prediction methods focus on the effect of one degradation process. Experience, however, has shown that degradation results when one or more degradation processes are operative or from the interaction of the environment and loads (Hookham 1990). This synergistic effect complicates service-life prediction for both new concrete structures where environmental factors and loads may have not been well defined, and existing structures where the contribution to degradation by various influences is difficult to assess. Primary factors that can limit the service life of reinforced concrete structures include the presence of chlorides, carbonation, aggressive chemicals, such as acids and sulfates, freezing-and-thawing cycling, and mechanical loads, such as fatigue, vibration, and local overloads. Typically, only one primary factor limits the service life and is the focus of service-life prediction. As limited information is available on the synergistic effect when more than one factor is operative, this chapter focuses on the prominent environmental influences noted previously. An overview of methods for predicting the service life of new and existing concrete along with some examples of their applications are presented. Examples illustrating the use of several of the service-life methods and models are provided in Chapter 6.

4.2—Approaches for predicting service life of new concrete

Methods that have been used for predicting the service lives of construction materials include estimates based on experience, deductions from performance of similar materials, accelerated testing results, mathematical modeling based on the chemistry and physics of expected degradation processes, and applications of reliability and stochastic concepts (Clifton and Knab 1989). Although these approaches are discussed separately, they often are used in combination.

4.2.1 *Predictions based on experience*—Semiquantitative predictions of the service life of concrete are based on the accumulated knowledge from laboratory and field testing and experience. This contains both empirical knowledge and heuristics; collectively, these provide the largest contribution to the basis for standards for concrete. It is assumed that if concrete is made following standard industry guidelines and practices, it will have the required life. This approach gives an assumed service-life prediction. The concrete can perform adequately for its design life, especially if the design life is fairly short and the service conditions are not too severe. This approach breaks down when it becomes necessary to predict the service life of concrete that is required to be durable for a time that exceeds our experience with concrete, when new or aggressive environments are encountered, or when new concrete materials are to be used. Several examples have been analyzed using this approach with the conclusion that experience or qualitative assessments of durability do not form a reliable basis for service-life predictions and are only estimates (Fagerlund 1985).

4.2.2 *Predictions based on comparison of performance*—The comparative approach has not been commonly used for concrete, but with a growing population of aging concrete structures its use will increase. In this approach, it is assumed that if concrete has been durable for a certain time, a similar concrete exposed to a similar environment has the same life. A problem with this approach is each concrete structure has a certain uniqueness because of the variability in materials, geometry, construction practices, and exposure to loads and environments. Also, over the years, the properties of concrete materials have changed. For example, portland cements are ground finer today than they were 40 years ago to achieve increased early-age strength. This results in concrete with lower density and higher permeability (Neville 1987). Another problem with the comparison approach is the difference in the microclimates (environment at concrete surface) can have unanticipated effects on the concrete's durability. In contrast, advances in chemical and mineral admixtures have led to the development and use of concrete with improved performance and durability. Therefore, comparing the durability of old and new concrete is not straightforward, even when conditions are as similar as possible.

4.2.3 *Accelerated testing*

4.2.3.1 *Approach*—Most durability tests for concrete use elevated loads or more severe environments, such as a higher concentration of reactants, temperature, and humidity, to accelerate degradation. Accelerated testing programs, if properly designed, performed, and interpreted, can help predict the performance and service life of concrete. Accelerated testing has been proposed as a method for predicting the service life of several types of building materials (Frohnsdorff et al. 1980). The degradation mechanism in the accelerated test should be the same as that responsible for the in-service deterioration. If the degradation proceeds at a proportional rate by the same mechanism in both accelerated aging and long-term in-service tests, an acceleration factor, K, can be obtained, from

$$K = R_{AT}/R_{LT} \tag{4-1}$$

where R_{AT} is the rate of degradation in accelerated tests, and R_{LT} is the rate of degradation in long-term in-service testing. If the relationship between the rates is nonlinear, then mathematical modeling of the degradation mechanism is recommended to establish the relationship.

ASTM E 632 gives a recommended practice for developing accelerated short-term tests that can obtain data for making service predictions and for solving service-life models. The practice consists of four main parts: problem definition, pretesting, testing, and interpretation and reporting of data. Application of this practice to concrete has been discussed (Clifton and Knab 1989).

A difficulty in using accelerated testing in predicting service life is the lack of long-term data on the in-service performance of concrete as required in Eq. (4-1). Accelerated tests, however, can provide information on concrete degradation that is needed to solve mathematical models for predicting service lives.

4.2.3.2 *Application*—An example of the application of accelerated testing service-life predictions is provided below (Vesikari 1986). In this application, the lifetime of a specimen in an accelerated test t^* is related to the service life of a structure t_1 by

$$t_1 = kt^* \tag{4-2}$$

where k is a constant that is derived from testing. This approach is then applied to freezing-and-thawing resistance testing of concrete as follows. In an accelerated freezing-and-thawing test, the performance of a specimen is expressed in terms of the number of freezing-and-thawing cycles needed to obtain a specified damage level. Assuming the number of freezing-and-thawing cycles that a structure is subjected to annually is constant, the service life of the structure can be evaluated by

$$t_1 = k_e N \tag{4-3}$$

where

k_e = a coefficient related to environmental conditions; and

N = number of freezing-and-thawing cycles damaging a laboratory specimen.

This approach was further developed to predict the life of concrete that is exposed to the combined effect of freezing-and-thawing and salt-scaling action. In this case, the service life was given by

$$t_1 = k_f P \tag{4-4}$$

where P is the freezing-and-thawing resistance index and is obtained by the Deutscher Beton Verein (DBV) freeze-salt test (Vesikari 1986). Values of the environmental factor k_f are based on field investigations that analyze the correlation between the degree of damage of the structure, age of the structure, and the freezing-and-thawing resistance of the structure.

The following study illustrates the application of an accelerated test method to estimate the service life of concrete exposed to sulfate salts. The U.S. Bureau of Reclamation combined the results of accelerated tests and long-term tests (Kalousek et al. 1972). In the long-term tests, concrete specimens were continuously immersed in a 2.1% sodium sulfate (Na_2SO_4) solution until failure occurred, defined as an expansion of 0.5%, or until the investigation was completed. The age of specimens at the completion of the continuous-immersion study ranged from 18 to 24 years. Companion specimens were subjected to an accelerated test in which the specimens were exposed to repeated cycles of immersion in a 2.1% sodium sulfate (Na_2SO_4) solution for 16 h and forced air drying at 54 C for 8 h. Comparing the times for specimens to reach an expansion of 0.5% in the accelerated test and the continuous immersion test, it was estimated that one year of accelerated testing was equivalent to eight years of continuous immersion. In this case, Eq. (4-1) becomes

$$K = 8 = R_{AT} / R_{LT} \tag{4-5}$$

where

R_{AT} = rate of expansion in the accelerated test, and

R_{LT} = rate of expansion in the long-term continuous immersion test.

A 2.1% solution of sodium sulfate (Na_2SO_4) is a severe environment and if concrete is exposed to a lower concentration of sulfate, the life expectancy would be expected to be longer. This method can be used to predict the service life of concrete continuously immersed in a different concentration of sulfate ions, provided the acceleration factor is known.

4.2.4 *Mathematical models*—Mathematical models are no better than their underlying conceptual base, so any solution calculated using a model has uncertainties related to the model as well as the material and environmental parameters. Several models have been developed to predict the service life of concrete subjected to degradation processes such as corrosion, sulfate attack, leaching, and freezing-and-thawing damage (Clifton 1991). The use of mathematical models to predict service life of concrete has been discussed (Pommersheim and Clifton 1985). Models used to predict service life of concrete used in the construction of underground vaults for the disposal of low-level nuclear waste, which are subjected to sulfate attack, corrosion of reinforcement, leaching, and freezing-and-thawing attack, have been reviewed (Walton et al. 1990). Many of the degradation processes of concrete, excluding those caused by mechanical loads, are associated with the intrusion into concrete of one or more of the following: water, salts, or gases. For such processes, mathematical models that predict service life can be developed by considering the rate of intrusion of aggressive media into concrete and the rate of chemical reactions and physical processes. Mathematical models have been developed for degradation processes controlled by the intrusion of water, salts, and gases into concrete by convection and diffusion (Pommersheim and Clifton 1990). Most models that predict service life include numerical variables related to transport processes, such as the chloride ion diffusion coefficient in corrosion models. Standard methods have been developed for testing nonsteady state water flow in concrete (Kropp and Hilsdorf 1995). Furthermore, methods for testing ion diffusion, such as chlorides, are also available (Nord Test 1995). Reliable data on transport properties, however, often are not available and standard ASTM test methods have not been developed.

4.2.4.1 *Model of corrosion of reinforcing steel*—Most corrosion models for reinforced concrete follow the same approach, and are based on a model that has been developed to predict the service life of reinforcing steel (Tuutti 1982). The model is based on the corrosion sequence schematically shown in Fig. 4.1, in which active corrosion (propagation period) starts after the end of an initiation period of no corrosion. The corrosion process is initiated by the diffusion of chloride ions to the depth of the reinforcing steel or by carbonation reducing the pH of the concrete in contact with the steel or by the combination of chloride ions and carbonation. Other transport properties are not covered by the model. Sorption could be another important transport process that also follows a $t^{1/2}$ dependence, where t is time. Cracking of the concrete would increase the diffusion coefficient and sorptivity of the concrete, thus accelerating corrosion.

In the following, only the effect of chloride ions on the initiation period is considered. The length of the initiation period is largely controlled by the rate of diffusion of the chloride ions in the concrete and by the threshold concentration for the process. The one-dimensional diffusion process follows Fick's second law of diffusion (Tuutti 1982)

$$\partial c_f / \partial t = D \partial^2 c_f / \partial x^2 \tag{4-6}$$

where

D = diffusion coefficient;

x = distance from the concrete surface to the steel reinforcement; and

t = time.

Because chloride ions react with the tricalcium aluminate of portland cement, the concentration has two components — concentration of bound chloride ions (c_b) and concentration of free ions (cf), related through R

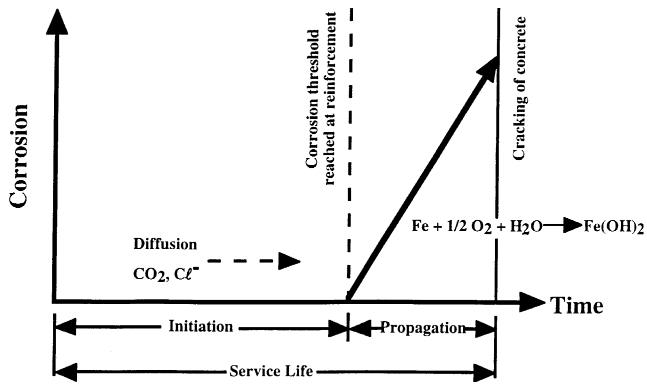

Fig. 4.1—Schematic of conceptual model of corrosion of steel reinforcement in concrete (Tuutti 1982).

$$c_b = R \cdot c_f \tag{4-7}$$

Because either carbonation or sulfate ions can release the bound chloride ions, R is usually assumed to be 0.

According to Tuutti's model, the corrosion rate in the propagation period is controlled by the rate of oxygen diffusion to the cathode, resistivity of the pore solution, and temperature. The initiation period is usually much longer than the propagation period. For example, in one bridge deck the initiation period has been estimated to be over five times longer than the propagation period (Tuutti 1982). A conservative estimate of the service life is usually made by only considering the initiation period. If the concrete is continuously saturated with water, the model predicts that corrosion processes active in the propagation period become the rate-controlling processes because of the extremely low diffusion rate of oxygen through the water. A conceptually similar but more complex model has been developed that predicts that reinforced concrete submerged in seawater can be unaffected by corrosion for thousands of years due to the absence or low level of oxygen present (Bažant 1979, 1979a).

The concepts of Tuutti's model have been used to predict the effects of the chloride-ion diffusion coefficient and the depth of cover on the length of the initiation period (Clifton et al. 1990). The period to initiate corrosion of a reinforced concrete element is determined as follows: C_0 is the concentration of chloride ions at the outside surface of the concrete, and C_i is the concentration at the depth of the reinforcement, that is assumed to be initially 0. The initiation period is completed when $C_i = C_t$, the threshold concentration to initiate steel reinforcement corrosion. The general solution to Eq. (4-6) for a reinforced concrete element under constant environmental conditions is

$$\frac{C}{C_0}(Z, t) = \tag{4-8}$$

$$\sum_{n=0}^{\infty} (-1^n)\left[erfc\left\{\frac{(2n+1)-y}{2\sqrt{r}}\right\} + erfc\left\{\frac{(2n+1)+y}{2\sqrt{r}}\right\}\right]$$

Table 4.1—Effect of cover and diffusion coefficient on time to initiation of corrosion of reinforced concrete

	Chloride ion diffusion coefficient D, m²/s*		
	5×10^{-11}	5×10^{-12}	5×10^{-13}
Cover, mm	Time, yr		
25	0.56	5.6	56
50	2.3	23.0	230
75	5.0	50.0	500
100	9.0	90.0	900

*Based on setting $C_t / C_0 = 0.55$, with $C_t = 0.4\%$ (by mass cement), and $L = 300$ mm.

where
erfc = complement of error function (Crank 1975);
y = $(L-x)/L$;
r = Dt/L^2;
t = time;
n = general solution, summation of all possible terms;
D = diffusion coefficient;
x = effective concrete cover depth (for example, uncracked thickness); and
L = thickness of concrete element.

In the present case, however, only the $n = 0$ term of Eq. (4-8) needs to be considered. Higher-order terms have insignificant contributions to the summation, reducing the equation to

$$\frac{C}{C_0} = erfc\frac{(1-y)}{2\sqrt{r}} \tag{4-9}$$

where $1 - y = x/L$. The model was solved for the case where the threshold concentration C_t of chloride ions was 0.4% (based on the mass of the cement), the concentration of chloride ions at the surface of the concrete C_o was 0.7% (based on the mass of cement), $x = 50$ mm, $L = 300$ mm, and $C_i = 0$ at $t = 0$. Results for different concrete cover depths and chloride ion diffusivity coefficients are presented in Table 4.1. The results show that the effect of the cover is proportional to x^2. For example, increasing x from 25 to 100 mm increases the service life by a factor of $(100/24)^2$ or 16. The model also predicts that a 10-fold decrease in the diffusion coefficient results in a 10-fold increase in the predicted service life. Although laboratory estimations of diffusion coefficients are too conservative for accurate estimates of the life of reinforced concrete, they do indicate the relative effects of important material and design variables on service lives.

Different solutions to Fick's second law have been developed to evaluate concrete under environmental conditions that vary with time (Amey et al. 1998). In such cases, the surface chloride concentration also changes with time (for example, by the application of chloride deicing salts). To obtain a relation that allows a surface build-up of chlorides, an equation other than Eq. (4-9) should be used due to the change in boundary conditions. Although there is no conclusive evidence for what function *Φ(t)* should be assigned to represent that build-up, there is some intuitive support for a linear or square root build-up of chloride over time. For the case where $\Phi(t) = kt$, where k is a constant under a linear build-up condition, the following simplified solution should be used

$$C(x,t) = \tag{4-10}$$

$$kt\left\{\left(1+\frac{x^2}{2Dt}\right)erfc\left(\frac{x}{2\sqrt{Dt}}\right)-\left(\frac{x}{\sqrt{\pi Dt}}\right)e^{-x^2/4Dt}\right\}$$

where $erfc$ ( ) = the complementary error function. For the case where $\Phi(t) = kt^{1/2}$, where k is a constant under a square root build-up condition, the following simplified solution should be used

$$C(x,t) = k\sqrt{t}\left\{e^{-x^2/4Dt}-\left(\frac{x\sqrt{\pi}}{2\sqrt{Dt}}erfc\left(\frac{x}{2\sqrt{Dt}}\right)\right)\right\} \tag{4-11}$$

Equations (4-10) and (4-11) are most suited for evaluating air-borne deicing salts applications. Additional information on models can be obtained from Vesikari (1988), who describes mechanistic models empirically fitted to data from field and laboratory studies, and HETEK (1996). Corrosion induced by chloride ions and by carbonation is addressed, and both the initiation and propagation periods are modeled. These models are useful in identifying the factors controlling the service life of reinforced concrete when corrosion is the major degradation process. They are solved using empirically derived coefficients for the quality of concrete, environments, and intensity of active corrosion. Effects of different types of cements, extent of carbonation, and compressive strength of concrete on corrosion are considered by the coefficient for the quality of concrete. The reliability of these models when projected to other concretes and environments needs to be determined before they are used.

Probabilistic models and computational methods for chloride ingress in concrete have also been developed (Engelund 1977).

4.2.4.2 *Sulfate attack*—A mechanistic model has been developed to predict the effect of ground water containing sulfates on the service life of concrete (Atkinson and Hearne 1990). The model is based on the following:

- Sulfate ions from the environment penetrate the concrete by diffusion;
- Sulfate ions react expansively with aluminates in the concrete; and
- Cracking and delamination of concrete surfaces result from the expansive reactions.

Cracking and delamination of the concrete surface exposes new surfaces to a concentration of sulfate ions similar to that of the ground water sulfate concentration rather than the lower concentration resulting from diffusion. The model indicates that the rate of sulfate attack is controlled by the concentration of sulfate ions and aluminates, diffusion and reaction rates, and the fracture energy of concrete. Relationships are developed for reaction kinetics, the concentration of reacted sulfate in the form of ettringite, the thickness of a spalled concrete layer, the time for a layer to spall, and the degradation rate. The depth of degradation (R) is linear in time, that is, m/sec, and is given by

$$R = X_{spall}/T_{spall} = (EB^2c_sC_0D_i)/[\alpha_0\tau(1-\nu)] \tag{4-12}$$

where

X_{spall} = the thickness of the reaction zone causing the spalling;
T_{spall} = the time for the spall to occur;
E = Young's modulus;
B = the linear strain caused by a concentration of sulfate reacted in a specific volume of concrete (such as 1 mole of sulfate reacted in 1 m^3 of concrete);
c_s = the sulfate concentration in bulk solution;
C_0 = the concentration of reacted sulfate in the form of ettringite;
D_i = the intrinsic diffusion coefficient of sulfate ions;
α_0 = roughness factor for fracture path;
τ = the fracture surface energy of concrete; and
ν = Poisson's ratio.

Some of the input data required to solve the model should be obtained from laboratory experiments, while some of the parametric values are not available for specific concretes and therefore typical values should be used. In the example calculation (Atkinson and Hearne 1990), the rate of attack for a sulfate-resistant portland cement (similar to ASTM C 150 Type V) was predicted to be only about 30% lower than that for ordinary portland cement (similar to ASTM Type I). The results agree with the generally accepted view that the permeability of the concrete (reflected in the sulfate diffusion coefficient) is more important in controlling sulfate attack than the chemical composition of the cement.

4.2.4.3 *Leaching*—A leaching model for the dissolution of gypsum and anhydrite (James and Lupton 1978) has been used to predict the rate of dissolution of portland-cement mortar exposed to flowing water (Jones 1989). It has the form

$$dM/dt = 2.6KA(C_s - C)^\theta \tag{4-13}$$

where

M = the mass lost in time t from an area A;
K = the experimentally obtained dissolution-rate constant (linearly dependent on the flow velocities within laminar flow regimes);
C_s = the solution potential of water;
C = the concentration of dissolved material at time t; and
θ = the kinetic order of the dissolution process.

The rate of dissolution of both silica and calcium from portland cement mortar was experimentally determined to give second-order kinetics. A loss of 0.8 mm/yr of mortar

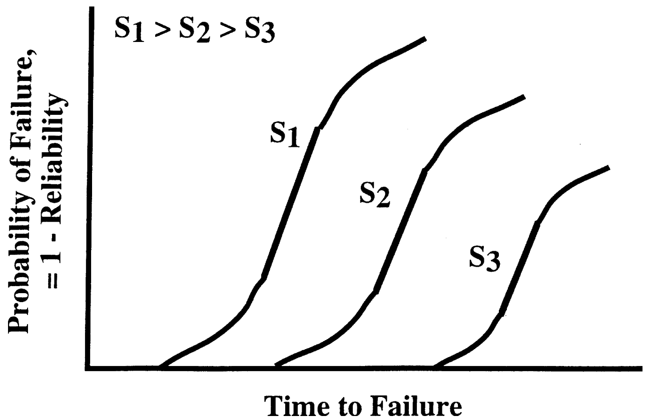

Fig. 4.2—Probability of failure at different stress levels (Martin 1985).

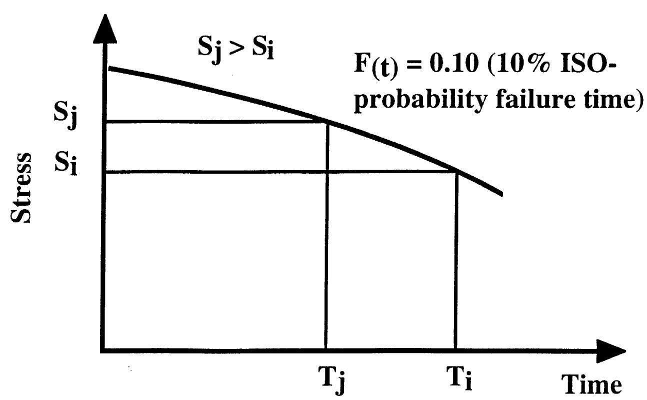

Fig. 4.3—Probability of failure stress-to-failure (P-S-T) diagram showing 10% probability of failure curve (Martin 1985).

was predicted at a flow velocity of 3 m/s, which is in reasonable agreement with the measured loss of 1 mm/yr at flow of 3 m/s.

4.2.5 *Stochastic methods*—The use of stochastic concepts in making service-life predictions of construction materials has been explored by several researchers (Sentler 1984; Martin 1985). Service-life models using stochastic methods are based on the premise that service life cannot be precisely predicted (Siemes et al. 1985). A large number of factors affect the service life of concrete, and their interactions are not well known. These factors include the extent of adherence to design specifications, variability in the properties of hardened concrete, randomness of the in-service environment, and a material's response to microclimates. Two stochastic approaches are the reliability method and the combination of statistical and deterministic models.

4.2.5.1 *Reliability method*—The reliability method combines the principles of accelerated degradation testing with probabilistic concepts in predicting service life. This method has been discussed (Martin 1985) and applied to coatings (Martin 1989) and roofing materials (Martin and Embree 1989). Application of the method is described by considering concrete subjected to a hypothetical laboratory durability test.

As is typical of any engineering material, supposedly identical concrete specimens exposed to the same conditions have time-to-failure distributions. The reliability method takes into account the time-to-failure distributions. By elevating the stresses that effect accelerating failure, probability of failure functions can be obtained, as shown in Fig. 4.2. These failure probabilities are based on the premise that time-to-failure data follow a Weibull distribution (Martin 1985). Testing multiple specimens is required to obtain the distribution. If the failure rate increases as the stress level increases, the service life distribution at in-service stresses can be related to the service-life distribution at elevated stress by the time transformation function $p_i(t)$ as follows (Martin 1985)

$$F_i(t) = F_o(p_i(t)) \tag{4-14}$$

where t is time, $F_i(t)$ is the life distribution at the i'th elevated stress level, and $F_o(t)$ is the service-life distribution at the in-service stress level. From Eq. (4-14), a probability of failure stress time-to-failure (P-S-T) diagram can be prepared as shown in Fig 4.3. The curves in the P-S-T diagram, such as the $F(t)$ = 0.10 curve, are iso-probability lines. The iso-probability lines give, for each stress level, the time at which a given percent of a group of specimens can be expected to have failed. The P-S-T diagram gives a basis to predict the service life of concrete if the in-service conditions are in the range covered by the diagram and are not anticipated to change significantly.

The time-transformation function approach is applicable if the deterioration mechanism under all tested stress levels is the same as that under in-service conditions. Deterioration begins at the instant of stress application, and deterioration is an irreversible cumulative process.

4.2.5.2 *Combination of statistical and deterministic models*—Often, statistical models are combined with deterministic models. For example, the mean service life of buildings has been predicted by using mean values for the parameters in deterministic models that have been developed (Siemes et al. 1985). The standard deviation of the service life is also calculated using the expression

$$\sigma^2(t_1) = \sum_{j=1}^{n}\left[\frac{\partial_{t_1}}{\partial_{x_1}} \cdot (x_i)\right]^2 \tag{4-15}$$

where

$\sigma(t_1)$ = standard deviation of service life;
$\sigma(x_j)$ = standard deviation of the variables x_j affecting service life;
$\partial t_1/\partial x_j$ = partial derivative of t_1 with respect to x_j; and
n = number of variables.

The partial derivatives, $\partial t_1/\partial x_j$, are calculated for the mean values of the stochastic variables. In this approach, it is assumed that the x_j variables are independent of each other.

Instead of normal distributions, log-normal distributions are recommended for representing the service-life distributions (Siemes et al. 1985). A model for carbonation has been developed that demonstrates application of the stochastic method

(Sentler 1984). The depth of carbonation x in concrete was represented by the following form of Fick's diffusion law

$$x^2 = (2D/a)dp \cdot t \tag{4-16}$$

where
D = diffusion coefficient;
a = concentration of concrete constituents that can carbonate;
dp = partial pressure difference for CO_2; and
t = time.

When represented as a stochastic process the depth of carbonation is expressed by

$$f(x, x_0;t) = \frac{1}{\sigma(2\pi t^{1/2})^{1/2}} \exp\left\{\frac{(x - x_0 - \mu t^{1/2})^2}{2\sigma^2 t^{1/2}}\right\} \tag{4-17}$$

which is a normal density function f with mean, $x_o + \mu t^{1/2}$, and variance, $\sigma^2 t^{1/2}$. The initial value of x, x_0, accounts for faster carbonation taking place in the concrete surface layer. Equation (4-17) gives the same mean rate of carbonation as Eq. (4-16), but with variability in the depth of carbonation determined by a normal density function. The model was solved for a case where the concrete cover over steel reinforcement was 25 mm, the concrete had a *w/cm* of 0.5, and the concrete had carbonated for 50 years. An initial fast carbonation was assumed (x_0 = 3 mm). The statistical parameters were based on data obtained during a field study of the relationship between the *w/cm* and depth of carbonation in mm/yr$^{1/2}$. Approximately 16% of the data were more than one standard deviation from the mean value, indicating a normal distribution. A probability of 2.3×10^{-4} for carbonation at 25 mm after 50 years was obtained. If the *w/cm* was increased to 0.6, the probability becomes 3.3×10^{-2}.

4.3—Prediction of remaining service life

Although the methods for predicting the remaining service life of existing concrete structures are basically the same as those for new structures, the existing structures can have the benefit of additional information available (for example, derived material properties and environmental effects). Methods for predicting the remaining service lives of concrete structures usually involve the following general procedures: determining the condition of the concrete, identifying the cause(s) of any concrete degradation, determining the condition constituting the end-of-service life of the concrete, and making some type of time extrapolation from the present state of the concrete to the end-of-service life state to establish the remaining service life.

4.3.1 *Failure due to corrosion*—Most of the reported work on predicting remaining service lives of reinforced concrete structures has dealt with corrosion of the concrete reinforcement. Two major prediction approaches that have been pursued are the modeling approach and corrosion measurements.

4.3.1.1 *Modeling approach*—The modeling approach is illustrated by the work of Browne (1980). He used a diffusion-based model for predicting the remaining service life of in-service reinforced concrete structures exposed to chloride ions. The model only considers the initiation period (Fig. 4.1) and assumes that the diffusion of chloride ions is the rate-controlling process. The following steps help make predictions about the service life:

- Samples are obtained from a concrete structure at different depths from the concrete surface and their chloride contents determined; and
- The following equation is used to obtain values of C_0 and D_{cl}

$$C(x, t) = C_0[1 - erf(x/2(D_{cl}t)^{1/2})] \tag{4-18}$$

where
$C(x,t)$ = chloride concentration at depth x after time t, for a constant chloride concentration of C_0 at the surface;
D_{cl} = chloride ion diffusion coefficient; and
erf = error function.

- Once the values of C_0 and D_{cl} are obtained, then the chloride-ion concentration at any distance from the surface, at any given time, can be calculated; and
- A chloride ion concentration of 0.4%, based on mass of cement, is used by Browne (1980) as the threshold value. The time to reach the threshold concentration at the depth of the reinforcing steel gives the remaining service life.

4.3.1.2 *Corrosion measurements*—The measurement of corrosion current density of steel reinforcement in concrete has been used (polarization resistance technique) in estimating the remaining service life of reinforced concrete in which corrosion is the limiting degradation process (Rodriquez and Andrade 1990; Andrade et al. 1989; Andrade et al. 1990; Clear 1989).

Rodriquez and Andrade (1990) and Andrade et al. (1989, 1990) modeled corrosion current density to estimate the remaining service life. The model measures reduction in steel cross section instead of cracking or spalling of the concrete. The corrosion current density was converted to reductions in the diameter of reinforcing steel by the relationship

$$\theta(t) = \theta_i - 0.023 * i_{corr} * t \tag{4-19}$$

where
$\theta(t)$ = steel reinforcement diameter at time t, mm;
θ_i = initial diameter of the steel reinforcement, mm;
i_{corr} = corrosion rate ($\mu A/cm^2$);
t = time after the beginning of the propagation period, years; and
0.023 = conversion factor of $\mu A/cm^2$ into mm/yr.

The results were converted into service-life predictions by modeling the effects of reducing the cross section of the reinforcement on the load capacity of the reinforced concrete.

Based on the combination of laboratory, outdoor exposure, and field studies, Clear (1989) suggested using the following relationships (that assume constant corrosion rates with time) between corrosion rates i_{corr} and remaining service life:

- i_{corr} less than 0.5 μA/cm^2—no corrosion damage expected;
- i_{corr} between 0.5 and 2.7 μA/cm^2—corrosion damage possible in the range of 10 to 15 years;
- i_{corr} between 2.7 and 27 μA/cm^2—corrosion damage expected in 2 to 10 years; and
- i_{corr} in excess of 27 μA/cm^2—corrosion damage expected in 2 years or less.

4.4—Predictions based on extrapolations

The remaining service life of a concrete structure or element can be predicted from knowing its present condition and extrapolating to when it needs extensive repair, restoration, or should be replaced. The problem is to make the proper extrapolation starting from its condition at inspection to a condition that is used to define end-of-service life.

Rather than making an empirical extrapolation, the time-order approach gives a technical basis for the extrapolation (Clifton 1991). This approach has been previously used for diffusion processes, for example, those involving depth of carbonation or chloride ion diffusion. In the following, the basis for the approach is given.

The amount of degradation of concrete is dependent on the environment, geometry of the structure, properties of the concrete, the specific degradation processes, and the concentration of the aggressive chemical(s). In the time-order approach, these factors are constant and can be represented by a term k_d (Pommersheim and Clifton 1990). Climate changes each season, but usually the variation between years smoothes out over several decades. If this assumption is valid, then only the number of service years need to be represented by the time function t_y, and k_d has an average value over the period considered. Implicit in this analysis is that the same degradation process(es) is active during the past and future life of the concrete.

In this approach, the amount of degradation A_d can be represented by (Clifton 1991)

$$A_d = k_d t_y^{\,n} \qquad (4\text{-}20)$$

where

A_d = amount of accumulative deterioration at time t_y, (years); and

n = time order.

Note that if $n = 0$, there is no degradation. If an initiation period has occurred and its duration is known, then the right-hand side of Eq. (4-20) would be $k_d(t_y\text{-}t_o)^n$, with t_o being the duration of the initiation period. In the development of the approach, the term time order has been used to avoid confusion with the order of a chemical reaction, for example, a second-order reaction that can indicate that two molecules react together.

The overall rate of degradation, R_d is given by

$$R_d = nk_d t_y^{\,n-1} \qquad (4\text{-}21)$$

Equation (4-21) indicates that when $n < 1$, the rate of degradation decreases with time; when $n = 1$, the rate is constant; and when $n > 1$, the rate increases with time.

Defining A_{df} as the amount of damage at failure, it follows from Eq. (4-21) that

$$t_{yf} = (A_{df}/k_d)^{1/n} \qquad (4\text{-}22)$$

where t_{yf} is the time-to-failure. The remaining service life is obtained by subtracting the age of the concrete when the inspection was made from t_{yf}.

The value of n depends on the rate-controlling process. It can be obtained by a theoretical analysis of rate-controlling processes, mathematical modeling of degradation processes, and empirically from accelerated degradation tests (Clifton 1991; Clifton and Pommersheim 1994). Values of n for common degradation processes are available (Clifton and Pommersheim 1991). Examples of using the time order approach for predicting remaining service lives are also available (Clifton 1991; Clifton and Pommersheim 1994).

4.5—Summary

Methods that are used for predicting the service lives of construction materials include estimates based on experience, deductions from performance of similar materials, accelerated testing, applications of reliability and stochastic concepts, and mathematical modeling based on the chemistry and physics of degradation processes. Often these approaches are used in combination. The most promising methods are accelerated testing, applying reliability and stochastic concepts, and using mathematical models.

In comparison to predicting the life of new concrete, few studies on predicting the remaining service life of in-service concrete have been reported. Most of the reported studies have dealt with corrosion of concrete reinforcement, reflecting the magnitude and seriousness of corrosion problems. The most promising approach for predicting the remaining service life of concrete involves applying mathematical models to the degradation process. Theoretical models should be developed, rather than relying solely on empirical models. Many advantages of this approach are apparent, including more reliable predictions, less data needed, and wider applications, such as applicability to a broad range of environmental conditions. Deterministic and stochastic models should be combined to give realistic predictions of the service life. Purely stochastic models have limited application because of the lack of adequate databases that determine statistical parameters. Accelerated tests do not provide a direct method for making the life predictions but can be useful in obtaining data required to support the use of analytical models.

CHAPTER 5—ECONOMIC CONSIDERATIONS

5.1—Introduction

The development of new facilities in both the public and private sectors, as well as existing concrete structures and facilities, requires decisions based on economics and service-life (or life-cycle) information.[‡] Some of the questions that are encountered in making these service-life decisions include:

- Are higher initial construction capital investments justified to obtain longer service life?
- Are higher initial construction capital investments justified to reduce operating or maintenance costs?
- Are higher annual inspection and maintenance costs justified to increase the service life of an existing facility?
- Should outdated facilities be replaced with facilities requiring less frequent, less costly periodic maintenance?
- Should an existing facility be repaired or replaced to reduce day-to-day operating and maintenance costs, or to increase its safety margin?

Service life and pertinent costs are the key elements when addressing these questions. In the above context, service life refers to the effective period for subroutines, such as periodic rehabilitation, as well as the system as a whole. Selecting technically feasible alternatives that result in the minimum overall cost for the defined planning horizon constitutes the scenario for minimum life-cycle cost policy in facilities management. It is, then, the effects of serviceability (or service life) on cash flow over time that constitute the basis for rational management of facilities and assets.

5.2—Economic analysis methods

5.2.1 *General*—Economic analysis is a tool for making rational decisions in engineering situations where a choice should be made from a group of alternatives with differences that can be expressed in monetary terms. The first two steps involved in an engineering economic analysis are the same for all economic analysis methods. First, all technically feasible alternatives that are applicable should be identified. Doing nothing can constitute a viable alternative. Second, cash-flow elements need to be costed-out and time-based cash flow diagrams prepared. In carrying out the latter, a target economic service-life period (planning horizon) needs to be established in which all the cost alternatives are evaluated. Therefore, engineering economic analysis can be used to make decisions affecting the service life of a concrete structure.

5.2.2 *Methods*—Once the alternatives and their respective cash flows have been established, a variety of techniques exist whereby the analysis can be carried out. All analyses should provide the same result in terms of the selection of alternatives, but the nature of the scenario in which the alternatives are being evaluated can favor the use of a particular procedure. ASTM E 1185, describes the following five methods:

- Life-cycle cost (LCC)—Provides the equivalent of the relevant cash flow in either present-value or annual-value terms for each alternative over the selected planning horizon. The details are presented in ASTM E 917.
- Benefit-to-cost ratio (BCR)—Provides a ratio of benefit and cost items that can be quantified in monetary terms for each alternative, based on equivalent values expressed in either present or annual value. The details are presented in ASTM E 964.
- Internal rate of return (IRR)—Provides the interest rate at which the equivalent net cash flow (expressed in terms of either present or annual value) equals zero for comparing alternatives and for comparison with the acceptable discount rate or desired rate of return. The details are presented in ASTM E 1057.
- Net benefits (NB)—Provides the difference between benefit and cost (including disbenefit) items that can be expressed in monetary terms, based on equivalent values of either present or annual value. The details are presented in ASTM E 1074.
- Payback (PB)—Calculates the time to recover investment costs and expenses from income or cost savings, based on equivalent values expressed in terms of either present or annual value for the selected discount rate. The details are presented in ASTM E 1121.

While the ASTM procedures are directed at complete building construction and investment options, the methodologies described are equally applicable to specific components, such as the concrete structure. Furthermore, while many engineering activities, particularly in the case of public works sector, involve cash flows that consist mostly or entirely of disbursements, those methods that involve income (receipts), such as BCR, IRR, NB, and PB, are also applicable. In situations where benefit or revenue streams are not quantifiable, a least-cost economic analysis can be performed. This occurs because, in comparing alternatives, differences between comparable-cost elements result in savings of one alternative over another. The life-cycle cost method is the simplest and most readily applicable procedure for engineering economic analysis. When using these techniques for concrete structures, it is important that the alternatives be analyzed on a common-cost basis. Only those costs relative to the concrete structure should be considered (or alternatively, facility-related costs should be equitably assigned). Similarly, it is critical that the beneficial aspects of rehabilitation be measured correctly in terms of service-life gains.

5.2.3 *Uncertainty and risk*

5.2.3.1 *Approach*—Because engineering economic analysis deals with the future, risk and uncertainty are inherent in the process. ASTM E 1369 describes the range of techniques that are available for addressing uncertainties and risk. The two most commonly used approaches, stochastic processes and sensitivity, are briefly discussed below in general terms.

5.2.3.2 *Stochastic processes*—In some cases, certain future costs are predicated on the occurrence of events that are governed by the laws of probability. Examples include flood damage costs for concrete hydraulic structures resulting from peak flows in excess of design values and other casualty losses such as fire, wind, and vandalism. If the probability of the event occurring during any given year is known or can be estimated from past records, then the most probable annual value assignable to the event is the product of the probability and the cost of the consequence when the event occurs as shown as follows

[‡] Standard terminology of building economics is provided in ASTM E 833.

$$A = Cp \quad (5\text{-}1)$$

where

A = average annual value of the unfavorable consequence of the event over the long run;

C = cost of unfavorable event when it occurs; and

p = probability that the unfavorable event can occur in any given year (decimal).

Note that the probability should range between zero (no chance of the event occurring) and 1 (certainty of the event occurring).

5.2.3.3 *Sensitivity*—Engineering economic analysis deals with cash flows that extend from the present into the future. Uncertainties always exist regarding the length of service life, the timing, and the amounts of future receipts and disbursements. Also, unanticipated expenditures or receipts can occur. Sensitivity analysis determines the effect of variability in the elements of cash flow in an economic decision. The general procedure involved in carrying out a sensitivity analysis should consist of the following:

- Determining which elements of the cash flow are most likely to vary from estimated values;
- Estimating the probable range and choosing the increment of variation for each of the selected elements;
- Selecting an evaluation method (such as present value, annual value, or rate of return) to carry out the evaluations;
- Carrying out the computations using the selected evaluation method for each increment within the estimated range of variability for each of the variable elements; and
- Plotting the computed values (ordinate) against the respective increments of the element (abscissa) for each element in question.

The plots that result from the above procedures (spider diagrams) immediately reveal the sensitivities of the elements in question. The more vertical the plot, the greater the sensitivity. That is, a vertical line represents infinite sensitivity while a horizontal line depicts zero sensitivity.

5.3—Economic issues involving service life of concrete structures

5.3.1 *New facility—predicted service life of candidate alternatives*—The most commonly encountered issue in engineering economic analysis relative to service life is the assignment of the service life values to candidate alternatives, such as leave as is, perform repairs or renovations, or replace. Obviously, this issue is important to the reliability of the service-life assessment. At the same time, it is becoming increasingly difficult to assign estimated service life due to lack of historical perspective due to the accelerated change in technology. Under present conditions, service-life termination is often dictated as much by functional obsolescence as it is by deterioration. Therefore, it is very important that a methodology for estimating service life be developed and applied consistently to the alternatives under consideration. Even if the absolute values may be in error, the comparisons between alternatives should remain reasonably valid.

Table 5.1—Percentage differences between annual values of first costs based on infinite life versus finite life basis

	Percentage difference for perpetual service at interest rate i, %			
Finite life, yr	5%	10%	15%	25%
25	–30.0	–9.2	–3.0	–0.4
30	–23.0	–5.7	–1.5	-0.1
40	–14.0	–2.2	–0.4	0.0
50	–8.7	–0.8	–0.1	0.0
80	–2.0	0.0	0.0	0.0
100	–2.0	0.0	0.0	0.0

5.3.2 *Replacement analysis*—Replacement analysis does not differ in principle from engineering economic analysis in other situations. One of the alternatives, however, is to retain an existing concrete structure that is being considered for replacement; it is referred to as the defender. The other alternatives considered in a replacement analysis (the challengers) are possible candidates for replacing the defender. There could be any of a number of reasons for considering replacement of a structure, including:

- The inability of the existing structure to continue to perform its intended duties without extensive repair or modifications;
- The inability of the existing structure to meet current or predicted future requirements due to changes in demand; and
- The appearance on the market of challengers that can perform the duties of the structure more economically.

The major issue involving service life in replacement analysis is that it is almost never appropriate (in the case of the defender) to invoke the repeatability concept regularly used in economic analysis of alternatives over periods of time longer than the service life of the alternative. Rather, the time period for the replacement analysis (usually called planning horizon) is based on the future need for the structure. If that period of time exceeds the expected remaining service life of the defender, the alternative involving the defender includes a deferred challenger. In this case, a market value (salvage value) has to be estimated for the add-on challenger at the termination of the planning horizon. For convenience and because of the difficulty in predicting future events, it is common practice to limit the planning horizon to the remaining life of the defender. Then it is necessary only to estimate the market values at the end of the planning horizon for each challenger.

5.3.3 *Break-even service life*—In certain instances, the elements of the cash flow vary with time in such a way that a minimum cost (or a maximum profit, depending on circumstances) exists at some point in the service life of the concrete structure. This situation commonly occurs in cases of a structure that produces net receipts that decrease as a function of time.

5.3.4 *Perpetual service (capitalized cost)*—Most concrete and civil structures have very long life expectancies, 50, 100, or more years. Examples include highways, bridges, dams, major buildings, water supply systems, and sewage-collection systems. Because it is difficult to predict future developments, it is often convenient to assume infinite life (perpetual service) in such instances.

Table 5.1 shows that there are not significant differences between perpetual service and finite time spans of approximately 25 years for high interest rates (25%), or 100 years for low interest rates (5%). Also, certain assets, such as land, do possess perpetual life.

CHAPTER 6—EXAMPLES OF SERVICE-LIFE TECHNIQUES

Seven examples representing applications of service-life techniques to concrete structures or structural components are discussed in this chapter. Six of the examples were chosen because of their usefulness in approach and they were applied to actual structures. The seventh example was selected because it illustrates the application of time-dependent reliability methods described briefly in Section 3.4. It is not the intent of these examples to be all inclusive or fully comprehensive but to give guidance on how service-life techniques can be used (for example, establishment of in-service inspection and maintenance strategies). Insight from service-life estimations is essential to establish life-cycle costs for a structure and to justify constructing with more costly materials with enhanced performance characteristics. Also, decisions on using protection systems, repair materials, or demolition and reconstruction, should be based on life-cycle cost estimates. In the first four examples, visual observations and measurements taken from existing structures or materials were input into the methodology. Five of these examples address the situation of steel corrosion in concrete structures, while one specifically addresses chemical attack of concrete. Example I illustrates the technique of comparing cumulative steel corrosion to concrete spalling to obtain the service life. Example II gives an example of how to treat competing degradation mechanisms. Example III describes the challenge in evaluating the many measurements needed to characterize the condition of a structure and predict its service life. Example IV describes how treating each process individually answers questions, such as when to repair and when to rehabilitate. Example V describes how complex environments can be characterized and modeled based on the reaction efficiency of an environment with the concrete. Example VI provides an illustration of calculations used to estimate service life and maintenance demands of a diaphragm wall exposed to saline ground water. Finally, Example VII illustrates the application of time-dependent reliability concepts for service-life predictions.

6.1 *Example I*—Relationship of amount of steel corrosion to time of concrete spalling

The first example describes an investigation of a 30-year-old water-discharge structure of a thermal plant facing the Tokyo Bay in Japan (Morinaga et al. 1994). The approach is based on an analysis that calculates the corrosion rate of the steel reinforcement for each year based upon several parameters, calculates the total amount of corrosion, and compares that to the amount necessary for concrete cracking to occur. Once concrete cracking occurs, that year defines the ending service life of the structure.

The life of a reinforced concrete structure or structural member can be calculated as the amount of corrosion to cause cracking of the cover concrete and the corrosion rate under various conditions of materials, structures, and environments. That is

$$t = Q_{cr}/q \tag{6-1}$$

where

t = life of the structure or member;

Q_{cr} = amount of corrosion to cause cracking of the concrete cover; and

q = corrosion rate.

Q_{cr} is estimated from the diameter of the reinforcement bar d, and the concrete cover c, as in

$$Q_{cr} = 0.590(1 + 2(c/d))^{0.85}d \tag{6-2}$$

The corrosion rate q of the reinforcement is a function of the corrosion rate q_1 of reinforcement in concrete with a known Cl^- content exposed to a specified condition, corrosion rate of concrete q_2 of reinforcement in a concrete containing a known Cl^- content that corresponds to the condition of the structure or member; and corrosion rate q_3 of a structure or member at reference conditions. The corrosion rate was determined as follows

$$q = q_1 q_2/q_3 \tag{6-3}$$

where

$$q_1 = [(-0.50) - 7.45N + 44.1(W/C)^2 + 66.64N(W/C)^2]d/c^2; \tag{6-4}$$

$$q_2 = 2.54 - 0.05T - 6.76H - 22.43O - 0.97N + 0.14TH + 0.50TO + 0.01TN + 59.63HO + 3.30HN + 7.18NO\text{; and} \tag{6-5}$$

$$q_3 = 0.55435 + 1.4027N\text{, given the following} \tag{6-6}$$

where

N = NaCl by mass of mixing water (%) = $165 \times Cl^- / W$;

Cl^- = Chloride content in concrete, kg/m^3;

W = water content per unit volume of concrete, kg/m^3;

w/cm = water-cementitious materials ratio, %/100;

T = temperature, C;
H = humidity, [H = (RH - 45)/100];
RH = relative humidity, %; and
O = oxygen concentration, %/100.

A reference condition of 15 C and 69% RH with 20% oxygen was used.

Field and laboratory investigations were conducted before calculations were made. The field investigations examined the concrete surfaces, removed concrete cores, removed cover concrete to observe the condition of the steel reinforcement, judged the degree of corrosion based on a table (Morinaga et al. 1994), and measured the cover thicknesses. Laboratory tests were performed to measure Cl^- content, determine concrete compressive strength, and estimate mixture proportions through chemical analyses.

Under conditions of constant Cl^- content, Eq. (6-1) can be used to estimate the life of the structure or member. For the current conditions, however, the Cl^- content increases with time. The life-prediction procedure used to address this includes:

- Calculate corrosion rate at each year q_i, based on average Cl^- content at each year;
- Calculate cumulative amount of corrosion at nth year Q_{nyear} by summing q_i to nth year, as follows

$$Q_{nyear} = \sum_{i=1}^{n} (q_i t') \qquad (6\text{-}7)$$

where for the present study t' = 1 and n = 30; and

- Compare Q_{nyear} with the amount of corrosion that cracks cover concrete Q_{cr}, and with end-of-service life defined when $Q_{nyear} > Q_{cr}$.

Results of the model were compared with the actual structure under two environmental conditions—segments in a splash zone and in an intertidal zone. The splash zone exhibited more cracking and more spalled concrete, while the intertidal zone exhibited concrete that was in good to fair condition. The results suggest a satisfactory correlation (53 to 90%) between the model prediction and the observed environmental condition, depending on the concrete and surrounding environmental condition (such as splash or intertidal zone).

6.2 *Example II*—Comparison of competing degradation mechanisms to calculate remaining life

The second example reports on an evaluation and analysis of an ore dock located off an island intersecting the Rouge and Detroit Rivers in Michigan (Hookham 1990). The 11.4 m wide by 412 m long dock was built in 1909 and consisted of a reinforced concrete deck supported by cast-in-place pile caps and wood piles. Additions or repairs to the dock were made in 1920, 1939, 1950, and 1959. A physical evaluation and a structural analysis of the dock were made in the late 1980s to provide for increased traffic flow, designation of a new heavy equipment lay-down area, and increased space for ore storage.

Visual examinations, limited load testing, concrete core sampling and testing, carbonation testing, chloride profiling, and half-cell potential tests were conducted to assess existing conditions. While a large portion of the dock was found to be in good condition, several anomalies were noted, such as local through-deck failures, structural and settlement cracking, spalling, abrasion, and scaling from freezing-thawing cycles. Most of the damage was in the deck and cantilevered portions of the dock. To further assess the structural capacity of the dock relative to the proposed functional changes, the dock was analyzed for bucket impact loads, increased traffic loads, and overall capacity. Results indicated that the dock's condition was acceptable for the proposed changes following repairs, but overhead ore-crane bucket drops could be catastrophic. The repair design included partial replacement of the dock, epoxy injection of passive cracks, expansion joint refurbishment, patching of scaling damage, and deck overlays. Following repair an estimation of the service life of the dock was calculated.

In estimating the service life, models were established for the most likely parameters that could cause loss of function, such as impact loads, corrosion of steel reinforcement, and freezing-thawing damage. An impact loading from a bucket drop was considered to be a singular event that could be catastrophic to the overlay and deck and fitness-for-service would require an assessment of structural adequacy after each event. Freezing-thawing damage was considered to be important to exposed cantilever segments and the deck but not life limiting to the structure. Reinforcing steel corrosion was considered to have the greatest potential to cause structural failure in this environment. Noticeable corrosion damage was evident from cracking and spalling of cover concrete and local reinforcing steel section loss. Two models, based on information presented elsewhere (Vesikari 1988, Sentler 1983), were prepared for predicting service life due to corrosion of steel resulting from carbonation and chloride penetration (deicing salts were used in large quantities during winter).

The time to full-cover carbonation was estimated by

$$t_1 = L / R_c \qquad (6\text{-}8)$$

where

t_1 = the time to full-cover carbonation, yr;
R_c = the rate of carbonation, mm/yr; and
L = the remaining uncarbonated cover, mm.

From observations and measurements after applying phenolpthalein solution, average values for R_c = 0.028 cm/yr and L = 0.95 cm, were determined and used in Eq. (6-8) to estimate full carbonation at t_1 = 34 years from present.

The time to chloride attack based on the existing state of corrosion (corrosion had already initiated at some locations) was modeled as

$$t_2 = [k_c \times k_e \times (L')^2] + (k_a \times L') \qquad (6\text{-}9)$$

where

t_2 = time for chloride ions to reach the depth of the steel reinforcement, yr;
L' = concrete cover, cm;

k_c = quality coefficient of concrete;
k_e = coefficient of environment; and
k_a = coefficient of active corrosion.

From the carbonation model (Eq. (6-8)), the total thickness of uncarbonated concrete cover remaining was determined to be 0.95 cm. On the basis of observed exposure conditions and review of research data (Tuutti 1979, Vesikari 1988), values of 7.59, 0.85, and 4.0 were selected for k_c, k_e, and k_a, respectively. Substituting these values into Eq. (6-9) results in an estimated time for chloride ions to reach the depth of the steel as 9.62 years. Because $t_1 >> t_2$, chloride corrosion was judged to dominate the system. Therefore, the remaining service life of the structure was estimated from t_2 to be approximately 10 years. For design and safety reasons, a remaining life of approximately 5 years was recommended to the client. Follow-up condition surveys have indicated that the owner's use of salts has been discontinued, repairs were still functional, and that the structure has additional service life beyond the predicted 5 years.

6.3 *Example III*—Utilization of multiple input to calculate the life of a structure

The third example describes the examination and analysis of several tunnels that are part of 260 km of underground mains in operation in Moscow (Shilin et al. 1994). The tunnels are used for routing heat and water mains as well as electrical cables. Operational experience of tunnel structures shows that one of the main reasons requiring either change or strengthening of separate members is inadmissible corrosion of steel reinforcement that can lead to accidents and interruption of normal operations. Several of these tunnels were evaluated for service life on the basis of in-place investigations and modeling of the steel reinforcement corrosion process.

Twenty-six kilometers of the tunnels were inspected between 1989 and 1993 to evaluate the general condition of the tunnels. Laboratory (Cl^- contents and concrete strength) and in-place investigations (carbonation depths and reinforcement bar diameters) were also conducted to identify reasons for corrosion and determine the corrosion rate for reinforcement in the tension zone of the roof panel ribs. Visual examinations showed that only the stiffening ribs of the roof panels were being wetted, primarily due to penetration of ground water through joints. Longitudinal cracks of various widths along the steel reinforcement were noted as well as spalling of concrete cover and exposure of steel reinforcement. Chloride contents ranged from 0.11 to 2.07% (by mass of cement). Carbonation depths ranged from 2 to 36 mm. Loss of steel reinforcement section ranged from none to completely corroded.

Because the study was aimed at investigating probabilistic service-life distribution, Monte Carlo simulation methods were used to evaluate how stochastic variations of the following parameters in the course of corrosion propagation influenced the service life of roof panels (such as, structural behavior)

L = depth of concrete cover, mm;
D = diffusion coefficient, mm^2/yr;
C_s = concentration of chlorides in soil, %;
t_{co} = time of waterproofing failure, yr;
k = carbonation coefficient, $mm/yr^{1/2}$;
d_{in} = initial diameter of steel reinforcing bars, mm;
r_{cb} = corrosion rate without chlorides, mm/yr;
r_{Cl} = corrosion rate with chlorides, mm/yr;
r = corrosion rate in air, mm/yr;
R_s = strength of the steel reinforcement, MPa;
R_b = compressive strength of the concrete, MPa; and
M = applied bending moment of the roofing panel, kNm.

Distribution functions were determined for each of these parameters based on laboratory and field investigations and information presented in the literature. Residual service life was predicted for the roof panels according to two limit states: longitudinal cracking due to reinforcement corrosion and ultimate flexural resistance. Variations in estimates of service life for the tunnel roof panels were attributed to variations in both external conditions—time of waterproofing failure, Cl^- concentration in ground, operational loads, and temperature and humidity inside tunnels, and internal conditions—structural parameters, and material properties. The simulations were carried out by generating random numbers for these conditions.

The modeling procedure used a program based on a flow-chart algorithm that calculated the service-life distribution for roof panels by comparing the reinforcement cross section as a single realization of random function to a random value of critical cross section. Minimum service life is the threshold value for time at which the first roof-panel failure occurs and is determined by approximating the failure probability for a tunnel of N panels. This type of analysis resulted in a mean service life for a given tunnel and provides some indication of when damage might first be observed on the tunnel roof panels. Basic components of the flow-chart algorithm include:

- Obtain service life estimations for all roof panels of a tunnel by calculating the probability of failure due to longitudinal cracking or loss of bearing capacity due to reinforcement corrosion for each roof panel.
  —For each section of steel reinforcement in a roof panel:
    - Input data from measurements and observations;
    - Calculate values for remaining parameters identified previously (for example, L, D, C_s, and k);
    - Determine initial diameters of steel reinforcement; and
    - Calculate probability of corrosion and corrosion rate due to carbonation, chloride ions, or the atmospheric conditions based on:
      t_{sp}, time for longitudinal cracking and spalling;
      t_{ct}, time to reach chloride threshold at the steel reinforcement section; and
      t_{cb}, time to carbonate concrete cover.
  —Calculate the final quantities for parameters based on interpolated values.
  —Calculate the lifetime of a roofing panel and repeat for all panels in the tunnel.

—Establish the minimum service life T_o (when the first panel fails).

- Make an histogram of estimates for longitudinal cracking and spalling, and average service life and minimum service life for the tunnel.
- Use histograms to indicate when first roof panel fails and estimate the mean life of the tunnel.

- Repeat for each tunnel.

When comparing algorithm results to actual observations, the estimations were reasonably close. The average remaining service life for roofing panels in tunnels operating under conditions of high temperature and humidity was 22 years, based on cracking, and 73 years, based on loss of bearing capacity. The minimum service life, or time after construction when the first panel would fail, was estimated as 33 years.

6.4 *Example IV*—When to repair, when to rehabilitate

The fourth example presents a methodology to predict the service life of a bridge deck by combining field data and theoretical models (Cady and Weyers 1984; Weyers et al. 1993, 1994). The actual calculation of the service life was made by breaking down the entire process into several independent phenomena, such as corrosion initiation, visible corrosion damage requiring maintenance, and subsequent damage requiring rehabilitation.

The premise of these studies lies in the fact that for a structure to degrade, several subsequent processes occur that can be independently modeled. The chloride level at the structure's surface builds up to a near constant level that is dependent on the surrounding environment. The chloride diffuses through the concrete initiating reinforcement corrosion and eventually concrete spalling occurs. With time, the chloride penetrates deeper and involves more of the reinforcement in the corrosion process. This process continues until a significant amount of damage has occurred, warranting repair and rehabilitation.

The focus of the previously described approach is on defining the time when a bridge deck needs maintenance (repair and patch) T_m, and when it might need resurfacing T_{rehab}. The time to initiate repair or patching of the structure T_m can be calculated by determining the time to corrosion initiation T_i and the time after corrosion has initiated to significant corrosion T_{cor}

$$T_m = T_i + T_{cor} \tag{6-10}$$

In addition, the time to rehabilitation, or resurfacing, of the structure, T_{rehab}, can be calculated using the value determined for T_m, determining the time after significant corrosion occurrence to deterioration T_{det} and the equation that follows

$$T_{rehab} = T_i + T_{cor} + T_{det} \tag{6-11}$$

6.4.1 *Methodology development*—The amount of chloride at a given depth X and time t in a semiinfinite slab with constant surface chloride concentration C_0 can be expressed as a solution to Fick's second law

$$C(x, t) = C_0\left\{1 - erf\left(\frac{x}{2\sqrt{Dt}}\right)\right\} \tag{6-12}$$

where $C(x,t)$ is the chloride concentration at the surface of the steel at the time of interest t. For concrete, $C(x,t)$ can be set equal to the chloride threshold to initiate corrosion. The challenge is to carefully characterize a structure to accurately estimate C_0, D, and X so that the time at which time t becomes T_i (the time to initiate corrosion) can be estimated.

Several models have been developed relating the properties of the concrete and reinforcement to the time-to-corrosion concrete cracking (Cady and Weyers 1984, Weyers et al. 1993). It was concluded that the corrosion rate of the steel was the dominant factor in estimating the time to cracking T_{cor}, and the rate was inversely proportional to T_{cor}. The exact relationship between the corrosion rate and the time-to-cracking, however, has yet to be precisely defined. Based on the models and observations, values of 2 to 5 years were used for T_{cor}.

Subsequent work has resulted in the development of a useful model for predicting the time to cracking after corrosion initiation (Liu and Weyers 1996, Liu 1996). The model is based on the measured field corrosion rate that is used to determine the critical mass of rust products required to crack the concrete while considering the concrete's properties and the construction parameters, such as steel reinforcement bar diameter, spacing, and cover depth. At measured corrosion rates of 0.01 to 0.05 A/m^2 (1 to 5 μA/cm^2) and cover depths of 51 and 76 mm, the time to cracking ranged from five to one years, and ten to two years, respectively.

To estimate the time between initial cracking and the effective functional service life (EFSL), the following equation was used

$$T = ESFL - \left(\frac{ID}{DR}\right) \tag{6-13}$$

where

ID = noticeable initial surface damage resulting from the initiation of corrosion; and

DR = deterioration rate.

The latter is defined as the amount of surface area deteriorated per year after initial cracking. This requires that the damage expected or noticed on a structure be quantified.

Before predicting the service life of a bridge deck, key parameters were obtained for the environment and the structure (Weyers et al. 1993). These parameters included the equilibrium surface concentration of chloride C_0, the diffusion coefficient for chloride in the concrete D, the modified concrete cover on the steel X, the initial damage seen on the surface when spalling occurs ID, the deterioration rate on the surface after significant spalling has occurred DR, and $EFSL$, the effective functional service life based on surface damage.

Using data from 50 bridge decks in New York state, the mean annual snowfall (MAS) and the average annual daily traffic (AADT) were related to the equilibrium surface chloride concentration C_0 measured a short distance into the concrete. This characterization of environments had a correlation constant of $R = 0.76$, and the surface chloride concentration was represented as

$$C_0 = 0.110 \times MAS - 0.000189 \times AADTL + 3.349 \quad (6\text{-}14)$$

where C_0 is in kg/m^3, MAS is in cm, and AADTL is in AADT per lane. Typical values for C_0 range from 1 to 9 kg/m^3. In addition, 2700 samples from 321 bridges in 16 states were measured to obtain chloride concentrations as a function of depth. These data permitted C_0 values to be estimated for environmental exposure conditions of low, moderate, and severe for the various states (Weyers et al. 1994). In addition, this allowed apparent diffusion coefficients D to be calculated for each bridge deck and concrete mixture.

To account for the fact that reinforcement is not necessarily at the design depth, field data were compiled via pachometer readings on actual cover versus design cover (Cady and Weyers 1984). An effective concrete cover L was calculated assuming a normal distribution of concrete covers and utilizing the following equation

$$L = d - \alpha\sigma \quad (6\text{-}15)$$

where d is the design cover; σ is the standard deviation; and α is a factor based on a normal distribution that allows L to represent the cover depth for an amount of reinforcement at or below a given cover depth. In other words, L represents the effective cover depth for an amount of reinforcement at or less than d. Values of α corresponding to different cumulative amounts of reinforcing steel are provided as follows:

Cumulative steel reinforcement, %	α
2.5	1.96
5.0	1.65
10	1.28
15	1.04
20	0.85
30	0.52
40	0.26

These results can be used to calculate the effective cover depth L for different amounts of reinforcing steel in bridge decks.

Each bridge was evaluated for the amount of damage that occurs on the road surface for a given year and the amount of damage necessitating rehabilitation (Cady and Weyers 1984). This damage was viewed in two ways: damage from spalling and total damage (spalls, delamination, patches). From a survey of historical data on a variety of structures, it was determined that 1.4 to 5% spalls warrant the overlayment of a bridge deck. The functional service life, depending on treatment, was determined when total damage between 9.3 to 13.6% of the pavement in the worst traffic lane, or 5.8 to 10% of the pavement in the entire deck area, was reached. The study also showed that other components of a bridge, such as concrete piles, might have total damage levels between 20 and 40%, at the end of service life.

6.4.2 *Application of the methodology*—Using the historical data, two primary approaches to predict the service life of bridge decks were evaluated: the diffusion-cracking-deterioration model (DCDM) and the diffusion-spalling model (DSM) (Cady and Weyers 1984). The DCDM predicts the service life based on five processes:

DCDM processes	Equations to quantify process
Early damage related to construction defects	Eq. (6-15)
Diffusion of the chloride through concrete and initiation of corrosion at a depth of reinforcing steel equal to the initial observable level of damage, 2.5%	Eq. (6-12) and (6-14)
Corrosion of 2.5% of steel and subsequent spalling	Eq. (6-10)
Damage of concrete until cumulative damage results in end of functional life (EFSL)	Eq. (6-13)
Level of cumulative damage at EFSL	Eq. (6-11)

The DSM predicts the service life based on two processes:

DSM processes	Equations to quantify process
Diffusion of the chloride to a depth of reinforcing steel whose corrosion defines the EFSL	Eq. (6-10), (6-12), (6-14), and (6-15)
Corrosion of reinforcing steel at the critical depth resulting in surface damage defining EFSL	Eq. (6-11) and (6-13)

For these two approaches, Eq. (6-10) to (6-15) are used with the primary difference between the DCDM and DSM methods being the definition of L in Eq. (6-15). The DCDM model treats each process separately and calculates initiation from a smaller fraction of reinforcement. The DSM model assumes that the percent surface damage defining EFSL is equal to the surface cover located over the first layer of steel.

Based on observations of 249 bridges in Pennsylvania, a rate of damage per year DR of 2.1% of the deck surface was used. From the same data set, EFSL was defined as 40% damage of the deck surface based on the condition of the decks at the time they were rehabilitated. From the study where engineers defined the conditions of the EFSL for decks, 12% damage in the worst traffic lane is the EFSL (Weyers et al. 1993, 1994). For the substructures, 40% damage of the total surface area was defined as the EFSL. The models were applied using data obtained from several states for the values of C_0, d, σ, and D. The DCDM model correlated with historical estimates on bridge decks, however, the DSM model gave more historically accurate estimates for the piles and caps. In both of these models, the degree of correlation was dependent on being able to accurately define the corrosion rate and the chloride diffusivity.

6.5 *Example V*—Utilization of reaction rate to calculate the life of a sewer pipe

The fifth example addresses a variety of concrete sewer pipes that were studied in California from 1962 to 1976 (Gilbert & Associates 1979). The study included physical inspections of approximately 100 manholes and characterized the general conditions of the sewer line. The data collected were compared with a theoretical equation that had been developed (Pomeroy 1974). The equation quantifies several variables found in a sewer pipe environment that directly affect the rate of concrete deterioration. The equation is as follows

$$C = 0.45k\Phi_{sw}(1/A) \tag{6-16}$$

where

C = average rate of corrosion (chemical dissolution) of concrete by acid, in./yr;

0.45 = experimentally derived constant;

k = acid efficiency coefficient;

Φ_{sw} = flux of hydrogen sulfide (H_2S) gas to the pipe wall; and

A = alkalinity of the concrete.

The flux of hydrogen sulfide (H_2S) gas Φ_{sw} is expressed as

$$\Phi_{sw} = 0.44(sv)^{3/8}j[DS](b/P') \tag{6-17}$$

where

0.44 = empirical constant;

$(sv)^{3/8}$ = energy term: s = energy gradient of waste stream; and v = velocity;

j = fraction of dissolved sulfide present as H_2S, a function of pH (j = 1.0 for pH < 4);

$[DS]$ = concentration of dissolved sulfide in waste stream; and

b/P' = ratio of surface width of waste stream to exposed perimeter of pipe wall above water surface.

The flow characteristics of waste water moving through concrete pipes of different diameters (0.69 m to 1.5 m) and the dissolved sulfide concentrations as a function of time were measured in the California study. Finally, each manhole was characterized by the average pH and penetration depth of attack at the crown and at the springline. The flow characteristics and the sulfide data were used with Eq. (6-16) and (6-17) to calculate the predicted corrosion. The actual corrosion at the manholes was compared to the predicted values and it was found that they were 1 to 1.5 times greater than the predicted values. This was explained by the fact that the manholes were more turbulent than the pipe itself. Also, no measured values for the reaction efficiency, k, had been made. The authors assumed k to have a value of one for the prediction of service life.

Because the life of the concrete in this application is closely tied to the production of sulfuric acid by the Thiobacillus bacteria, the rate of deterioration is tied significantly to k. Most of the other parameters in Eq. (6-16) are constant. Therefore, if k can be measured or approximated, many of the other parameters in the California study could be assumed. With reliable measurements, the values calculated for C can be used to determine when the failure condition of a pipe is reached and to predict the service life of in-service pipes.

6.6 *Example VI*—Estimating service life and maintenance demands of a diaphragm wall exposed to saline groundwater

The sixth example provides information on use of calculations to estimate the service life and maintenance demands of a diaphragm wall exposed to saline ground water (9 gm Cl^-/L) on one side and air on the other (Rostam and Geiker 1993, Geiker et al. 1993). The calculations assume homogeneous concrete in an 8 C environment and were made in connection with a large Scandinavian traffic link. The 10 m high by 0.8 m thick walls were cast as diaphragm walls in bentonite-filled furrows. The concrete *w/cm* was limited to a minimum of 0.4 by the casting method. Steel reinforcement was located at both sides of the walls. The combined selection of concrete composition, cover, and provision for protective and remedial measures was based on considerations of the following: environmental exposure, possibility of obtaining the required quality by the casting method, and required service life of the structure (100 years) taking into account the requirements of the maintenance budget.

The durability of the walls was affected by environmental aggressors attacking both from the internal and external faces of the wall, such as chemical, physical, and mechanical forces. Chloride-induced corrosion was the primary degradation mechanism considered. Due to self-dessication during hydration, the concrete was initially nonsaturated. The saline ground water was expected to flow through the wall because of a difference in moisture content and pressure. Chlorides accumulated at the air exposed side due to evaporation of the flowing water. The service life t_{sl} of the walls consisted of the sum of three periods of time: time-to-steady-state moisture transport (that is, time-to-concrete saturation by chloride containing water), time-to-reach critical concentration at a level of reinforcement at the wall face exposed to air, and time until required repair.

6.6.1 *Time until steady-state moisture transport* t_1—The moisture transport in the wall is expected to occur by capillary suction through the first 100 mm and permeation through the remaining 700 mm (Betonbogen 1985). The time t_c for saturation of the pore system with saline water by capillary suction can be calculated from the following (Betonbogen 1985)

$$t_c = z^2M \tag{6-18}$$

where

z = depth of penetration, assumed 0.1 m; and

M = resistance number, assumed 7 x 10^7 s/m^2.

The permeation of water through the remaining part of the wall is described by D'Arcy's law, and the time t_p until the remaining part of the wall becomes saturated with saline water was estimated from

$$t_p = (L - z - x)\rho\Delta u/q \tag{6-19}$$

where

L = wall thickness, 0.8 m;

z = depth of penetration by capillary suction, assumed 0.1 m;
x = distance between air exposed side and the evaporation zone (see as follows);
ρ = density of the concrete, assumed 2300 kg/m^3;
Δu = difference in moisture content between saturated and nonsaturated concrete, assumed 40% of a total moisture content at 5% by weight of concrete; and
q = rate of water transfer according to D'Arcy's law (see as follows).

The value of x was estimated to be 7 mm from

$$x = L/(1 + K_c(-p_h, 95\%)/(K_p p_s(0.95 - \phi))) \quad (6\text{-}20)$$

where
K_c = transport coefficient for concrete, assumed 5×10^{-15} kg/(Pa m s);
K_p = transport coefficient for paste, assumed 2×10^{-12} kg/(Pa m s);
P_h, 95% = -7×10^6 Pa;
p_s = saturated vapor pressure, 1073 Pa at 8 C; and
ϕ = relative humidity on the air exposed side, assumed 80%.

The value of q was estimated at 4×10^{-8} kg/m^2 s from

$$q = K_c(-p_h, 95\%)/(L - x) \quad (6\text{-}21)$$

For the described concrete wall and the above assumptions, steady-state moisture flow occurred after 17 years (that is, $t_c + t_p = t_1$). At this time, chloride accumulation occurs.

There could be a higher relative humidity, 95% RH, in the evaporation zone than that on the air-exposed side. This would increase the estimated value of t_1 significantly.

6.6.2 *Time to reach the critical concentration at the reinforcement* t_2—Based on the estimated rate of water transfer, the annual flow of saline water through the saturated wall is estimated at 1400 g/m^2. Taking into account the chloride concentration in the ground water, the annual ingress of chlorides is 12.4 g. As mentioned previously, the evaporation front is estimated to be 7 mm below the air exposed surface. At this depth, chlorides are expected to accumulate and then diffuse back into the concrete. Diffusion through the concrete is assumed to follow Fick's Second Law. In this case, however, the surface concentration varies with time so Fick's Second Law was modified as follows

$$C_x = C_s(t) - (C(t) - C_i) erf\frac{x}{2\sqrt{D \cdot t}} \quad (6\text{-}22)$$

where
C_x = chloride concentration at depth x;
Cs = chloride concentration at surface;
Ci = initial chloride concentration;
D = diffusion coefficient;
x = depth;
t = time; and
erf = error function.

Based on an assumed concrete cover (100 mm), the time to reach a critical chloride level (assumed to be 0.1% by weight of concrete for the actual type of structure and concrete) is estimated to be t_2 = 30 years.

6.6.3 *Time until required repair* t_3—For Danish concrete structures, 10 years of active corrosion leads to distress that requires repair. For the diaphragm walls, the moisture condition impedes the corrosion process by decreasing the rate of oxygen diffusion to the cathode. Based on this, the time to develop damage sufficient to require repair is estimated as twice the typical time, or t_3 = 20 years.

6.6.4 *Service life estimate of walls*—Consequently, the service life t_{sl} of the actual structure under the conditions described is estimated to be

$$t_{sl} = t_1 + t_2 + t_3 = 17 + 30 + 20 = 67 \text{years} \quad (6\text{-}23)$$

For a service life of 100 years to be achieved, remedial actions, such as cathodic protection or future maintenance, is required.

6.7 *Example VII*—Application of time-dependent reliability concepts to a concrete slab and low-rise shear wall

Time-dependent reliability concepts are illustrated in example seven for two hypothetical structures—a reinforced concrete slab and a low-rise shear wall. The results presented are drawn from recent research on aging of concrete structures in nuclear power plants (Mori and Ellingwood 1994, 1994a).

6.7.1 *Concrete slab*—The reinforced concrete slab of the first example was designed using the requirements for flexure strength found in ACI 318

$$0.9R_n = 1.4D_n + 1.7L_n \quad (6\text{-}24)$$

in which R_n is the nominal or code resistance, and D_n and L_n are the code-specified dead and live loads, respectively. It is assumed that significant structural loads can be modeled as a sequence of load pulses, the occurrence of which is described by a Poisson process with a mean rate of occurrence λ, random intensity S_j, and duration τ. Such a simple load process has been shown to be an effective model for extreme loads on structures, because the normal service loads challenge the structure to only a small fraction of its strength (Larrabee and Cornell 1981; Pearce and Wen 1984). At the same time, the strength of the slab changes in time, initially increasing as the concrete matures and then decreasing due to corrosion of steel reinforcement. This situation is illustrated by the sample functions $r(t)$ and $s(t)$ for strength and load in Fig. 6.1. The behavior of the resistance over time should be obtained from mathematical models describing the degradation mechanism(s) present (Chapter 4).

With the assumption that load occurrence is a Poisson process, the reliability function (Eq. (3-3)) becomes (Ellingwood and Mori 1993)

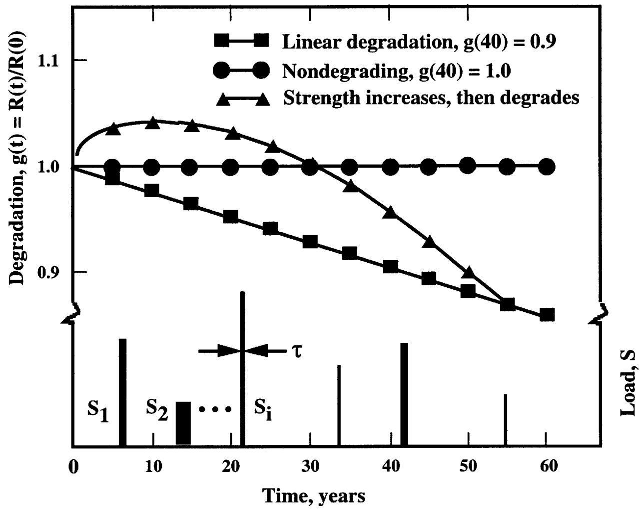

Fig. 6.1—Mean degradation functions of one-way slab (Naus et al. 1996)

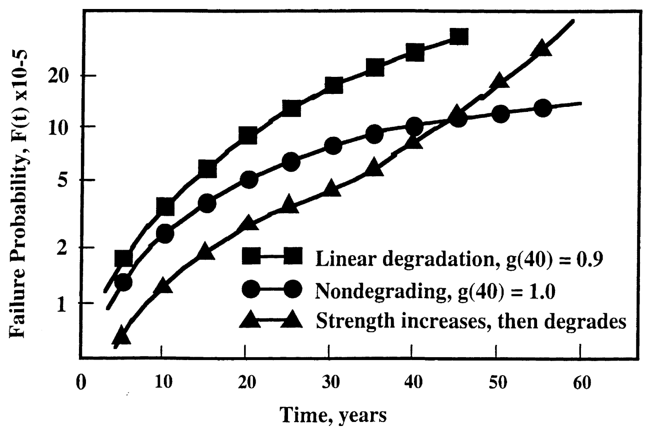

Fig. 6.2—Failure probability of one-way slab (Naus et al. 1996)

$$L(0,t) = \int_0^\infty \exp\left(-\lambda t\left[1 - t^{-1}\int_0^t F_s(rg)dt\right]\right) f_R(r)dr \quad (6\text{-}25)$$

in which $f_R(r)$ is the probability density function of initial strength and $g(t)$ is a function describing the degradation in strength with time normalized with respect to initial strength (Fig. 6.1). The limit state probability or probability of failure during the interval $(0,t)$ can be determined as $F(t) = 1 - L(0,t)$.

Figure 6.2 presents a comparison of limit-state probabilities for intervals $(0,t)$ for t ranging up to 60 years. Three cases are presented (Fig. 6.1):

- No degradation in strength (that is, $R(t)$ = a random variable—this case is analogous to what has been done in probability-based code work to date) (Ellingwood and Galambos 1982);
- $R(t)$ initially increasing with concrete maturity and then decreasing; and
- $R(t)$ decreasing linearly over time to 90% of its initial strength at 40 years.

The basis for the statistics used in this example, as well as the next, is provided elsewhere (Ellingwood and Mori 1993). Neglecting strength degradation entirely in a time-dependent reliability assessment can be quite unconservative, depending on the nature of the time-dependent behavior.

Forecasts of reliability of the type illustrated in Fig. 6.2 enable an analyst to determine the time period beyond which the desired reliability of the structure cannot be ensured. At such time, the structure should be inspected. Intervals of inspection and maintenance required as a condition for continued operation can be determined from the time-dependent reliability analysis. Also, optimized in-service inspection and maintenance strategies based on either full or partial in-service inspection and maintenance approaches can be developed (Mori and Ellingwood 1994, 1994a).

6.7.2 *Concrete low-rise shear wall*—A low-rise shear wall with a height-width ratio equal to one is considered as the second example. It is subjected to vertical load D, that is uniformly distributed on the top of the wall, and lateral load V that is concentrated at the top of the wall. The shear strength of concrete walls can be estimated from empirical models (ACI 318; Barda et al. 1977). These models are not sufficient to analyze the strength of deteriorating low-rise shear walls. Although finite-element analysis provides detailed information on the shear-resistance mechanisms, it requires lengthy computational effort, especially when adapted to reliability analysis. A recent theoretical approach for evaluating shear strength of low-rise reinforced concrete walls determines the ultimate shear strength as the sum of the forces sustained by a truss mechanism V_t and an arch mechanism V_a (Shiraishi et al. 1989; Shohara et al. 1989; Watanoabe and Ichinose 1992). It is assumed that the wall fails if all the reinforcing bars yield in tension and the concrete arch crushes in compression. According to the lower bound theorem of plasticity (Chen and Han 1988), this approach provides a conservative estimate of the shear strength. These models were modified for the reliability analysis of a degrading concrete shear wall (Mori and Ellingwood 1994b).

A wall subjected to chemical attack suffers a loss of concrete section. If the wall is not heavily reinforced in the transverse direction, the contribution of the truss mechanism is small. Therefore, only the strength of the arch mechanism decreases due to the loss of concrete section while the strength attributed to the truss mechanism is independent of the degradation. If the wall is reinforced in the longitudinal direction, the vertical reaction is sustained by the longitudinal reinforcement, and degradation of concrete outside the concrete strut in the arch mechanism can be neglected. Assuming that the stress in the concrete strut is uniform, the degradation function of the shear wall can be given by

$$G(t) = \frac{V_t + V_a(t)}{V_{u0}} = \frac{V_t + G_a(t)\cdot V_a(0)}{V_{u0}} \quad (6\text{-}26)$$

in which V_{u0} is the initial shear strength of the wall, $V_a(t)$ is the shear strength of the arch mechanism at time t, and $G_a(t)$ is the degradation function of the shear strength of the arch mechanism.

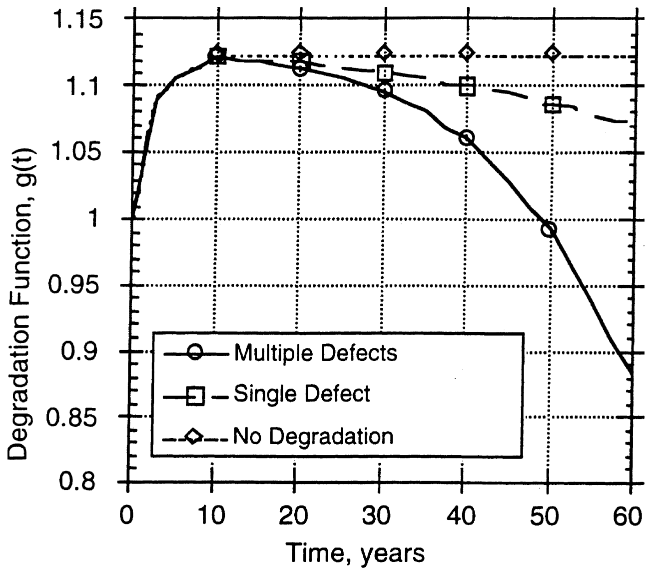

Fig. 6.3—Mean degradation function of wall in shear without repair (Naus et al. 1996).

The ultimate flexural capacity of a wall cross section M_u loaded out of plane, is

$$M_u = T_s\left(\frac{b}{2} - d_c\right) + C_c\left(\frac{b}{2} - k_2 c_u\right) + C_s\left(\frac{b}{2} - d_c\right) \quad (6\text{-}27)$$

in which

T_s and C_s = total force transferred to reinforcement in the tension and compression zone, respectively;

b = wall thickness;

d_c = concrete cover;

c_u = distance from the compressive face to the neutral axis; and

k_2c_u locates the compressive resultant C_c.

For illustration, assume that:

- The wall is subjected to time-invariant dead load *DL* that is uniformly distributed and intermittent lateral load *V* that is concentrated at the top of the wall and can act either in-plane or out-of-plane;
- The wall is designed for in-plane shear based on the current design requirement of ACI 349

$$0.9R_n = E_{ss} \quad (6\text{-}28)$$

in which

R_n = nominal shear strength; and

E_{ss} = structural action due to safe shutdown earthquake. It is assumed that the safe shutdown earthquake load = 3*DL* = 3.21 meganewtons.

- The defect intensity *Y(t)* is modeled

$$Y(t) = C(t - T_I)^2 \quad (6\text{-}29)$$

where

T_I = initiation time; and

C = time-invariant random variable described by a lognormal distribution with mean value m_c of 2.22×10^{-6}/yr and coefficient of variation V_c of 0.5. This value results in an average defect size that is large enough to be found by visual inspection several years after initiation.

- The 28-day specified compressive strength of concrete equals 27.6 MPa. The corresponding expected mean compressive strength at 28 days is 28.7 MPa, and the specified yield strength of the reinforcement is 414 MPa with a mean of 465 MPa (MacGregor et al. 1983); and
- Compressive strength of concrete is assumed to increase during the first 10 years, but does not change thereafter. According to Washa et al. (1989), and assuming the concrete and curing conditions are similar to this study, the mean compressive strength (in units of MPa) at time *t* is evaluated by

$$E[fc(t)] = \begin{Bmatrix} 15.51 + 3.95\ln t & t < 10 \text{ yr} \\ 47.91 & t \geq 10 \text{ yr} \end{Bmatrix} \quad (6\text{-}30)$$

in which *t* is in days. The concrete section area decreases with time as damage accumulates. Other engineering properties of the wall are assumed to be time-invariant.

The mean degradation in shear strength of the wall with a chemical attack occurring in one section of the concrete wall is illustrated in Fig. 6.3. Also illustrated in the figure is the mean degradation in wall shear strength evaluated when the cumulative effect of attack at multiple locations is addressed (Mori and Ellingwood 1992). For comparison purposes, results for no degradation are presented. The gain in shear strength due to the continuous hydration of concrete more than compensates for the strength degradation due to the loss of section area up to approximately 40 years. Subsequently, ignoring the cumulative effect of defects provides an overly optimistic degradation function.

The failure probabilities and the hazard functions associated with the strength degradation illustrated in Fig. 6.3 are presented in Fig. 6.4 and 6.5, respectively. The increase in failure probability due to the strength degradation is small because of the large variability in earthquake load intensity (Mori and Ellingwood 1994b). The hazard function, however, increases rapidly after about 50 years when the cumulative effect of defects is considered.

The mean degradation in flexure and compression strength of the wall is more sensitive to the loss of the outer part of the cross-section area than is the shear strength, as shown in Fig. 6.6. Because loss of the outer part of the wall leads to a reduction in the internal moment arm, the flexural strength degrades more rapidly than the shear strength, which decreases linearly as a function of loss of cross section. Thus, if the governing limit state of the wall is flexure, special attention should be given to the potential for degradation when performing a condition assessment.

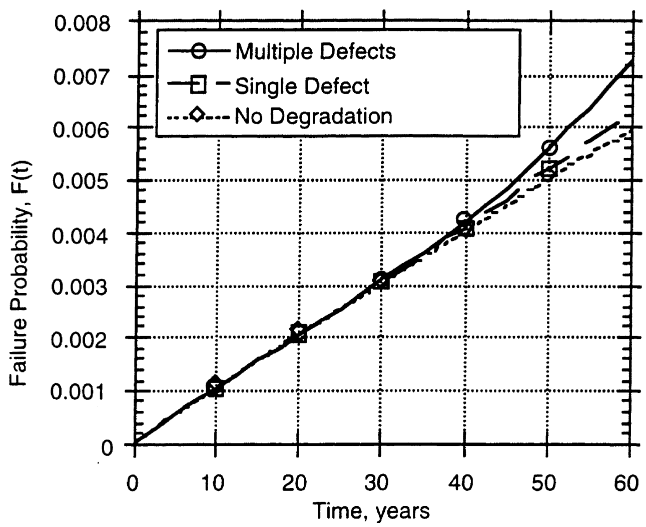

Fig. 6.4—Failure probability of wall in shear without repair (Naus et al. 1996).

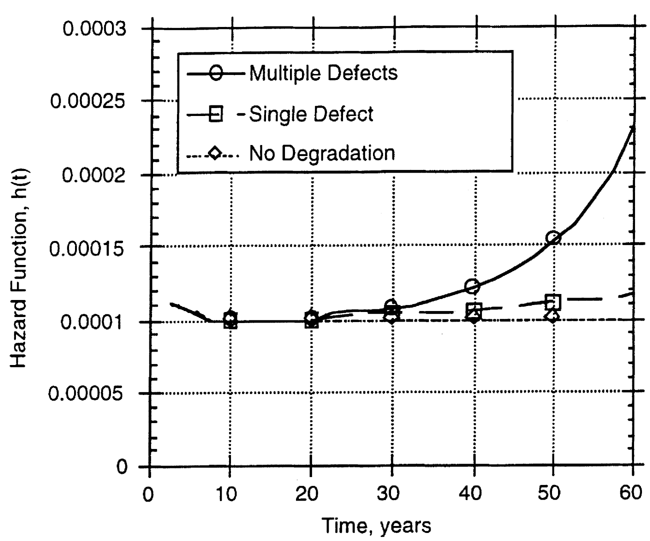

Fig. 6.5—Hazard function of wall in shear without repair (Naus et al. 1996).

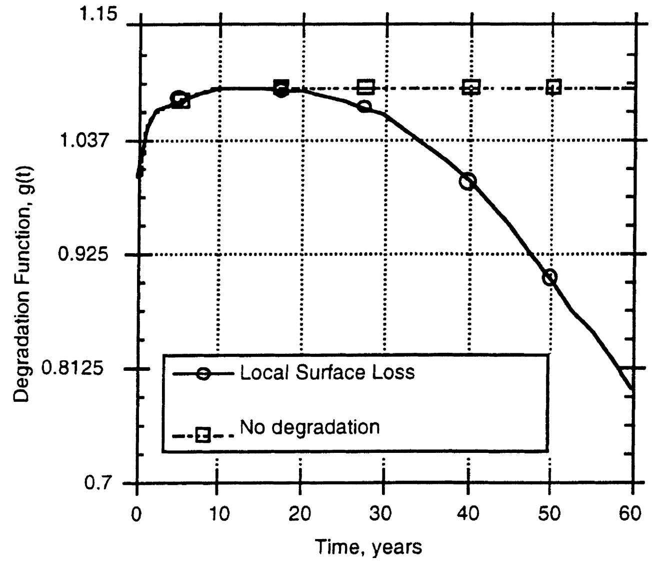

Fig. 6.6—Mean degradation function of wall in flexure and compression (Naus et al. 1996).

CHAPTER 7—ONGOING WORK AND NEEDED DEVELOPMENTS

7.1—Introduction

The service life of new and existing concrete structures is influenced by measures taken during design and construction to resist degradation from imposed loads and environmental conditions (for example, the degree of durability). Durability brings the time element into the design of reinforced concrete structures and should be given equal importance to that given to strength. Design and construction currently consists of seven components from a list (Sommerville 1986): 1) design loads and actions; 2) performance criteria; 3) factors of safety, or reliability; 4) design and detailing; 5) material specifications; 6) workmanship and construction practices; and 7) minimum levels of maintenance. Provisions for durability in the past have primarily been addressed under Items 5 and 6. With few exceptions, performance criteria often have not been directly considered at the design phase. The present approach, with respect to the durability of concrete structures, is based on satisfying specific requirements, such as maximum *w/cm* and minimum cover requirements for certain environmental conditions or applications. The assumption is that if these requirements are met, the structure achieves the desired durability but for an unspecified time. It has been suggested that a new set of standards and codes be required to cover the methodology of environmental interaction between applied loads and predicting of service life (Frohnsdorff and Masters 1990).

Needed developments to allow the service-life prediction of existing structures have been identified (Hookham 1992). These developments include establishing data requirements to support service-life prediction, refinement of mathematical models depicting aging and degradation, an improved understanding of the effects of microclimates on long-term behavior, synthesis of the interaction of physical loading and environmental degradation, and incorporation of the beneficial contributions of prudent inspection and rehabilitation into the service-life prediction process. The remainder of this chapter focuses on the durability aspects of service-life prediction and the design of new structures.

7.2—Designing for durability

Quantitative design for durability requires an improved understanding of the degradation mechanisms, improved characterization of service environments, data on materials, the development of advanced models, and the development of standards and guidelines for the use of design methods and acceptance for durability predictions (Hognestad 1986; Tassios 1985). Extensive research and studies have been carried out to determine the durability of concrete under various service conditions (Biczok 1972; Mehta 1989; Fagerlund 1983, Oland and Naus 1990; Pihlajavaara 1974; Klieger 1958), and progressive changes in the physical and chemical nature of concrete are well understood under such conditions. Using

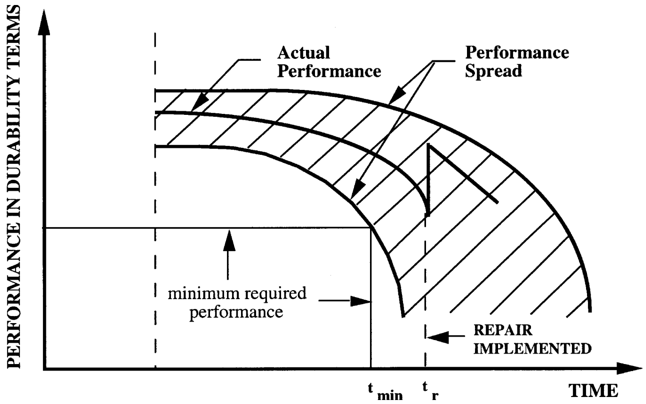

Fig. 7.1—Loss of durability with time (Sommerville 1986).

this information to develop criteria for service-life prediction is far from complete. A problem with a comparative approach such as this, is that each concrete structure is unique because of variability in materials, geometry, and construction practices. Also, over the years, the properties of the concrete materials have changed. Feedback from assessments of performance in practice increase the validity of this approach.

An important aspect in the development of designs for durable structures is that a database be available on measurements of performance of service and environmental influences. The database would contain applications indicating the expected response as well as the variability in response under a variety of conditions. Information in the database could be used in the development of advanced models and verification of existing models. The advanced models would be used in the development of performance versus time relations under defined conditions. Standards and guidelines of acceptance would be used to establish minimum required levels of performance. Performance criteria would be presented as bands of values that take into account uncertainties. Figure 7.1 presents an illustration of the relationship between performance, minimum required performance, and time (Sommerville 1986). Relationships of these types would permit a systematic approach to be used to optimize the design of concrete structures in terms of price and performance under the influence of a given environment. It could also be used during the assessment of an existing structure and the decision on whether or not to repair or rehabilitate the structure and what procedure to use. The effect of a repair or rehabilitation procedure on service life is also illustrated in Fig. 7.1. A report has been prepared that addresses systematic durability design of concrete structures (RILEM 1996). Work is under way to develop a performance-based durability design methodology (BRITE/EURATOM 1998). The objective of this activity is to develop a design approach for durability that is based on the same principles as structural design, such as safety, serviceability, limit states, and reliability. In addition, life-cycle total costs are considered and a realistic performance test procedure is being developed to establish material behavior. Some work has been done on designing structures with enhanced durability for offshore structures (Der Norske Veritas 1971) and pavements (AASHTO 1985), and in the assemblage of performance data (Philipose et al. 1991; Parrott 1987). Work is also under way on developing a database containing information on the effects of aging and environmental factors on concrete and metallic reinforcing materials (Oland and Naus 1994). The role of in-service inspection and maintenance in enhancing the reliability and extending the usable life of reinforced concrete elements in flexure and shear has also been addressed (Mori and Ellingwood 1993, 1994b). Additional effort in each of these areas is required, including development of a new set of standards and codes to cover environmental interaction between applied loads and predicting service life (Frohnsdorff and Masters 1990).

CHAPTER 8—REFERENCES

8.1—Referenced standards and reports

The documents of various standards producing organizations referred to in this document are listed below with their serial designation.

American Assocation of State Highway and Transportation Officials (AASHTO)

Standard Specification for Highway Bridges, 14th Edition.

"Proposed ASSHTO Guide for Design of Pavement Structures," *NCHRP Project* No. 20-7/24, Mar. 1985.

"Method of Test for Resistance of Concrete to Chloride Ion Penetration," AASHTO T 259

American Concrete Institute (ACI)

201.1R	Guide for Making a Condition Survey of Concrete in Service
201.2R	Guide to Durable Concrete
207.3R	Practices for Evaluation of Concrete in Existing Massive Structures for Service Conditions
209R	Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures
210R	Erosion of Concrete Hydraulic Structures
215R	Consideration for Design of Concrete Structures Subjected to Fatigue Loading
216R	Guide for Determining the Fire Resistance of Concrete Elements
222R	Corrosion of Metals in Concrete
224R	Control of Cracking in Concrete Structures
224.1R	Causes, Evaluation, and Repair of Cracks in Concrete Structures
228.1R	In-Place Methods for Determination of Strength of Concrete
228.2R	Nondestructive Test Methods for Evaluation of Concrete in Structures
301	Specifications for Structural Concrete for Buildings
305R	Hot Weather Concreting
306R	Cold Weather Concreting
308R	Standard Practice for Curing Concrete
311.4R	Guide for Concrete Inspection
318	Building Code Requirements for Reinforced Concrete

349 Code Requirements for Nuclear Safety Related Structures

349.1R Reinforced Concrete Design for Thermal Effects on Nuclear Power Plant Concrete Structures

350R Environmental Engineering Concrete Structures

355.1R State-of-the-Art Report Anchorage to Concrete

357R Guide for the Design and Construction of Fixed Offshore Concrete Structures

359 Code for Concrete Reactor Vessels and Containments

362R State-of-the-Art Report on Parking Structures

437R Strength Evaluation in Existing Massive Structures for Service Conditions

503R Standard Specification for Bonding Hardened Concrete, Steel, Wood, Brick, and Other Materials to Hardened Concrete with a Multi-Component Epoxy Adhesive

515.1R A Guide to the Use of Waterproofing, Dampproofing, Protective, and Decorative Barrier Systems for Concrete

SP-66 ACI Detailing Manual

SP-80 Fire Safety of Concrete Structures

SP-103 Anchorage to Concrete

SP-130 Anchorage in Concrete—Design and Behavior

American Society of Civil Engineers (ASCE)

ASCE 11-90 Guidelines for Structural Condition Assessment of Existing Buildings

ASTM Standards

ASTM C 39 Test Method for Compressive Strength of Cylindrical Concrete Specimens

ASTM C 42 Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete

ASTM C 85 Standard Test Method for Cement Content of Hardened Portland Cement Concrete

ASTM C 150 Specification for Portland Cement

ASTM C 215 Test Method for Fundamental Transverse, Longitudinal, and Torsional Frequencies of Concrete Specimens

ASTM C 294 Descriptive Nomenclature of Constituents of Natural Mineral Aggregates

ASTM C 295 Guide for Petrographic Examination of Aggregates for Concrete

ASTM C 341 Test Method for Length Change of Drilled or Sawed Specimens of Hydraulic-Cement Mortar and Concrete

ASTM C 457 Standard Recommended Practice for Microscopical Determination of Air-Void System

ASTM C 469 Test Method for Static Modulus of Elasticity and Poisson’s Ratio of Concrete in Compression

ASTM C 496 Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens

ASTM C 586 Standard Test Method for Potential Alkali Reactivity of Carbonate Rocks for Concrete Aggregates (Rock Cylinder Method)

ASTM C 595 Standard Specification for Blended Hydraulic Cements

ASTM C 597 Standard Test Method for Pulse Velocity through Concrete

ASTM C 642 Test Method for Specific Gravity, Absorption, and Voids in Hardened Concrete

ASTM C 666 Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing

ASTM C 671 Standard Test Method for Critical Dilation of Concrete Specimens Subjected to Freezing

ASTM C 672 Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals

ASTM C 682 Standard Test Method for Evaluation of Frost Resistance of Coarse Aggregates in Air-Entrained Concrete by Critical Dilation Procedures

ASTM C 803 Test Method for Penetration Resistance of Hardened Concrete

ASTM C 805 Test Method for Rebound Number in Concrete

ASTM C 823 Practice for Examination and Sampling of Hardened Concrete in Constructions

ASTM C 856 Standard Recommended Practice for Petrographic Examination of Hardened Concrete

ASTM C 876 Standard Test Method for Half Cell Potentials of Reinforcing Steel in Concrete

ASTM C 1084 Test Method for Portland-Cement Content of Hardened Hydraulic-Cement Concrete

ASTM C 1152 Test Method for Acid-Soluble Chloride in Mortar and Concrete

ASTM C 1202 Test Method for Electrical Indication of Concrete’s Ability to Resist Chloride Penetration

ASTM C 1218 Test Method for Water-Soluble Chloride in Mortar and Concrete

ASTM D 4580 Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding

ASTM D 4748 Test Method for Determining the Thickness of Bound Pavement Layers Using Short-Pulse Radar

ASTM D 4788 Test Method for Detecting Delaminations in Bridge Decks Using Infrared Thermography

ASTM E 632 Standard Practice for Developing Accelerated Tests to Aid Prediction of the Service Life of Building Components and Materials

ASTM E 833 Standard Terminology of Building Economics

ASTM E 917 Standard Practice for Measuring Life-Cycle Costs of Buildings and Building Systems
ASTM E 964 Standard Practice for Measuring Benefit-to-Cost and Savings-to-Investment Ratios for Buildings and Building Systems
ASTM E 1057 Standard Practice for Measuring Internal Rates of Return for Investments in Buildings and Building Systems
ASTM E 1074 Standard Practice for Measuring Net Benefits for Investments in Buildings and Building Systems
ASTM E 1121 Standard Practice for Measuring Payback for Investments in Buildings and Building Systems
ASTM E 1185 Standard Practice for Selecting Economic Methods for Evaluating Investments in Buildings and Building Systems
ASTM E 1369 Standard Guide for Selecting Techniques for Treating Uncertainty and Risk in the Economic Evaluation of Buildings and Building Systems

ASTM Adjuncts

"Discount Factor Tables," *Adjunct* to Practice E 917, Order PCN 12-509179-10

"Building Life-Cycle Cost (BLCC) Computer Program and Users Guide," Order PCN 12-506089-10

BS Documents

BS 1881: Part 6 Methods of Testing Concrete
BS 1881: Part 207 Recommendations for the Assessment of Concrete Strength by Near-to-Surface Tests

CEB Documents

Second CEB/RILEM International Workshop on the Durability of Concrete Structures, Bologna, Italy, 1986.

"CEB-FIP Model Code for Concrete Structures," May 1993.

Durable Concrete Structures, Design Guide, 1992.

CEN Code

Eurocode 2 Design of Concrete Structures

Nord Test

NT Build 443 Concrete, Hardened: Accelerated Chloride Penetration

RILEM Document

Calculation Methods for Service Life Design of Concrete Structures, 1996.

SHRP Documents

SHRP-C-315 "Handbook for Identification of Alkali-Silica Reactivity In Highway Structures"
SHRP-S-324 "Condition Evaluation of Concrete Bridges Relative to Reinforcement Corrosion," Volume 2: Method for Measuring the Corrosion Rate of Reinforcing Steel
SHRP-S-327 "Condition Evaluation of Concrete Bridges Relative to Reinforcement Corrosion," Volume 5: Method for Evaluating the Effectiveness of Penetrating Sealers
SHRP-S-328 "Condition Evaluation of Concrete Bridges Relative to Reinforcement Corrosion," Volume 6: Method for Field Determination of Total Chloride Content
SHRP-S-329 "Condition Evaluation of Concrete Bridges Relative to Reinforcement Corrosion," Volume 7: Method for Field Measurement of Concrete Permeability
SHRP-S-330 "Condition Evaluation of Concrete Bridges Relative to Reinforcement Corrosion," Volume 8: Procedure Manual

The above publications can be obtained from the following organizations:

American Association of State Highway and Transportation Officials
444 N. Capitol St., NW
Washington, D.C. 20001

American Concrete Institute
P.O. Box 9094
Farmington Hills, Mich. 48333-9094

American Society of Civil Engineers
1801 Alexander Bell Dr.
Reston, Va. 20191-4400

ASTM
100 Barr Harbor Dr.
West Conshohocken, Pa. 19428-2959

British Standards Institution (BS)
2 Park Street
London W1A 2BS
England

Comité Euro Internationale du Béton (CEB)
Ecole Polytechnique Federale du Lausanne
Case Postale 88
1015 Lausanne, Switzerland

European Committee for Standardization (CEN)
36, rue de Stassart
B-1050 Brussels
Belgium

Norwegian Building Research Institute
P.O. Box 123

Bindern
N-0314 Oslo
Norway

International Union of Testing and Research Laboratories for Materials and Structures (RILEM)
RILEM Publications S.A.R.L.
ENS
61 Avenue du Président Wilson
F-94235 Cachan Cedex
France

Strategic Highway Research Program (SHRP)
National Academy of Sciences
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

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