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Report on Service Life Prediction

Reported by ACI Committee 365

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This report presents information to the owner and design professional on the service life prediction of new and existing concrete structures. Key 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. This report assists in the application of available methods and tools to predict the service life of existing structures and provides procedures that can be taken at the design and construction stage to increase the service life of new structures. Techniques for predicting the service life of concrete and the relationship between economics and the service life of structures are discussed. Examples provided discuss which service life techniques are applied to concrete structures or structural components. Needed developments to improve the reliability of service life predictions are also identified.

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

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CHAPTER 1—INTRODUCTION AND SCOPE

1.1—Introduction

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). Since then, service life predictions of structures, equipment, and other components have been generally qualitative and empirical. An 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 concrete structures and components. In addition to actual or potential structural collapse, other factors can govern the service life of a concrete structure. This document reports on these service life factors for new and existing concrete structures and components.

Historically, three types of service life have been defined (Sommerville 1992):

(1) Technical service life is the time in service until a defined unacceptable state is reached, such as spalling of concrete, unacceptable safety level, or failure of elements. Examples of technical end of service life include:

(a) Structural safety is unacceptable due to material degradation or exceeding the design load-carrying capacity

(b) Severe material degradation, such as extensive corrosion of steel reinforcement

(c) Excessive deflection under service loads due to decreased stiffness

(2) 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. Examples include:

(a) Need for increased clearance, higher axle and wheel loads, or road widening

(b) Aesthetics become unacceptable—for example, excessive corrosion staining

(c) Functional capacity of the structure is no longer sufficient—for example, a football stadium with insufficient seating capacity

(3) Economic service life is the time in service until replacement of the structure or part of it is more economical than keeping it in service. Examples include:

(a) Maintenance requirements exceed available resource limits

(b) Replacement to improve economic opportunities—for example, replacing an existing parking garage with a larger one due to increased demand

Essentially, decisions concerning the end of service life are related to public safety, serviceability, functionality, and economic considerations.

In most cases, the performance, appearance, or capacity of a structure can be upgraded to an acceptable level bearing in mind costs, which are addressed in Chapter 6 of this report.

ACI 562, a performance-based code for the repair of structural concrete buildings, has taken the terms for "durability" and "service life," and defined "design service life" (refer to Chapter 2 of this report) such that licensed design professionals can design rehabilitation and repair programs for owners, allowing for extension of service life for a given structure. Regardless of the service life concept, the terms "durability" and "service life" are often erroneously interchanged. The distinction between the two terms is that durability is about performance for a given time frame in a given environment, and service life is the amount of time to be expected in a given environment or a specific structure.

Service life evaluation methodologies have application both in the design stage of a structure—where certain parameters are established, such as selection of the water-cementitious materials ratio (*w/cm*), concrete cover, and admixtures—and in the operation phase where inspection and maintenance strategies are developed in support of life cycle cost analyses (LCCA) (Zatar 2014). During the design stage, there is typically a design service life that is anticipated. This is either implicitly established or explicitly considered. The implicit design life relies on code minimums to achieve satisfactory performance for a typical life of a concrete structure. Explicitly considering a design service life allows the owner more control over the long-term expectations for the performance of the structure, although code minimums still need to be met.

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 (RILEM 1986). Service life can be predicted 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. This allows concrete structures to have a reasonable assurance of meeting the specified design service life (Jubb 1992; Clifton and Knab 1989; Sommerville 2003). The acceptance of advanced materials, such as high-performance concrete, can depend on life cycle cost (LCC) analyses that consider predictions of their increased service life.

Methodologies are being developed that predict the service life of existing concrete structures (Ahmad 2003; Zatar 2014). To make these predictions, information is required on the present condition of concrete and reinforcement, 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 a structure should be repaired, rehabilitated, or replaced. Service life evaluations have also been used to establish inspection frequencies to minimize expected expenditures (Mori and Ellingwood 1994a,b). For rehabilitation and repair programs, this methodology becomes complicated and is not yet well understood, as estimating the service life of a repaired component or structure depends on the type and quality of repair (ACI 546R) as well as the performance of the initial structure, and the materials and systems can vary from traditional concrete and its deterioration mechanisms.

Service life comparisons can also be performed by defining a study period over which alternative durability approaches are considered. Parameters of interest—for example, structural capacity, functionality or initial/repair costs—can then be monitored over the study period so that either a certain level of performance is maintained or the value is optimized over the entire study period.

1.1.1 *Service life and sustainability*—Service life calculation and performance estimation tools should be an integral part of sustainability design for concrete structures (Schokker 2010; ASTM E2921). Several techniques presented in this report for determining the expected service life are also effective methods for green building design. The key sustainability criteria of carbon dioxide (CO_2) emission, embodied energy, and other parameters are greatly impacted by the expected service life of a structure. The overall impact of construction activities is reduced the longer materials last and the more maintenance repair events are minimized. Sustainable design of concrete structures is thereby dependent on using appropriate methods for predicting service life.

Model building codes and sustainability codes in Europe, Canada, and many other parts of the world have established minimum service life performance criteria for buildings. In the United States, the codes have only recently included sustainability requirements that are primarily energy- and water-related, leaving the owners, designers, and contractors responsible for establishing the service life criteria. Sustainable design or green building design takes a holistic approach to the observation of the entire life cycle of the facility. Green design principles, when combined with service life design, can provide justifications for exceeding design code minimums. Often, the appropriate selection of construction materials and techniques can result in a service life of more than 75 years with normal maintenance.

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 prediction techniques are applicable to concrete structures or structural components. Finally, needed developments to improve the reliability of service life predictions are presented.

CHAPTER 2—DEFINITIONS AND NOTATION

2.1—Definitions

ACI provides a comprehensive list of definitions through an online resource, "ACI Concrete Terminology," https://www.concrete.org/store/productdetail.aspx?ItemID=CT13. Definitions provided herein complement that source.

design service life (of a building, component, or material)—is the period of time after installation or repair during which the performance satisfies the specified requirements if routinely maintained but without being subjected to an overload or extreme event.

durability—the ability of a material or structure to resist weathering action, chemical attack, abrasion, and other conditions of service, and maintain serviceability over a specified time or service life.

service life—an estimate of the remaining useful life of a structure based on the current rate of deterioration or distress, assuming continued exposure to given service conditions without repairs.

2.2—Notation

A = annual capital invested (6.2.2)
A = alkalinity of concrete (7.5)
A_d = amount of accumulative deterioration
A_{df} = amount of damage at failure
B = linear strain caused by a concentration of sulfate reacted in a specific volume of concrete
C = concentration of dissolved material (5.2.4.3)
C = cementitious material content (7.2)
C' = average rate of corrosion of concrete by acid (7.5)
C'_0 = concentration of reacted sulfate in the form of ettringite (5.2.4.2)
C_0 = initial design and construction costs (6.2.1)
C_0 = surface chloride concentration (7.4.1)
Cl^- = chloride content in concrete
C_s = solution potential of water (5.2.4.3)
C_s = chloride concentration at surface (7.6.2)
C_s = CO_2 concentration at surface (7.9)
C_{ss} = concentration of chloride in soil
C_t = time-dependent chloride concentration
$C(x,t)$ = chloride concentration as a function of depth and time
c = concrete cover
c_b = bound chloride ion concentration
c_f = free chloride ion concentration
c_i = chloride ion concentration at the depth of reinforcement (5.2.4.1)
c_i = concentration of species i in solution (5.5)
$c_i(t_i)$ = i-th expenditure at time t_i
c_s = sulfate concentration in bulk solution
c_0 = chloride ion concentration at outside surface of concrete
D = apparent diffusion coefficient (5.2.4.1)
D_{28} = 28-day diffusion coefficient
D_c = apparent diffusion coefficient (7.9)
D_i = intrinsic diffusion coefficient of sulfate ions
D_i^0 = diffusion coefficient of species i in free water
D_{MK} = diffusion coefficient for metakaolin concrete
D_n = code-specified dead load
D_{PC} = diffusion coefficient for portland-cement concrete
DR = discount rate (6.2.1)
DR = deterioration rate (7.4.1)
D_{SF} = diffusion coefficient for silica fume concrete
D_T = diffusion coefficient at temperature T
D_{UFFA} = diffusion coefficient for ultra-fine fly ash concrete
D_{ULT} = ultimate diffusion coefficient
$D(i)$ = damage state
$[DS]$ = concentration of dissolved sulfide in waste streams
d = diameter of reinforcing bar (7.2)
d = design cover (7.4.1)
d_c = concrete cover
$d_{c,meas}$ = measured concrete cover
d_m = initial diameter of steel reinforcing bars
E = Young's modulus (5.2.4.2)
E = electric field (5.5)
EFSL = effective functional service life
F = Faraday constant (5.5)
F = future value
$F_0(t)$ = service life distribution at the in-service stress level
$F_i(t)$ = life distribution at the i-th elevated stress level
H = humidity
ID = noticeable initial surface damage resulting for initiation of corrosion
i_{corr} = corrosion rate
j = fraction of dissolved sulfide preset as H_2S, as a function of pH
j_i = flux of an ion i in solution
j_i^{adv} = flux of an ion i in solution due to advection
j_i^{diff} = flux of an ion i in solution due to diffusion
K = experimentally obtained dissolution-rate constant
K_c = transport coefficient for concrete
K_p = transport coefficient for pasts
k = acceleration factor (5.2.3.1)
k = carbonation coefficient (7.3)
k = acid efficiency coefficient (7.5)
k_e,k_c,k_t = functions that consider the influence of the environment, including results obtained under accelerated and natural conditions
k_f = coefficient related to environmental conditions
L = thickness of concrete element (5.2.4.1)
L = depth of concrete cover (7.3)
L = amount of reinforcement at or below a given cover depth (7.4.1)
L = wall thickness (7.6.1)
L_n = code-specified live load
M = mass loss in time t from an area A (5.2.4.3)
M = applied bending moment of the roofing panel (7.3)
M = resistance number (7.6.1)
m = change in chloride apparent diffusion coefficient (decay coefficient)
N = number of freezing-and-thawing cycles damaging a laboratory specimen (5.2.3.2)
N = NaCl mass of mixing water (7.2)
n = number of years (5.2.5.2)
n = time order (5.4)
o = oxygen concentration
P = freezing-and-thawing resistance index obtained by the Deutscher Beton Verein (DBV) freeze-salt test (5.2.3.2)
P = principal or capital, present value (6.2.1)
p_f = probability of failure
p_i = time transformation function
p_o = target failure probability
p_s = saturated vapor pressure
Q_{cr} = amount of corrosion to cause cracking of the concrete cover
Q_{nyear} = cumulative amount of corrosion
q = corrosion rate (7.2)
q = rate of water transfer (7.6.1)
R = ideal gas constant
$R_{ACC,0}^{-1}$ = inverse effective carbonation resistance of dry concrete, determined at a certain point of time t_0 on specimens with the accelerated carbonation test
R_{AT} = rate of degradation in accelerated tests
R_b = compressive strength of concrete
R_s = strength of steel reinforcement
R_d = overall rate of degradation

R_{LT} = rate of degradation in long-term, in-service testing
R_n = nominal or code resistance
R_s = discounted residual value at the end of the life cycle
r = interest rate per year
r = corrosion rate in air
r_{cb} = corrosion rate without chlorides
r_{cl} = corrosion rate with chlorides
S_i = random intensity
s = energy gradient of waste stream
T = temperature (7.2)
T = life cycle (6.2.1)
T = target service life (7.9)
T_{cor} = time to cracking
T_{det} = time after significant corrosion occurrence to deterioration
T_i = time to initiate corrosion
T_m = time to maintenance
T_{rehab} = time to rehabilitation
T_{spall} = time for the spall to occur
t = time
t_1 = service life of a structure
t^* = lifetime of a specimen in an accelerated test
t_1 = service life of a structure
t_{co} = time of waterproofing failure
t_{crack} = contact added to time-to-corrosion to determine service life
t_{yf} = time-to-failure
v = velocity
W = water content per unit volume of concrete (7.2)
W = weather function that considers the effect of meso-climatic conditions (7.9)
X = depth
X_{spall} = thickness of the reaction zone causing the spalling
x = distance from concrete surface to steel reinforcement (5.2.4.1)
x = distance between air exposed side and evaporation zone (7.6.1)
$x_c(T)$ = carbonation depth at time T
z = depth of penetration (Eq. (7.6.1a))
z = depth of penetration by capillary suction (Eq. (7.6.1b))
z_i = valence number of the ion
α = parameter based on normal distribution (7.4.1)
α_0 = roughness factor of fracture path
β = statistical parameter
Δd_c = uncertainty in measured concrete cover
Δu = difference in moisture content between the saturated and nonsaturated concrete
ε_t = error term considering the inaccuracies which occur from the accelerated carbonation test method
ε_{xc} = error term that represents the nonuniform carbonation process
λ = mean rate of occurrence
θ = kinetic order of dissolution process
θ_i = initial diameter of the steel reinforcement
$\theta(t)$ = steel diameter at time t
γ_i = activity coefficient of ion i
σ = standard deviation
ρ = density of concrete
τ = duration (7.7)
τ = fracture surface energy of concrete (5.2.4.2)
ν = Poisson's ratio
ψ = electrochemical potential
Φ_{sw} = flux of hydrogen sulfide gas to the pipe wall
ϕ = relative humidity on the air-exposed side

CHAPTER 3—ENVIRONMENT, DESIGN, AND CONSTRUCTION CONSIDERATIONS

3.1—Introduction

Reinforced concrete structures continue to be designed in accordance with national or international consensus codes and standards such as ACI 318, Eurocode 2 (CEN 2006), and the Fédération Internationale du Béton Model Code MC 2010 (*fib* 2013). 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. Examples include designs that consider resistance to chloride ingress and 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 most reinforced concrete structures have initially met their functional and performance code requirements, numerous examples are available where structures, such as pavements, parking structures, marine structures or 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 is needed to design and detail concrete structures for long-term durability (Sommerville 1986; Richardson 2003; Bijen 2003). A more holistic approach is necessary for designing concrete structures based on service life considerations. This chapter addresses environmental and structural loading considerations, environmental and structural interaction, and design and construction influences on the service life of structures. Only a brief introduction is provided; refer to ACI 201.2R and ACI 222R for a more in-depth review.

3.2—Environmental considerations

Design of reinforced concrete structures to provide 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, loading, environmental exposure, and changes in use are important. Because water or some other liquid is involved in almost every form of concrete degradation, concrete penetrability is important and is composed of three parts:

1. Absorption is the process by which a liquid is drawn into and tends to fill permeable pores in a porous solid body;

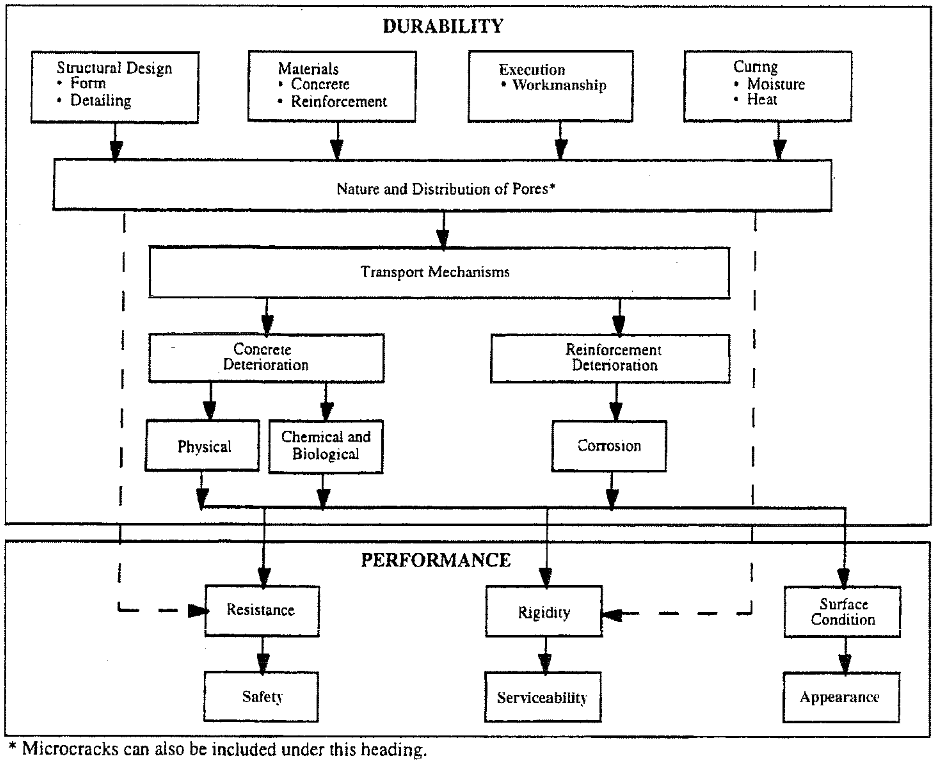

Fig. 3.2—Relationships between the concepts of concrete durability and performance (CEB 1992).

also, the increase in mass of a porous solid body resulting from the penetration of a liquid into its permeable pores

2. Permeability is the ease with which a liquid can flow through a solid under pressure

3. Diffusion is the movement of one medium through another due to concentration gradients

Penetrability is used as a general term encompassing all transport mechanisms. 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 elsewhere (Lawrence 1991; Pommersheim and Clifton 1990; Kropp and Hilsdorf 1995; Nilsson et al. 1997; European Union-Brite EuRam III 2000; Bijen 2003).

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. Concrete durability and performance are related concepts, as shown in Fig. 3.2 (CEB 1992).

Figure 3.2 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, which is the size distribution and tortuosity, 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 transition zone, which is the interfacial region between the particles of aggregate and hydrated cement paste. The physical-chemical phenomena associated with liquid movement through porous solids is controlled by the solid's permeability. Although the coefficient of permeability of concrete depends primarily on the water-cementitious materials ratio (*w/cm*), paste fraction, and maximum aggregate size, it is also influenced by age, consolidation, curing temperature, drying, and the addition of chemical or supplementary cementitious materials (SCMs). 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; Collepardi 2006). Table 3.2a presents a series of different concrete mixtures, made using 3/4 in. (19 mm) maximum size crushed limestone aggregate for which sample transport-related properties were measured as shown in Table 3.2b. The results presented are for this testing method, and would be somewhat different if another testing method had been used.

Two additional factors considered for construction of durable concrete structures are the environmental exposure condition and the specific design recommendations pertaining to the expected form of aggressive chemical or physical attack. An example is designing a structure to minimize the accumulation of water. Exposure conditions or severity are generally handled through a specification that addresses the concrete mixture such as its strength, *w/cm*, and cement content, and details such as concrete cover, as dictated by the anticipated exposure. The following paragraphs are summary descriptions of the primary chemical

Table 3.2a—Concrete mixture proportions and characteristics (Whiting 1988)

Mixture No.	Quantities, lb/yd³ (kg/m³)				Admixture(s)*	w/cm	Slump, in. (mm)	Air content, percent
	Cement	Fine aggregate	Coarse aggregate	Water				
1	752 (446)	1268 (752)	1739 (1032)	222 (132)	A + B	0.258	4.7 (119)	1.6
2	752 (446)	1332 (790)	1825 (1083)	216 (128)	C	0.288	3.5 (89)	2.0
3	642 (381)	1322 (784)	1812 (1075)	258 (153)	D	0.401	3.5 (89)	2.3
4	551 (327)	1322 (794)	1833 (1088)	276 (164)	—	0.502	3.7 (94)	2.1
5	501 (297)	1333 (791)	1831 (1086)	300 (178)	—	0.600	4.2 (107)	1.8
6	413 (245)	1365 (810)	1865 (1107)	312 (185)	—	0.753	4.9 (124)	1.3

*A = silica fume at 100 lb/yd³ (59.4 kg/m³); B = Type F high-range water reducer at 0.38 oz/lb (25 mL/kg); C = Type F high-range water reducer at 0.20 oz/lb (13 mL/kg); and D = Type A water reducer at 0.03 oz/lb (2 mL/kg).

Table 3.2b—Chloride transport-related properties for selected concretes*

Mixture no.†	Cure time, days	ASTM C1202, Coulombs	AASHTO T 259 percent Cl^- by mass of concrete‡	Permeability, μDarcys§		Porosity, percent 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

*Adapted from Whiting (1988).

†Refer to Table 3.2a for description of mixtures.

‡90-day ponding; average of three samples taken at depths from 0.08 to 1.57 in. (2 to 40 mm).

§To convert from μDarcys to m², multiply by 9.87×10^{-7}.

‖Permeability too small to measure.

and physical degradation processes that can adversely impact the durability of reinforced concrete structures, and guidelines for minimizing or eliminating potential consequences of these degradation mechanisms. Combined effects, where more than one of these processes are simultaneously occurring, are also briefly addressed. Available methods and strategies for prediction of the service life of a new or existing reinforced concrete structure for these mechanisms are described in Chapter 5.

3.2.1 *Chemical attack*—Chemical attack involves the alteration of concrete through chemical reaction with either the cement paste, aggregate, or embedded steel reinforcement. Generally, the attack occurs on the exposed surface region of the concrete or cover concrete; however, with the presence of cracks or prolonged exposure, chemical attack can affect entire structural cross sections. Chemical causes of deterioration are grouped into three categories (Mehta 1986; Mindess et al. 2003; Nayak and Jain 2012):

(1) Hydrolysis of cement paste components by soft water

(2) Cation-exchange reactions between aggressive liquids and cement paste, including acid attack

(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.

3.2.1.1 *Sulfate attack*—Sulfates present in aggregates, soils, groundwater, 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 constructing them using concrete that is dense, has low permeability, and incorporates sulfate-resistant cement. Because the C_3A is attacked by sulfates, the concrete vulnerability can be reduced by using cements low in C_3A, such as ASTM C150/C150M Types II and V sulfate-resisting cements. Under extreme conditions, supersulfated slag cements such as ASTM C595/C595M Types VP or VS can be used. Also, improved sulfate resistance can be attained by using SCMs 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. Requirements are provided in terms of cement type, cement content, maximum *w/cm*, and minimum compressive strength, depending on the potential for distress. A model to predict the effect of groundwater containing sulfate of service life is presented in Chapter 5 of this report.

3.2.1.2 *Leaching*—Water that contains little or no calcium ions, or acidic groundwater 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 of the cement paste, resulting in increased permeability. The rate of leaching is dependent on the amount of dissolved salts contained in the percolating liquid, rate of permeation of the liquid through the cement paste matrix, and temperature. The rate of leaching can be reduced by minimizing the permeation of water through the concrete, decreasing the interconnectivity of capillary cavities, by using low-permeability concretes and impermeable coatings or membranes. Factors related to 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, control of cracking, 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 350; ACI 357R). ACI 224R provides crack-control guidelines and ACI 515.2R provides information on selecting barrier systems for concrete. These are typically minimum requirements and are not necessarily adequate for extended service life. A leaching model to predict the dissolution of gypsum is presented in Chapter 5.

3.2.1.3 *Delayed ettringite formation*—Structures undergoing delayed ettringite formation (DEF) can exhibit expansion and cracking. This distress often is attributed to high temperatures greater than 158°F (70°C) during hydration, which is why concrete temperatures should be limited during hydration (ACI 201.2R). These can be caused by excessive steam curing or heat of cement hydration in a mass concrete element, which prevents the formation or causes decomposition of ettringite that is normally formed during the early hydration of portland cement. During service at normal temperatures, ettringite will form again when moisture is present. This may lead to volume expansion, causing cracking and related distress. Details on DEF are presented by ACI 201.2R, Detwiler and Taylor (2003), and Richardson (2003). Neither the mechanisms involved in DEF nor their potential consequences relative to concrete durability are completely understood. Delayed ettringite formation leads to a degradation in concrete microstructure and 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; Famy et al. 2001; Miller and Conway 2003; Ramlochan et al. 2003, 2004; Ekolu 2006; Eriksen et al. 2009).

3.2.1.4 *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 that are unstable in a high-pH environment. Expansive reactions can also occur as a result of the interaction of alkali ions and carbonate constituents in aggregates. 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 (Stark et al. 1993). Controlling alkali-aggregate reactions at the design stage is done by avoiding deleteriously reactive aggregate materials, either by using preliminary petrographic examinations, assessing aggregates using ASTM C1260, using materials with proven service histories, or a combination of these techniques. ASTM C586 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 minimizing the risk of alkali-silica reactions in new concrete include the use of SCMs; using low-alkali cements, which is the restriction of total alkali content in the concrete; adding lithium salts; and applying barriers to restrict or eliminate moisture. The latter procedure is generally the first step in addressing affected structures. For detailed guidance, refer to ASTM C1778.

3.2.1.5 *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 significant 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. However, carbonation, the presence of chloride ions, or both, can destroy the protective film. Corrosion of steel reinforcement also is accelerated by the presence of stray electrical currents (NACE 01110; ACI 222R).

Carbonation occurs when penetrating carbon dioxide (CO_2) from the environment reacts with moisture to form carbonic acid, which 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 is accelerated by the presence of cracks or porosity of the concrete. Concretes that have low permeability and that have been properly cured provide the greatest resistance to carbonation. Also, concrete cover over the embedded steel reinforcement is increased to delay the onset of corrosion resulting from the effects of carbonation (Broomfield 2007).

The presence of chloride ions is a 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—for example, from some accelerating admixtures or chloride-contaminated concrete constituents such as water or aggregates. Potential external sources of chlorides include application of deicing salts, groundwater, or exposure to seawater or spray. Maximum permissible chloride ion contents, as well as minimum specified concrete cover requirements, are provided in codes and guides (*fib* 2013; ACI 318; ACI 222R; ACI 201.2R; Trejo et al. 2016). Two methods are most commonly used for determination of chloride contents in concrete: acid-soluble tests or total chlorides, and water-soluble tests. The chloride ion limits are presented by member type, either prestressed or conventionally reinforced; and exposure condition, either dry or moist. Because water, oxygen, and chloride ions are important factors in the corrosion of embedded steel reinforcement, concrete penetrability is the key to controlling the process. Concrete mixtures should be designed to ensure low penetrability by using low *w/cm*, appropriate cementitious 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. Refer to ACI 548.1R regarding polymers in concrete. ACI 222R also notes that enhanced corrosion resistance can be provided by corrosion-resistant steels, such as stainless steel, stainless steel cladding, galvanized steel (ASTM A767/A767M) and steel (ASTM A1035/A1035M); the application of sacrificial or nonsacrificial coatings such as fusion-bonded epoxy powder; the use of chemical admixtures such as corrosion inhibitors during the construction stage (ACI 212.3R); and cathodic protection (NACE SP0290-2007; NACE SP0408-2008; ISO 12696), 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 515.2R. The resistance of structures to corrosion can also be increased by designing and detailing them to promote the runoff of moisture. Maintenance efforts to minimize the exposure of a structure to chlorides and other aggressive chemicals should also be instituted.

3.2.1.6 *Prestressing steel corrosion*—High-strength steel, such as that used in pre- or post-tensioning systems, corrodes in the same manner as mild steel. Chloride limits are typically lower for prestressing steel compared to mild steel (*fib* 2013; ACI 318; ACI 222R; ACI 201.2R). In addition, it can degrade due to corrosion fatigue, stress corrosion cracking, and hydrogen embrittlement (Nagi and Whiting 1994). Although corrosion of prestressing steel can be either highly localized or uniform, most prestressing corrosion-related failures are the result of localized attack resulting in pitting, stress corrosion, hydrogen embrittlement, or a combination of these. For further discussion of this, refer to ACI 222.2R.

3.2.2 *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; Eide et al. 2011). Although concrete damage due to overload is not considered in this report, it can lead to loss of durability because the resulting cracks or spalling, among other effects, can provide direct pathways for entry of deleterious chemicals. An example is exposure of steel reinforcement to chlorides.

3.2.2.1 *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 groundwater containing large quantities of dissolved salts such as calcium sulfate ($CaSO_4$), sodium chloride (NaCl), and 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. Folliard and Sandberg (1994) reviewed the mechanisms of sodium sulfate crystallization and its effect on concrete durability. 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.

3.2.2.2 *Freezing and thawing*—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 (Whiting and Nagi 1998; Richardson 2003). Structures constructed without adequate air entrainment can have an increased risk for freezing-and-thawing damage. 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* and minimum design compressive strength are also provided, based on the environment 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 C666/C666M 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. The effect of mixture proportioning, surface treatment, curing, or other variables on the resistance of concrete to scaling can be evaluated using ASTM C672/672M. These test methods were designed to simulate the field exposure conditions in an accelerated fashion. As shown in Chapter 5 of this report, such laboratory accelerating tests were used to predict the service life of concrete structures subjected to freezing and thawing and deicer-scaling action.

3.2.2.3 *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 liquid containing solid particles in suspension. Cavitation causes loss of surface material through the formation of vapor-filled cavities and their sudden collapse. The abrasion and erosion resistance of concrete are 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 hardeners of cement and iron aggregate on the concrete surface, can be used to increase abrasion resistance. If unchecked, abrasion or erosion in severe environments can progress from cosmetic to structural damage over a fairly short time frame, such as in industrial facilities. 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).

3.2.2.4 *Thermal damage*—Elevated temperature and thermal gradients affect the strength and stiffness of the concrete. In addition, restrained thermal movement can result in cracking and rapid temperature change, or thermal shock can cause surface spalling. Guidance on minimum reinforcement for concrete exposed to daily temperature fluctuations is provided in ACI 318. A design-oriented approach for considering thermal loads on reinforced concrete structures is provided in ACI 207.2R and 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 address elevated temperature applications by requiring special provisions, such as cooling, to limit the concrete temperature to a maximum of 150°F (65°C), except for local areas where temperatures can increase to 200°F (93°C). At 160°F (70°C) or higher, there is the potential for DEF to occur as previously discussed in 3.2.1.3. 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.

3.2.3 *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; Mindess et al. 2003). 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).

Where the concrete is also exposed to a combination of chlorides and sulfates, the two deterioration mechanisms should be considered when a cement is selected. To reduce the impact of sulfate attack, a low-C_3A cement is desirable, as discussed in 3.2.1.1. However, a lower C_3A content in cement can lead to enhanced corrosion due to chloride ingress. Essentially, chloride binds to hydrates of C_3A. Supplementary cementitious materials containing reactive aluminate phases will also bind to chlorides. If the C_3A content in cement is reduced, the time required to reach a chloride threshold value decreases because there is less chloride binding (Zhang et al. 2003; Suryavanshi et al. 1998; Rasheeduzzafar et al. 1993; Page et al. 1991; Collepardi 2006). The cement used in this case has to balance the benefit of higher C_3A for binding of chlorides versus the low C_3A content preferred for sulfate resistance. An intermediate C_3A content is frequently desirable.

3.3—Design and structural loading considerations

Designers of new projects involving concrete structures address service life by defining several critical concrete parameters. These include maximum *w/cm*, admixtures, reinforcement protection (cover or use of bar coatings or other corrosion-resistant reinforcement), 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. An example is drainage to minimize moisture accumulation and joint details.

Many of the parameters important to service life are established by ACI 318. Chapter 19 of ACI 318-14 list the exposure categories and classes and requirements to achieve durable concrete. Error, omission, or improper identification of these parameters are design deviations that can compromise construction. For durability design to have any value, the above-mentioned condition limit states and the minimum period before these states are reached must be specified (Bamforth and Pocock 2000).

Another important design parameter is the definition of structural loads. Minimum design loads and load combinations are prescribed by legally adopted building codes such as ACI 318. There is a balance between the selection of a design to meet minimum loading conditions and the selection of a more durable design that would result in a higher initial cost but lower life cycle cost (LCC). 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 discussed further in 3.4.

3.3.1 *Background on code development*—While AASHTO (2012) specifies a 75-year structural design life for highway bridges, ACI 318 makes no specific life-span requirements. Other codes, such as Eurocode 2, are based on a design life of 50 years for common structures, but not all environmental exposures are considered. The British Standard Institution's BS 8500-1:2006 provides recommended concrete strengths, maximum *w/cm*, and minimum cover for desired 50- or 100-year service lives for different atmospheric and ground exposure classes. The Canadian Highway Bridge Design Code (CAN/CSA-S6) also requires a 75-year design service life for highway bridges. ACI 318 addresses serviceability through limits on crack width, deflections, and stresses under service loads. 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 concrete and the reinforcement material requirements and detailing. These include a maximum concrete *w/cm*, a minimum air content, and minimum specified concrete cover over the reinforcement depending on exposure conditions. 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 allowed a maximum *w/cm* of 0.45 with air entrainment. In 1963, an appendix was added to ACI 318-63 permitting strength design. Then in 1971, strength design was moved into the body of ACI 318-71, and allowable stress design was placed into the appendix. The use of strength design provided more uniform and consistent levels of 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.

3.3.2 *Load and resistance factor design*—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 is written as

capacity > demand (over the desired service life)

Formulation of this approach is completed 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 as well as 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 because safety is considered the overriding requirement (ACI 318). Service life, other than as affected by cover and concrete strength, generally is not a variable in the calibration process, except as needed to establish loads and load factors. Consequently, the selection of load and resistance factors as formulated offers no specific insight into long-term performance of the structure. Today's practice for designing structures for 50 years or more is to conduct a durability study along with the structural design. The purpose of the study is to define the exposure conditions and provide protective design measures to achieve the required service life (Concrete Society 2008).

3.4—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 and ultimate 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 specific 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 1996; Sturrup et al. 1987; Avram 1981; Price 1951). Guidance for prediction of change due to external influences is found in ACI 357R, 215R, and 209R.

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 such as 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 nondestructive testing 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. Standard methods have been developed for testing non-steady-state water flow (Kropp and Hilsdorf 1995). Laboratory test methods were developed to assess the diffusion coefficient of concrete. ASTM C1556 and Nord Test (NT Build 443) are used to measure the diffusion coefficient at trial stages of construction, either during the design development stage, the preconstruction testing, or both. Such a parameter is used as an input in computer models to predict the service life of structures. In addition, a non-steady-state migration coefficient is also measured in accordance with NT Build 492 and AASHTO T 357. A rapid test was found to be comparable with diffusion coefficient test (Nagi et al. 2014). Diffusion coefficients are currently in some construction specifications as part of durability requirements to achieve the design service life.

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; Nilsson et al. 1996). This type of an approach has been used to predict the remaining service life of structures such as dams subject to leaching of the cement paste by percolating soft water (Temper 1931). The rate of lime loss was measured to estimate the dam service life.

3.5—Construction-related considerations

Construction plans and specifications affect construction 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 by the construction documents (Peck 2006).

The means and methods of construction are the contractor's responsibility. Most often, the construction methods employed meet both the intent and 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 construction; finishing and curing; and sequential construction. With the exception of material procurement and qualification, addressed in 3.3, each stage and the corresponding service life impacts are discussed as follows.

3.5.1 *Initial construction*—Initial construction 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.

Table 3.5.1.1—Summary of impact on service life performance: soil preparation and formwork placement

Condition	Potential service life impact
Improper soil/ subgrade propagation	Structural damage such as cracking, component movement or misalignment.
Formwork too wide or too deep	Excess concrete weight, potential long-term deflection, or excessive cracking. If larger cross section increases strength sufficiently to compensate for extra load, then potentially no service life impacts.
Formwork too narrow or shallow	Decreases structural capacity; excess deflections; cracking.
Formwork not in alignment	Excess waviness or misalignment of form surface relative to reinforcing steel can decrease cover, reducing bond and increasing potential for corrosion, as well as reducing fire resistance.

3.5.1.1 *Soil/subgrade preparation and form placement*—Improper soil/subgrade preparation can lead to excessive or differential settlement, resulting 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. Tolerance information for subgrade elevation and formwork is presented in ACI 117. Examples of the impact of these factors on service life performance are summarized as in Table 3.5.1.1.

3.5.1.2 *Steel reinforcement placement*—Tolerances for reinforcement placement are also given in ACI 117. These documents are referenced in project specifications. Deviation from these standards can result in service life complications such as those listed in Table 3.5.1.2.

3.5.1.3 *Concrete batching, mixing, and delivery*—Concrete can be batched either on the project site or a remote batch plant, then transported to the site. Activities influencing the service life performance include batching errors, improper equipment operation, or improper preparation. Alternatively, concrete can be precast in controlled factory conditions where higher quality and service life behavior can be expected.

Many concrete batch operations incorporate computer-controlled weight and batching equipment. Sources of error are lack of equipment calibration or incorrect mixture selection, or incorrect aggregate moisture corrections. 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 cementitious content, are likely to have the greatest impact on service life.

Equipment preparation is the source of subtler effects. For example, washwater retained in the drum of a transit mixing truck mixes with newly batched concrete to result in a higher *w/cm* than specified. This effect is cumulatively deleterious

Table 3.5.1.2—Summary of impact on service life performance: steel reinforcement placement

Conditions	Potential service life impact
Reinforcement out of specification	Cracking due to inability to support design loads.
Deficient cover	Increased corrosion risk, possible bond failure, reduced fire resistance.
Excessive cover	Potential decreased corrosion risk. However, potential reduction in structural capacity, increased deflection, increased crack width at surface that could increase 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/corrosion of prestressing steel.

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 305R and ACI 306R 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.

3.5.1.4 *Concrete placement*—Proper placement, including consolidation and screeding, is important to the service life of concrete structures. Lack of proper consolidation leads to outcomes such 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 and cracking, or contributing to premature failures.

3.5.2 *Finishing and curing*—Improper finishing or curing leads to premature deterioration of the concrete and reduction of service life. An example is production of a porous and abrasive cover concrete. Table 3.5.2 summarizes common service life issues affecting slabs and other structures.

A guide for curing concrete that maintains the original service life design intent is provided in ACI 308R.

3.5.3 *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 given in 3.5.3.1 and 3.5.3.2 illustrate how this service life impairment can occur.

3.5.3.1 *Shoring and reshoring*—In multiple-story buildings, shoring is used to support the formwork for placing concrete on the next floor. Normal practice is to remove the shoring when the form is removed and then reshore until the

Table 3.5.2—Summary of 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 or early protection	Excessive shrinkage, lower strength, cracking, curling, or freezing and thawing damage.
Use of calcium chloride or any other chloride-contamination of concrete mixture	Degradation of embedded reinforcing steel.

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 is more vulnerable to ingress of deleterious chemicals or materials.

3.5.3.2 *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. Construction joints, which is where fresh concrete is placed against hardened concrete, are prone to poor bonding, allowing leakage, water or moisture ingress, and, thus, premature deterioration.

CHAPTER 4—IN-SERVICE INSPECTION, CONDITION ASSESSMENT, AND REMAINING SERVICE LIFE

4.1—Introduction

Detection and assessment of the environmental factors that cause degradation are vital in predicting service life and in maintaining the capability of reinforced concrete structures to meet their operational requirements. An evaluation methodology that is capable of developing the information necessary to perform a service life prediction is recommended. 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 4.1 presents a flow diagram of a methodology proposed as a guide in assessments of safety-related concrete structures in nuclear power plants (Naus and Oland 1994). This chapter provides information to rate the present condition and assess remaining service life. The level of effort and extent of assessment will vary depending on the objective of the assessment and initial condition of the structure.

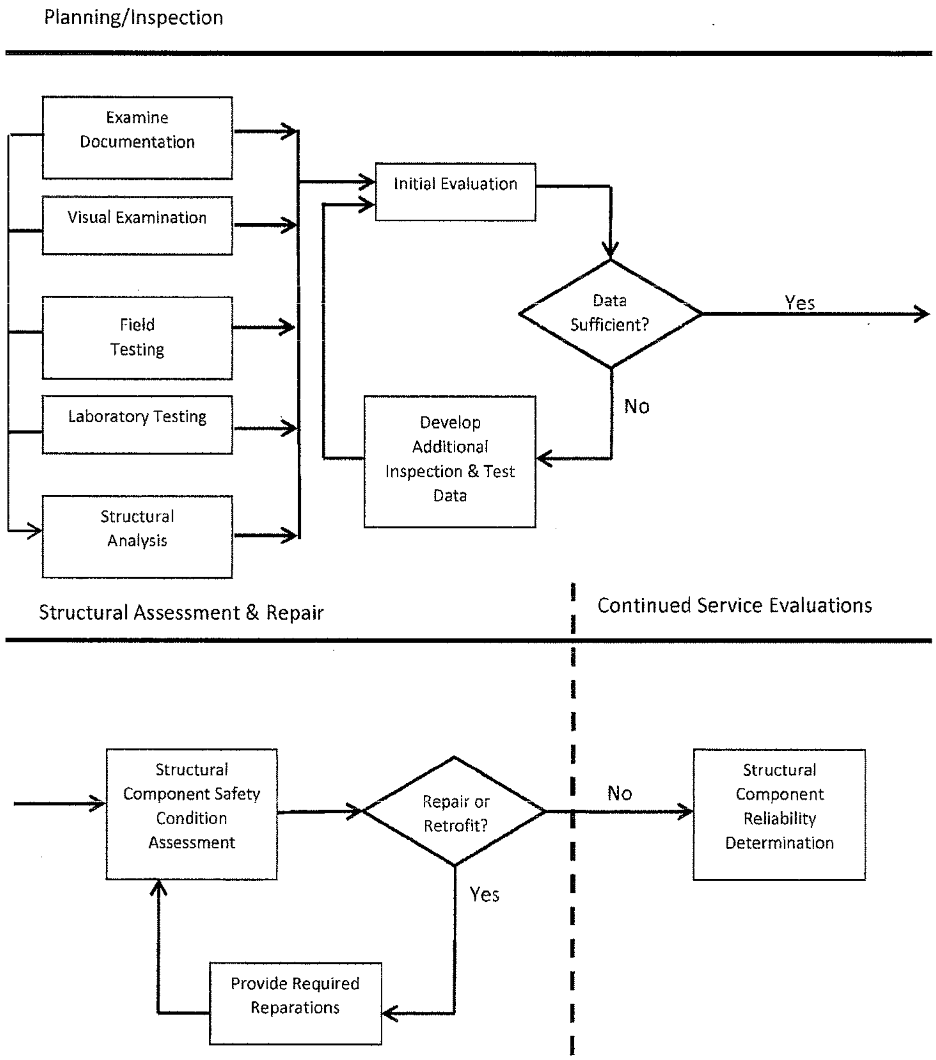

Fig. 4.1—Concrete component evaluation methodology (Naus and Oland 1994); adaptation of a procedure presented in Rewerts (1985).

Performance of a structure is measured by the physical condition and functioning of component structural materials. Murphy (1984) lists four reasons to conduct tests on reinforced concrete to assess performance of the structure:

(1) Noncompliance of properties with specifications

(2) Inadequacies in placing, consolidating, or curing concrete

(3) Damage resulting from overload, fatigue, freezing and thawing, abrasion, chemical attack, fire, explosion, or other environmental factors

(4) Concern about 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 operation and maintenance of the structure.

Chapter 3 indicates that the ability of a reinforced concrete structure to meet its functional and performance requirements over an extended period 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 embedment. In some cases, soil testing could also be required.

Prediction of the remaining service life of a concrete structure requires the evaluation of the existing characteristics and assessment of the extent and causes of observed distress. This evaluation is generally accomplished by a condition appraisal that is discussed in 4.2. Verifying that the structural condition is as depicted in the construction documents, such as drawings; determining current physical condition; quantifying applied loads; and examining any degradation are important precursors to a prediction of the remaining service life. The concerns 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, and the subsequent levels of inspection, maintenance, and repair.

4.2—Preliminary condition assessment

4.2.1 *Current condition*—Determining the existing performance characteristics and extent and causes of observed distress is accomplished through a condition assessment by personnel having broad knowledge in structural engineering, concrete materials, and construction practices. Several resources are available to aid in conducting a condition assessment of reinforced concrete structures and components (ACI 201.1R; ACI 224.1R; ACI 228.2R; ACI 437R; ACI 207.3R; ACI 311.4R; ACI 364.1R; ACI 362.2R; ASTM C823/C823M; Bresler 1977; Perenchio 1989; ASCE 11-99; 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 conducting surveys of existing structures have been prepared (Perenchio 1989; ASCE 11-99). Before conducting a condition assessment, a definitive plan should be developed to optimize the information obtained. The type of information typically required for a service life assessment is shown in Table 4.2.1. 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 is 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 documented (ACI 201.1R). As part of the assessment, it is important to note irregularities or inconsistencies in properties of materials, design, construction and maintenance practices, and the presence and effects of environmental factors. After the visual survey has been completed, the need for additional surveys to determine areas of delamination, corrosion potential or rates, reinforcement location, and cover and the location(s) of embedded metal elements are assessed. Results of these surveys are used to select portions of the structure to be studied in greater detail using the tests described in 4.2.2. Any elements that appear to be structurally marginal, due to either nonconservative design or effects of degradation, are identified and appropriate calculation checks made (4.3.1). A report is prepared after the field and laboratory results have been collated and studied and calculations completed.

4.2.2 *Concrete material systems*—Primary manifestations of distress that can occur in reinforced concrete structures include, for example, cracking and delaminations (cracking parallel to the surface), excessive deflections, and mechanical property (strength) losses. A further discussion of distress symptoms is included in ACI 201.2R. Whether the concrete was batched using the proper constituents and mixture proportioning or it was properly 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. X-ray fluorescence (XRF), X-ray diffraction (XRD), and similar technologies are increasingly adapting to identifying both material composition and presence of contaminants in concrete samples.

Table 4.2.1—Example of information types collected as part of assessment*

Conformance of structure to original design
(a) Documentation review (b) Preliminary site visit i. Visual inspection ii. Pachometer (covermeter) survey to locate and characterize steel reinforcement (for example, size and spacing) and cover depth (c) Preliminary analysis
Inspection for presence of degradation
(a) Visual inspection (b) Crack survey (c) Delamination/spall survey (d) Chloride survey (e) Carbonation survey (f) Sample removal (g) Steel reinforcement corrosion (h) Environmental characterization (i) Evaluation of uniformity of concrete by nondestructive means
Laboratory testing
(a) Petrographic studies; examples are air content, air-void distribution, unstable aggregates, types of distress, and estimation of *w/cm* (b) Chemical studies; examples are chemical constituents of cementitious materials, pH, presence of chemical admixtures, and characteristics of paste and aggregates (c) Concrete and steel reinforcement material properties; examples are strength and modulus of elasticity (d) Concrete absorption and permeability (relative)
Degradation assessment
(a) Current-versus-specified material properties (b) Presence and quantification of excessive concrete crack widths, spalling, or delaminations (c) Pattern of cracking (d) Depth of chloride penetration and carbonation and chloride content profile (e) Steel reinforcement corrosion activity; examples are half-cell potentials, linear polarization resistance, and concrete resistivity
Structural reanalysis for current conditions
(a) Reanalysis for typical dead and live loads considering effects of degradation (b) Examination of demands from other loads; for example, seismic and wind, considering effects of degradation

*This list is not all inclusive.

4.2.2.1 *Field and laboratory test methods*—To supplement visual examination and site work, testing of an existing structure can provide valuable information. Field and laboratory test methods are used to determine hardened concrete properties and to evaluate the condition of concrete in structures. Table 4.2.2.1a presents field and laboratory test methods for evaluating material properties of hardened concrete in existing construction. Table 4.2.2.1b presents test methods that are used to determine structural properties and assess conditions of concrete. A description of the method and principle of operation and applications for the most commonly used test methods are given in other documents (ACI 228.1R; ACI 228.2R; Bungey 1996; Malhotra 1984; Malhotra and Carino 2003) and are thus not presented herein.

Drilling powder samples or removal of cores (ASTM C42/C42M) can help provide additional information. When core samples are removed from areas exhibiting distress, a great

Table 4.2.2.1a—Field and laboratory test methods for determining material properties of hardened concrete in existing construction

Property	Possible methods — Primary	Possible methods — Secondary	Comment
Compressive strength	Cores for compression testing (ASTM C42/C42M; ASTM C39/39M)	Pullout testing (LOK and CAPO tests) (ASTM C900)	Strength of in-place concrete; comparison of strength in different locations; and drilled-in pullout test.
Relative compressive strength	Rebound number (ASTM C805/C805M); ultrasonic pulse velocity (UPV) (ASTM C597)	Pulloff test (BS 1881-207:1992)	Rebound number influenced by near-surface properties. UPV gives average result through thickness.
Tensile strength	Splitting-tensile strength of core (ASTM C496/C496M)	In-place pulloff test (ASTM C1583/C1583M); BS 1881-207:1992)	Pulloff test assesses tensile strength of concrete in the surface layer.
Density	Specific gravity of samples (ASTM C642)	Nuclear gauge	—
Moisture content	Moisture meters; moisture content (ASTM C642)	Nuclear gauge	—
Static modulus of elasticity	Elastic modulus test of cores (ASTM C469/469M)	—	—
Dynamic modulus of elasticity	Resonant frequency testing of sawed specimens (ASTM C215)	UPV (ASTM C597); impact echo; spectral analysis of surface waves (SASW)	Requires knowledge of density and Poisson's ratio (except ASTM C215); dynamic elastic modulus is typically greater than the static elastic modulus.
Shrinkage/expansion	Length change of drilled or sawed specimens (ASTM C341/C341M)	—	Measure of potential additional length change.
Resistance to chloride penetration	Apparent chloride diffusion (ASTM C1556)	90-day ponding test (ASTM C1543; AASHTO T 259); electrical indication of concrete's ability to resist chloride ion penetration (ASTM C1202); PERMIT Ion Migration Test (Basheer et al. 2005)	Establishes relative susceptibility of concrete to chloride ion intrusion; assess effectiveness of chemical sealers, membranes, and overlays.
Air content; cementitious type and content; and aggregate properties including scaling, alkali-aggregate reactivity, freezing-and-thawing susceptibility	Petrographic examination of concrete samples removed from structure (ASTM C856; ASTM C457/C457M); Cement content (ASTM C1084)	Petrographic examination of aggregates (ASTM C294; ASTM C295/C295M)	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)	Petrographic examination of samples (ASTM C295/C295M)	Establish in field if observed deterioration is due to alkali-silica reactivity.
Carbonation, pH	Phenolphthalein (qualitative indication); pH meter (RILEM CPC-18 1988)	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 C805)	SASW; UPV; impact-echo (I-E); impulse-response	Rebound number permits demarcation of damaged concrete.
Freezing-and-thawing damage	Petrography	SASW; impulse response; UPV	Comparison of UPV values enables identification of severely affected areas.
Chloride ion content	Acid-soluble (ASTM C1152/C1152M) and water-soluble (ASTM C1218/C1218M) of extracted samples	Specific ion probe (SHRP-S-328)	Chloride ingress increases susceptibility of steel reinforcement to corrosion.
Air permeability	SHRP surface airflow method (SHRP-S-329)	Autoclam; permeability system (Basheer et al. 1993); Torrent test	Measures in-place permeability index of near-surface concrete
Water absorption (sorptivity)	ASTM C1585	Autoclam permeability system; initial surface absorption tests (Hall 1989)	Measures absorption characteristics of near-surface concrete.
Electrical resistance of concrete	AC impedance using four-probe resistance meter	SHRP surface impedance test (SHRP-S-327)	AC impedance useful for evaluating effectiveness of admixtures and cementitious additions; SHRP method useful for evaluating effectiveness of sealers.

*Strategic Highway Research Program (SHRP).

Table 4.2.2.1b—Nondestructive and minimally destructive test methods to determine structural properties and assess conditions of concrete (adapted from ACI 228.2R)

Property	Methods		Comment
	Primary	Secondary	
Reinforcement location	Covermeter; ground-penetrating radar (GPR) (ASTM D4748)	X-ray and γ-ray radiography	Steel location and distribution; concrete cover.
Concrete component thickness	I-E; GPR (ASTM D4748)	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 gauge (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.
Prestressing steel rupture	Screwdriver test (MacDougall and Li 2007)	Magnetic flux leakage	
Local or global strength and behavior	Load test; deflection or strain measurements	Acceleration; strain, and displacement measurements	Determine acceptability without repair or strengthening; determine accurate load rating.
Corrosion potentials	Half-cell potential (ASTM C876)	—	Identification location of likely areas of reinforcement corrosion.
Corrosion rate	Linear polarization (SHRP-S-324; SHRP-S-330)	Galvanic pulse	Corrosion rate of embedded steel; rate influenced by environmental conditions.
Locations of delaminations, voids, and other hidden defects	Sounding (ASTM D4580/D4580M); I-E; infrared thermography (ASTM D4788); impulse-response; radiography; GPR	Pulse-echo; SASW; intrusive drilling and borescope; UPV	Assessment of reduced structural properties; extent and location of internal damage and defects; sounding limited to shallow delaminations

deal can be learned about the cause and extent of deterioration through strength studies (Hindo and Bergstrom 1985) and petrographic studies (ASTM C856). Additional uses of concrete core samples include calibration of nondestructive testing devices, conducting chemical analyses, visual examinations, determination of steel reinforcement corrosion, and detection of the presence of voids or cracks (Munday and Dhir 1984; Bungey 1979).

4.2.2.2 *Mixture composition*—The question of whether the concrete in a structure was cast using the specified mixture composition is answered through examination of core samples (Mather 1985). By using a point count method (ASTM C457/C457M), the nature of the air void system (volume and spacing) is determined by examining a polished thin section of the concrete under a microscope. An indication of the type and relative amounts of fine and coarse aggregate, as well as the amount of cementitious matrix and cement content, is also determined (ASTM C856). Determination of the original *w/cm* is not covered by a standard test procedure, but the original water, which is the volume of capillary pores originally filled with capillary and combined water, can be estimated for concretes containing only portland cement or ground-granulated blast furnace slag (BS 1881-124:1988). Thin-section analysis can also indicate the type of cementitious material, estimate the *w/cm* and 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 concrete mixture composition, especially at exposed surfaces, becomes increasingly difficult as a structure ages, particularly if it has been subjected to leaching, chemical attack, or carbonation.

4.2.3 *Steel reinforcing material systems*—Assessments of the steel reinforcing system are primarily related to determining its presence and size, cover depth, as well as evaluating the occurrence of corrosion. Determination of material properties such as tensile and yield strengths, and modulus of elasticity, involves removal and testing of representative samples. Pertinent nondestructive test methods that address the steel reinforcing material system are provided in Table 4.2.2.1a and 4.2.2.1b. 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.

4.2.4 *Steel anchorage*—Failure of steel anchorage 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 in an anchorage zone and provide a cursory examination of the anchor to check for weld or plate tearing, plate rotation, or plate buckling. Mechanical tests can verify that pullout and torque capacities of anchors 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.

4.3—Detailed structural assessments

4.3.1 *Detailed structural assessment*—Based on the results of the condition assessment, a detailed structural assessment may be required to confirm the current ability of the structure to sustain the desired loads. If the structure is not structurally sound, it will require appropriate repair to extend the technical service life. A structural assessment could be necessary for one of the following reasons:

(1) There is evidence of possible structural weakness; for example, excessive cracking or spalling

(2) The structure or a portion of it has undergone general or local damage; for example, environmental, impact or earthquake effects

(3) There is doubt concerning the capacity of the structure

(4) Portions of a structure are suspected to be deficient in design, detail, material, or construction

The detailed structural assessment can be of the entire structure, or only for critical elements. Procedures to evaluate the strength of existing structures, found in ACI 437R and the concrete repair code ACI 562, describe detailed structural assessment requirements in depth. Methods for strength evaluation of existing concrete structures include either an analytical assessment or a load test.

For some structures, other types of performance criteria in addition to structural performance may be critical, such as leakage rate or permeability. These performance requirements are addressed through supplementary tests for these characteristics.

4.4—Inspection and maintenance to maintain or predict structural reliability

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. An assessment of parking structures is discussed in ACI 362.2R. 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 are used to predict reliability and usable life of structures. For many service life modeling approaches, the level of maintenance is either not considered (for new structures), or assumed to remain similar to historic norms (for existing 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, as they indicate the presence of a defect; however, they might not provide quantitative

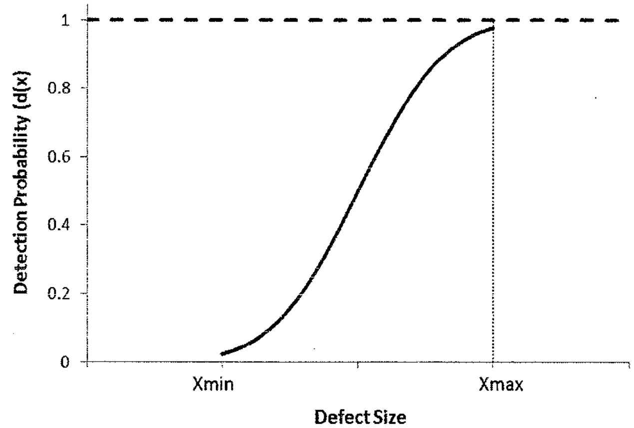

Fig. 4.4a—Defect detectability function (Ellingwood and Mori 1992).

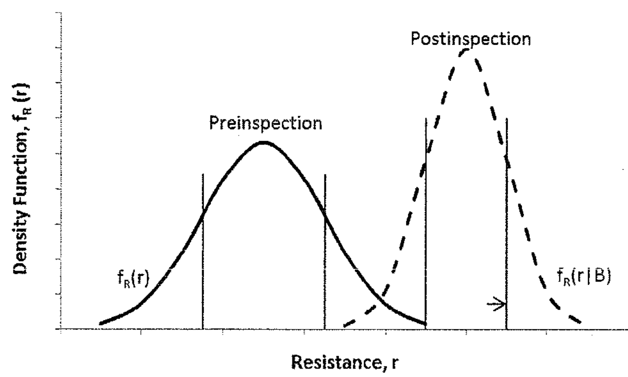

Fig. 4.4b—Role of in-service inspection on strength distribution (Ellingwood and Mori 1992).

data about the defect's size, precise location, and other characteristics that are needed to determine its impact on structural performance. None of these methods can detect a given defect with certainty. The uncertainty of these methods can be described in statistical terms. This randomness affects the calculated reliability of a component. Figure 4.4a 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 allows 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. 4.4b.

The frequency distribution of resistance, based on prior knowledge of the materials used to build 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 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

$$M(t) = R(t) - S(t) \tag{4.4a}$$

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

$$P_f(t) = P[M(t) < 0] = \int_0^\infty F_R(x) f_S(x)dx \tag{4.4b}$$

in which $F_R(x)$ and $f_S(x)$ are the probability distribution function of R and density function of S. Equation (4.4b) 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 reliability over some period of time, $(0,t)$, is more important than the reliability of the structure at the particular time provided by Eq. (4.4b). 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

$$L(0,t) = P[R(t_1) > S_1, ..., R(tn) > Sn] \tag{4.4c}$$

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

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

$$h(t) = -d(\ln L(0,t))/dt \tag{4.4d}$$

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, is evaluated as

$$P[T_f \leq t + \Delta t | T_f > t] = 1 - \exp\left[-\int_t^{t+\Delta t} h(x)dx\right] \tag{4.4e}$$

The hazard function for pure chance failures is constant. When structural aging occurs and strength deteriorates, $h(t)$ characteristically increases with time, as illustrated in Fig. 4.4c.

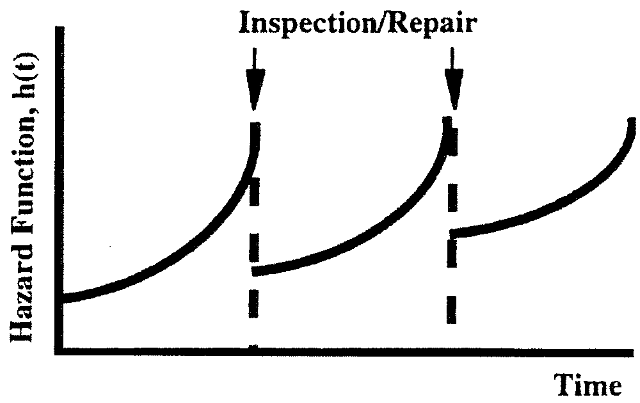

Fig. 4.4c—Role of in-service inspection/repair in controlling hazard function (Ellingwood and Mori 1992).

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

$$f_R(r|B) = P[r < R \leq r + dr,B]/P[B] = cKf_R(r) \tag{4.4f}$$

where $K(r)$ is 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. (4.4e). The effect of in-service inspection or repair on the hazard function is also illustrated in Fig. 4.4c.

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 (LCC). Preliminary investigations have found that LCCs 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 reinforced concrete elements in flexure and shear has been reported (Mori and Ellingwood 1993, 1994c).

CHAPTER 5—METHODS FOR PREDICTING THE SERVICE LIFE OF CONCRETE STRUCTURES

5.1—Introduction

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

Another approach for selecting concrete involves predicting service life using calculations based on likely degradation mechanisms that can manifest in the structure and the reaction rates of these mechanisms. This approach is being used to select concrete for significantly longer 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 not yet 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 can be more severe 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, cracking, freezing-and-thawing cycles, 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. The discussion, therefore, assumes that the influence of otherwise important phenomena such as cracking are appropriately considered tested or treated. 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 7.

5.2—Approaches for predicting service life of new concrete structures

Methods that have been used for predicting the service live 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.

5.2.1 *Predictions based on experience*—Semi-quantitative predictions of the service life of concrete are based on the accumulated knowledge from laboratory and field testing and experience. This knowledge includes both empirical knowledge and heuristics; collectively, these provide the largest contribution to the basis for standards for concrete. If concrete is made following standard industry guidelines and practices, it is assumed that it will have the required service life. This approach gives an assumed service life prediction. Concrete can perform adequately for its design life, especially if the design life is 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 experience with concrete, when new or aggressive environments are encountered, or when new concrete materials will 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).

5.2.2 *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 will have the same life. A problem with this approach is that 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. In addition, some new concrete structures may be located in areas where historical data are not available.

5.2.3 *Accelerated testing*

5.2.3.1 *Approach*—Most durability tests for concrete use more severe environments, such as a higher concentration of reactants, temperature, humidity, hydraulic pressure or electrical potential, 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 is obtained from

$$k = R_{AT}/R_{LT} \tag{5.2.3.1}$$

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. Degradation in many structures is the result of multiple deterioration mechanisms that in many structures acts synergistically. This complicates the modeling process and interpretation of the results.

A recommended practice for developing accelerated short-term tests that can obtain data for making service predictions and for solving service life models consists of four main parts: 1) problem definition; 2) pretesting; 3) testing; and 4) 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. (5.2.3.1). Accelerated tests, however, can provide information on concrete degradation that is needed to solve mathematical models for predicting service lives or allow comparison of different design alternatives.

5.2.3.2 *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

$$t_1 = kt^* \qquad (5.2.3.2a)$$

where k is a constant that is derived from testing. This approach is then applied to freezing-and-thawing resistance testing of concrete. 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 is evaluated by

$$t_1 = k_e N \qquad (5.2.3.2b)$$

where k_e is a coefficient related to environmental conditions, and N is the number of freezing-and-thawing cycles for 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

$$t_1 = k_f P \qquad (5.2.3.2c)$$

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 percent sodium sulfate (Na_2SO_4) solution until failure occurred, defined as an expansion of 0.5 percent, 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 percent sodium sulfate (Na_2SO_4) solution for 16 hours and forced air drying at 129°F (54°C) for 8 hours. Comparing the times for specimens to reach an expansion of 0.5 percent in the accelerated test and the continuous immersion test, it was estimated that 1 year of accelerated testing was equivalent to 8 years of continuous immersion. In this case, Eq. (5.2.3.1) becomes

$$K = 8 = R_{AT}/R_{LT} \qquad (5.2.3.2d)$$

where R_{AT} is the rate of expansion in the accelerated test, and R_{LT} is the rate of expansion in the long-term continuous immersion test.

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

5.2.3.3 *Application to corrosion evaluation*—Accelerated testing is frequently used to predict long-term resistance against corrosion for concrete structures. Generally, these methods involve a short-term evaluation of the ionic transport properties of the concrete to consider the time-to-corrosion initiation, neglecting the propagation of corrosion damage.

Considerable research has been conducted using electrochemical methods to move ions into concrete in a much shorter period than the 90 days required for diffusion testing such as ASTM C1556. A commonly used test is ASTM C1202, where a 60 V DC charge is passed through a 2 in. (50 mm) slice of a 4 in. (100 mm) diameter concrete cylinder. With these tests, the amount of current measured is directly proportional to the electrical conductivity of the concrete. Additional standard tests useful for service life modeling are ASTM C1760 for bulk conductivity and Nordtest (NT) Build 492 for chloride migration (Tang and Sørensen 2001). Results of these tests are often used as a measure of the quality of the concrete with respect to diffusion, which has then been related to the expected service life of the structure (Barde et al. 2009). One weakness with tests is that it not only moves chloride ions within the sample, but activates any electrical ion, particularly hydroxyl ions (Andrade 1993). Thus, increased pH of the pore solution can increase the charge passed under the electrical test methods and potentially lead to poor predictions. This discrepancy is resolved in NT Build 492 by measuring the chloride penetration on autopsied specimens using silver nitrate spray. In addition, caution is required when applying laboratory tests directly to the field where conditions may vary significantly from those in standardized tests.

The Southern Exposure Test (Virmani 1983) is used to investigate the propagation period, after the reinforcing steel has begun to corrode. An aggressive test, it is primarily used to investigation the relative influence of different corrosion protection mechanisms such as epoxy coating (Lee and Krauss 2004), steel composition (Cui and Krauss 2008), or the use of corrosion inhibitors (Powers et al. 1999). However, due to the high variability in corrosion rates in real structures, it is difficult to establish a unique correlation between the rate of corrosion in the Southern Exposure Test and in service.

Accelerated modelling of carbonation is also possible (*fib* Bulletin 34 2006), typically by introducing higher concentrations of carbon dioxide into the test chamber than would be experienced in the environment. The relationship between the accelerated testing and the performance in service will depend on the characteristics of both these conditions (Bouzoubaâ et al. 2010).

5.2.4 *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 are 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 diffusion coefficient in corrosion models. Standard methods have been developed for testing non-steady-state water flow in concrete (Kropp and Hilsdorf 1995). Furthermore, methods for testing transport processes, including chlorides or moisture, are also available (Nordtest (NT) Build 443 and 492; ASTM C1556; ASTM C1585).

5.2.4.1 *Model of corrosion of reinforcing steel*—Most corrosion models for reinforced concrete follow the same approach and are based on a general deterioration 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. 5.2.4.1a, 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

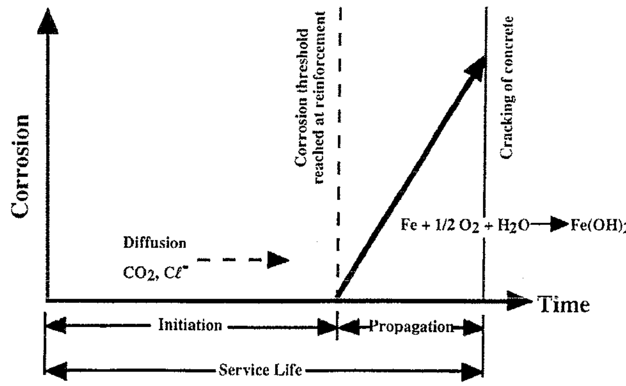

Fig. 5.2.4.1a—Schematic of conceptual model of corrosion of steel reinforcement in concrete (Tuutti 1982).

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 (Gowripalan et al. 2000; Pacheco and Polder 2010; Boulfiza et al. 2003).

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)

$$\frac{\partial c_f}{\partial t} = \frac{D\partial^2 c_f}{\partial x^2} \qquad (5.2.4.1a)$$

where D is the apparent diffusion coefficient; x is distance from concrete surface to steel reinforcement; and t is time.

Because chloride ions react with some of the hydration products of cementitious binders, the concentration has two components—concentration of bound chloride ions (c_b) and concentration of free ions (c_f).

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 for carbon steel reinforcing bars. 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. (Bažant 1979a,b). However, when the oxygen levels are very low, the potential can fall to below the hydrogen/water equilibrium and corrosion can continue with hydrogen evolution as the corresponding cathodic reaction (Hansson 1986).

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. (5.2.4.1a) for a reinforced concrete element under constant environmental conditions is

$$\frac{C}{C_0}(Z,t) = \sum_{n=0}^{\infty}(-1^n)\left[\text{erfc}\left\{\frac{(2n+1)-y}{2\sqrt{r}}\right\}+\text{erfc}\left\{\frac{(2n+1)+y}{2\sqrt{r}}\right\}\right] \tag{5.2.4.1b}$$

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

In the present case, however, only the $n = 0$ term of Eq. (5.2.4.1b) requires consideration. Higher-order terms have insignificant contributions to the summation, reducing the equation to

$$\frac{C}{C_0} = \text{erfc}\frac{(1-y)}{2\sqrt{r}} \tag{5.2.4.1c}$$

where $1 - y = x/L$. The model was solved for the case where the threshold concentration C_t of chloride ions was 0.4 percent (based on the mass of the cement), the concentration of chloride ions at the surface of the concrete C_0 was 0.7 percent (based on the mass of cement), $x = 2$ in. (50 mm), $L = 11.8$ in. (300 mm), and $C_i = 0$ at $t = 0$. Results for different concrete cover depths and chloride ion diffusivity coefficients are presented schematically in Fig. 5.2.4.1b.

Results show that the effect of the cover is proportional to x^2. For example, increasing x from 1 in. (25 mm) to 4 in. (100 mm) increases the service life by a factor of $(4/1)^2$ $[(100/25)^2]$, or 16. The model also predicts that a tenfold decrease in the diffusion coefficient results in a tenfold increase in the predicted service life. Although laboratory estimations of diffusion coefficients are often too uncertain for precise 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 buildup of chlorides, an equation other than Eq. (5.2.4.1c) should be used due to the change in boundary conditions. Although there is no conclusive evidence for what function $\Phi(t)$ should be assigned to represent that buildup, there is some intuitive support for a linear or square root buildup 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

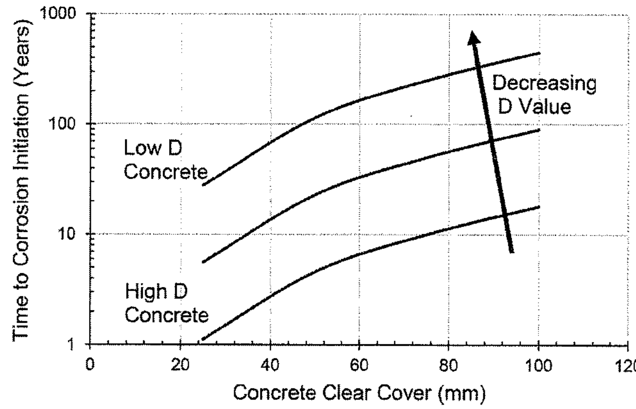

Fig. 5.2.4.1b—Schematic of time to corrosion initiation from Eq. (5.2.4.1c). (Note: 1 in. = 25.4 mm.)

$$C(x,t) = kt\left\{\left(1+\frac{x^2}{2Dt}\right)\text{erfc}\left(\frac{x}{2\sqrt{Dt}}\right)-\left(\frac{x}{\sqrt{\pi Dt}}\right)e^{-x^2/4Dt}\right\} \tag{5.2.4.1d}$$

For the case where $\Phi(t) = kt^{1/2}$, where k is a constant under a square root buildup condition, the following simplified solution should be used

$$C(x,t) = k\sqrt{t}\left\{e^{-x^2/4Dt}-\left(\frac{x\sqrt{\pi}}{2\sqrt{Dt}}\text{erfc}\left(\frac{x}{2\sqrt{Dt}}\right)\right)\right\} \tag{5.2.4.1e}$$

Equations (5.2.4.1d) and (5.2.4.1e) are most suited for evaluating airborne deicing salts applications. Additional information on models is 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, should be determined before they are used.

Probabilistic models (Engelund 1977; Hartt 2014) and computational methods (Smith 2001) for chloride ingress in concrete have also been developed.

5.2.4.2 *Sulfate attack*—A mechanistic model has been developed to predict the effect of groundwater containing sulfates on the service life of concrete (Atkinson and Hearne 1989). The model is based on:

(a) Sulfate ions from the environment penetrating the concrete by diffusion

(b) Sulfate ions reacting expansively with aluminates in the concrete

(c) Cracking and delamination of concrete surfaces resulting 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 groundwater 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/s—and is given by

$$R = X_{spall}/T_{spall} = (EB^2c_sC_0D_i)/[\alpha_0\tau(1-\nu)] \quad (5.2.4.2)$$

where X_{spall} is the thickness of the reaction zone causing the spalling; T_{spall} is the time for the spall to occur; E is Young's modulus; B is linear strain caused by a concentration of sulfate reacted in a specific volume of concrete (such as 1 mole of sulfate reacted in 1 yd^3 [m^3] of concrete); c_s is sulfate concentration in bulk solution; C_0 is concentration of reacted sulfate in the form of ettringite; D_i is intrinsic diffusion coefficient of sulfate ions; α_0 is roughness factor for fracture path; τ is fracture surface energy of concrete; and ν is 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 1989), the rate of attack for a sulfate-resistant portland cement (similar to ASTM C150/C150M Type V) was predicted to be only about 30 percent 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 cement.

5.2.4.3 *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 (James 1989). It has the form

$$dM/dt = 2.6KA(C_s - C)^\theta \quad (5.2.4.3)$$

where M is mass loss in time t from an area A; K is experimentally obtained dissolution-rate constant (linearly dependent on the flow velocities within laminar flow regimes); C_s is solution potential of water; C is concentration of dissolved material at time t; and θ is 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 0.03 in./year (0.8 mm/year) of mortar was predicted at a flow velocity of 9.8 ft/s (3 m/s), which is in reasonable agreement with the measured loss of 0.04 in./year (1 mm/year) at a flow of 9.8 ft/s (3 m/s).

5.2.5 *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). Many factors affect the service life of concrete; 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 stochastic cumulative damage models.

5.2.5.1 *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 with all service life calculations, computer analysis tools to assist with these calculations are often available and should be explored.

As is typical of any engineering material, supposedly identical concrete specimens exposed to the same conditions have time-to-failure distributions. The reliability method considers the time-to-failure distributions. By elevating the stresses that effect accelerating failure, probability of failure functions is obtained, as shown in Fig. 5.2.5.1a. 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)

$$F_i(t) = F_o(p_i(t)) \quad (5.2.5.1)$$

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. (5.2.5.1), a probability of failure stress time-to-failure (P-S-T) diagram is prepared as shown in Fig. 5.2.5.1b. 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. Deteriora-

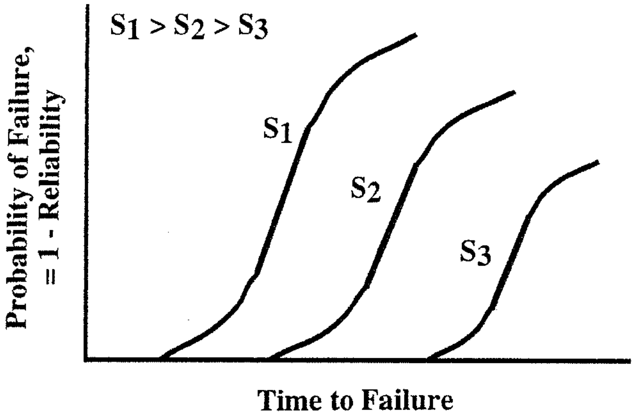

Fig. 5.2.5.1a—Probability of failure at different stress levels (Martin 1985).

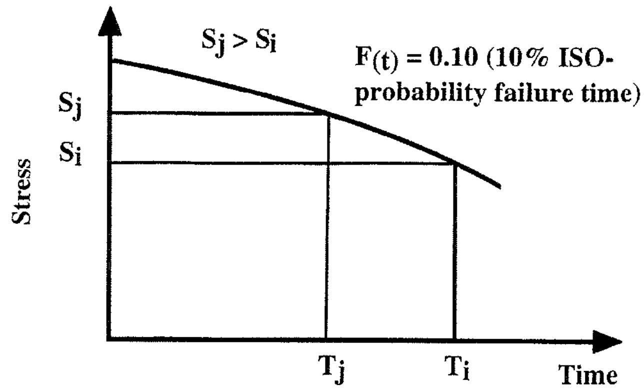

Fig. 5.2.5.1b—Probability of failure stress-to-failure (P-S-T) diagram showing 10 percent probability of failure curve (Martin 1985).

tion, which begins at the instant of stress application, is an irreversible cumulative process.

5.2.5.2 *Stochastic cumulative damage model*—Concrete structures subjected to the effects of aggressive environmental factors and mechanical loadings deteriorate with time and exhibit a certain level of variability from the mean value, which could also vary with time. An example is that for corrosion and freezing-and-thawing cycles. The effects of these factors initiate damage in concrete and steel that accumulates with time until a critical level of damage or limit state is reached, which is defined as the end of life of the structure. The sources of uncertainty are grouped into three major categories: 1) physical or inherent uncertainty in the magnitude and time-variation of environmental factors and load, as well as structural design and material parameters—for example, structure geometry, reinforcement size and layout, concrete cover, surface chloride concentration, diffusion coefficient, and concrete strength; 2) statistical uncertainty due to sampling simplifications and measurement errors such as small sample size, use of simple random variables as opposed to stochastic processes to model the different parameters and performance functions; and 3) model uncertainty as a result of the use of simple mathematical models to describe complex physical mechanisms such as a simple diffusion model for chloride ingress in concrete.

A reliable model of this time variant and uncertain performance of concrete structures is achieved with the use of stochastic processes. A stochastic process is a mathematical model that enables the capturing of both the uncertainty and parameter dependence of any given variable such as load and resistance. There are several models of stochastic processes with different levels of capabilities and complexities, such as Poisson, Markov, gamma, Gaussian, and Wiener-Levy processes (Parzen 1962; Papoulis 1965). The Markov chain model is a practical stochastic process that has been used extensively in the last decade for the modeling of deterioration of different infrastructure systems. This model is used to model the deterioration resulting from cumulative damage as first pioneered by Bogdanoff (1978) and Lounis (1999, 2000). Lounis (2000) used a seven-state Markov chain to model the time-dependent deterioration of concrete bridge structures built in corrosive environments in which State 1 corresponds to the undamaged condition of the structure (for example, just after construction), States 2, 3, 4, 5 and 6 correspond to growing damage levels, and State 7 corresponds to the failed state.

The performance of a concrete structure is discretized into a finite set of conditions or damage states from the initial construction until its failure, which can include no damage, minor damage, major damage, and ultimately failure. The basic element of the model is Bogdanoff's concept of "duty cycle," which is a repetitive period of operation in the life of a concrete structure in which the damage accumulation is assumed to occur at a constant or time-varying rate. A duty cycle is defined as one, two, or any number of years, n, during which the concrete structure is subjected to environmental factors and loads. Examples are deicing salts in winter, freezing-and-thawing cycles, and dead and live loads. The performance of the concrete structure after a duty cycle is assumed to depend only on the duty cycle itself and the damage accumulated at the start of the duty cycle; thus, it is assumed to be independent of how the damage was accumulated prior to the start of the duty cycle. These key assumptions lead to the statement that the damage process can be modeled as a discrete-time and discrete-state Markovian process (Bogdanoff 1978; Lounis 1999, 2000).

The Markov chain is based on probabilities in a transition probability matrix. This matrix is generated using historical data collected during the inspections of concrete structures. Contrary to mechanistic models, the transition matrix, and thus the cumulative damage model, is developed from a limited set of data that then is further refined using the Bayesian updating approach when more data become available.

An example of probabilistic prediction of accumulation of damage in concrete structures using this method is illustrated in Fig. 5.2.5.2, which indicates the evolution with time of the probability mass function of the damage.

Figure 5.2.5.2 shows that in the early stages of life in the concrete structure, the probability mass of the damage is near State 1 (no damage), but with aging, damage accumulates and this probability mass shifts to high damage states.

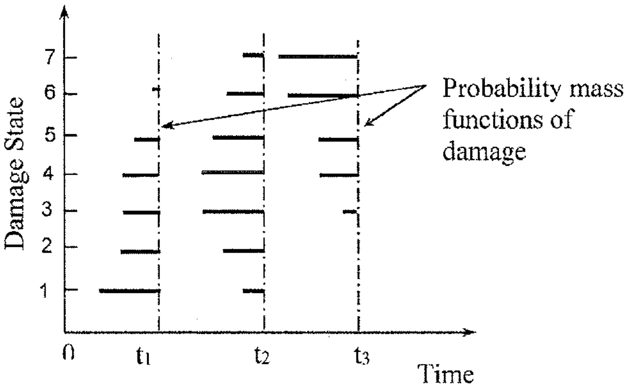

Fig. 5.2.5.2—Stochastic modeling of deterioration in concrete structures (Lounis 2000).

Ultimately, if no rehabilitation is taken, all the probability mass accumulates in the final or absorbing state, State 7.

5.3—Prediction of remaining service life of existing concrete structures

Although methods for predicting the remaining service life of existing concrete structures are basically the same as those for new structures, existing structures can have the benefit of additional information available. An example is 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 structure; identifying the cause(s) of any reinforcement or 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. This approach relies on the assumption that the environmental exposures and concrete characteristics will generally be similar over the remaining service life. Thus, it is often assumed, for example, that chloride diffusion constants and surface chloride levels can be taken as constant for the remaining service life, although this is unlikely to be strictly accurate in practice.

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.

5.3.1 *Modeling approach*—The modeling approach is illustrated by the work of Collipardi et al. (1970) and Browne (1980). For example, Browne 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. 5.2.4.1a) and assumes that the diffusion of chloride ions is the rate-controlling process. These steps assist in making predictions about the service life:

(1) Samples are obtained from a concrete structure at different depths from the concrete surface and their chloride contents determined

(2) Eq. (5.3.1) is used to obtain values of C_0 and D_{cl}

$$C(x,t) = C_0[1 - \mathrm{erf}(x/2(D_{cl}t)^{1/2}] \tag{5.3.1}$$

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

(3) 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, is calculated

(4) A chloride ion concentration of 0.4 percent, based on mass of cement, is used by Browne (1980) as the threshold value; the time it takes to reach the threshold concentration at the depth of the reinforcing steel gives the remaining service life

5.3.2 *Corrosion measurements*—The measurement of corrosion current density of steel reinforcement in concrete has been used (linear 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, 1990; Clear 1989). However, such corrosion measurements provide only instantaneous corrosion rates. Ideally, corrosion rates used to predict remaining service life include the averages of repeated measurements over a representative period, capturing seasonal changes in moisture and temperature. Also, corrosion rate measurements are highly variable due to boundary conditions, instruments used, operator skill, and degree of visible validation (Poursaee and Hansson 2009). Due to these variables, a range of estimated remaining service life should be considered.

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

$$\begin{gathered}\theta(t) = \theta_i - 0.0058 \times i_{corr} \times t \\ \theta(t) = \theta_i - 0.023 \times i_{corr} \times t\end{gathered} \tag{5.3.2}$$

where $\theta(t)$ is steel reinforcement diameter at time t, in. (mm); θ_i is initial diameter of the steel reinforcement, in. (mm); i_{corr} = corrosion rate, μA/in.^{2} (μA/cm^2); and t is time after the beginning of the propagation period, years.

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) recommended using the following relationships, which assume constant corrosion rates with time, between corrosion rates i_{corr} and remaining service life:

(a) i_{corr} less than 3.2 μA/in.^{2} (0.5 μA/cm^2)—no corrosion damage expected

(b) i_{corr} between 3.2 and 17.4 μA/in.^{2} (0.5 and 2.7 μA/cm^2)—corrosion damage possible in the range of 10 to 15 years

(c) i_{corr} between 17.4 and 174 μA/in.^{2} (2.7 and 27 μA/cm^2)—corrosion damage expected in 2 to 10 years

(d) i_{corr} exceeding 174 μA/cm^2 (27 μA/cm^2)—corrosion damage expected in 2 years or less

Note that there is significant variation of results obtained from different methods; some of these are discussed in Chapter 7 with more detail in Broomfield (2007).

5.4—Predictions for existing structures based on extrapolations

The remaining service life of a concrete structure or element is predicted from knowing its present condition and extrapolating to the time it requires for extensive repair, restoration, or replacement. 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 represented by a term k_d (Pommersheim and Clifton 1990). Climate changes each season, but usually the variation between years smooths 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, is represented by (Clifton 1991)

$$A_d = k_d t_y^n \tag{5.4a}$$

where A_d is the amount of accumulative deterioration at time t_y, years; and n is 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. (5.4a) would be $k_d(t_y - t_o)^n$, with t_o being the duration of the initiation period. In development of the approach, the term "time order" is used to avoid confusion with the order of a chemical reaction; for example, a second-order reaction that indicates two molecules react together.

The overall rate of degradation, R_d, is given by

$$R_d = nk_d t_y^{n-1} \tag{5.4b}$$

Equation (5.4c) 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. (5.4b) that

$$t_{yf} = (A_{df}/k_d)^{1/n} \tag{5.4c}$$

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, which is 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 1994). Examples of using the time order approach for predicting remaining service lives are also available (Clifton 1991; Clifton and Pommersheim 1994).

5.5—Multi-species approaches

The modeling noted above typically considered only the movement of one ion, primarily the diffusion of chlorides. The pore solution in concrete is a complicated ionic solution, so this is a simplification. This approach does not consider electroneutrality or the influence of other ionic phases in the concrete on ionic transport, although it is possible to develop equations that consider multi-species within the pore solution and their interaction.

Hydrated cementitious materials are divided into three different phases: 1) solid; 2) aqueous; and 3) gaseous. Ionic transport, which includes chloride ions, occurs in the aqueous phase of the pore space of cementitious materials. The flux of ions in the aqueous phase at the pore scale is the sum of diffusion and advection phenomena (Helfferich 1961)

$$j_i = j_i^{diff} + j_i^{adv} \tag{5.5a}$$

where the subscript i relates to a given ionic species in solution. The diffusion part of the flux comes from the gradient of electrochemical potential, which leads to the extended Nernst-Planck equation

$$j_i^{diff} = -D_i^o \text{grad}(c_i) - \frac{D_i^o z_i F}{RT} c_i \text{grad}(\psi) - D_i^o c_i \text{grad}(\ln \gamma_i) \tag{5.5b}$$

where D_i^o is the diffusion coefficient of species i in free water; c_i is the concentration of species i in solution; z_i is the valence number of the ion; F is the Faraday constant; R is the ideal gas constant; T is the temperature; ψ is the electrochemical potential; and γ_i is the activity coefficient of the ion. The first term on the right-hand side of Eq. (5.5b) is the classical Fick's first law—for example, the movement of ions under their thermal agitations. The second term on the right-hand side of Eq. (5.5b) accounts for the electrical coupling between the charged ions. The electric field $E = -\text{grad}(\psi)$ is created in solution because the ionic species have different drifting velocities. The electrical coupling will affect the various fluxes in order to maintain electroneutrality of the solution. The last term on the right-hand side of Eq. (5.5b) (chemical activity effects) accounts for the non-ideal character of the solution.

When the concentrations in solution are weak, the chemical activity coefficients tend to 1 and the solution exhibits a behavior close to the ideal, very diluted case. However, for high concentrations, as is often the case in cementitious materials, the ideal hypothesis is no longer valid; in this case, the chemical activity coefficients drop below 1.

Methods based on these concepts have been developed by Truc et al. (2000); Masi et al. (1997); Samson et al. (1999); and Marchand (2001) in their ionic transport model. Although these methods tend to be mathematically complex, they introduce the potential for better modeling of actual concrete behavior, particularly under the influence of aggressive exposure cases where a simple chloride diffusion model is insufficient.

5.6—Summary

Methods used for predicting the service lives of concrete structures 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 current state of knowledge has developed methods including accelerated testing, applying reliability and stochastic concepts, multi-species approaches, 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 probabilistic 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 are useful in obtaining data required to support the use of analytical models. They can also provide a way to make comparisons between different design or repair alternatives.

CHAPTER 6—ECONOMIC CONSIDERATIONS

6.1—Introduction

The construction of new concrete facilities and rehabilitation of existing ones require making decisions that will satisfy the physical performance requirements such as safety, serviceability and durability, and economic performance requirements. Different types of costs are incurred at varying points in time within the service life of a concrete facility, such as a bridge or building, which include the costs of initial construction, inspection and maintenance, repair and rehabilitation, demolition and replacement, environmental costs, as well as the indirect cost to the users. A relevant question arises—namely, how to compare the total costs of several competing alternatives that have different service lives—that incur different costs at different points in time. Economic analysis methods enable decision-makers to select a cost-effective alternative from a set of feasible design and rehabilitation alternatives that satisfy all physical performance requirements.

Although life cycle cost analysis (LCCA) has been in use for several decades, its adoption in decision-making regarding the selection of construction materials, structural systems at the design stage, or both, as well as in the selection of a maintenance or repair option at the maintenance management stage has been slow and its implementation limited. Several research papers, guidelines, and software applications are available to assist owners, managers, and designers perform economic evaluation of construction and rehabilitation projects (Purvis et al. 1994; Ehlen and Marshall 1996; Zayed et al. 2002; Hawk 2003; ISO/DIS 15686-5; Lounis and Daigle 2008, 2013; ASTM E917).

There are several techniques used to undertake an economic evaluation of different design and rehabilitation alternatives that include: 1) LCCA; 2) benefit-to-cost ratio; 3) internal rate of return (IRR); 4) net benefits (NB); and 5) payback (PB) method (Haveman and Margolis 1983; ASTM E833; ASTM E917; ASTM E1121; ASTM E1185; ISO/DIS 15686-5; Hawk 2003). The detailed descriptions of these methods are given in ASTM E917, ASTM E964, ASTM E1057, ASTM E1074, and ASTM E1121, respectively. The LCCA method is the simplest and most widely used economic evaluation method. Life cycle cost analysis of a concrete facility provides the equivalent in current cost terms the total cost or expenditures needed for the construction, inspection, maintenance, repair, rehabilitation, disposal, and replacement of a facility or its components over a specified period referred to as life cycle or planning horizon.

The LCCA method is a valuable technique used to assess the economic performance of constructed facilities. Life cycle cost analysis provides decision-makers with relevant information to assist with selecting a cost-effective design for new construction, or with the rehabilitation of a damaged structure. Under the LCCA decision-making framework, materials or structural designs that are deemed to be expensive initially, based on initial costs, could actually be more cost-effective and desirable over the long term. For example, high-performance concrete (HPC) (for example, incorporating silica fume) can be a more expensive material in terms of initial cost; however, its use in concrete structures provides higher strength and extended service life, which in turn can lead to lower life-cycle costs (LCC) when compared to normal concrete. Conversely, low initial cost design may require higher costs of maintenance, repair, demolition, and disposal, and could lead to higher social costs, greater environmental impacts, and severe failure consequences.

The LCCA method provides the equivalent of cash flow in either present value or annual value for each design or rehabilitation alternative over the selected life cycle or analysis period. The cost-benefit ratio (CBR), IRR, NB, or PB methods are generally used in the economic evaluation of buildings and building systems where the stream of benefits, expressed in monetary terms, is more easily identified and quantified. The benefits of developing new infrastructure systems or repairing/rehabilitating them are generally passed on to users and are, most of the time, difficult to quantify in monetary terms. Examples of this are bridges and pavements. One advantage of using the CBR, IRR, or NB methods over the LCCA is that the alternatives compared do not have to be equivalent in their objectives or requirements, although they have to be compared over the same period or planning horizon. Internal rate of return, NB, and PB methods measure the economic performance of an investment as opposed to the economic performance of a structure providing an analysis period that is often substantially shorter than the structure service life.

6.2—Life cycle cost analysis

Life cycle cost analysis is recognized as an important technique for assisting with investment decisions regarding the selection of the cost-effective design/rehabilitation option from a set of competing alternatives. The use of the initial cost analysis approach for the selection of the optimal option for design, inspection, and rehabilitation for a given reinforced or prestressed concrete structure may have serious shortcomings, as it considers only the costs of design and construction and ignores the long-term costs associated with the inspection, maintenance, repair, rehabilitation, replacement, and user costs, as well as additional costs, which can be quite high for high maintenance options. The use of LCC as a criterion for the selection of the optimal option overcomes limitations of the initial cost-based selection approach. It also allows the cost of monitoring the structure to be included in a consistent way, as this could result in proactive repairs, which could result in lower long-term costs.

The LCCA method is based on the concept of "the time value of money," which states that a dollar spent in the future is worth less than a dollar in present value terms because that dollar can be invested so that its value in the future is greater in real terms; this is even after the effects of inflation are considered. By converting future expenditures to present value cost by a process known as discounting, it is possible to compare two or more alternative designs that have different expenditures at different points in time within their respective service lives.

6.2.1 *Present-value life cycle cost (PVLCC)*—The present-value life cycle cost (PVLCC) approach provides the equivalent of cash flow in either present-value or annual-value terms for each design or rehabilitation alternative over the selected life cycle or planning horizon. The concept of PVLCC involves the determination of the costs of different alternatives for design/inspection/repair/rehabilitation/replacement in present-day monetary terms. The PVLCC approach determines the required costs in today's dollar value to carry out different activities and associated expenditures in the future. Based on the simple investment principle, it is described in the following.

If a capital or principal P is invested at an interest rate (r) per year, the interest for the first year is rP and the total amount of principal and interest at the end of the first year is $P + rP$ or $(1 + r)P$. In the second year, the interest on this is $rP(1 + r)$, and the amount at the end of the second year is $P(1 + r) + rP(1 + r) = P(1 + r)^2$. Similarly, at the end of the third year is $P(1 + r)^3$, and at the end of n years, it is $P(1 + r)^n$. The mathematical expression for the compound amount F, obtained in n years from a principal P, is therefore given by

$$F = P(1 + r)^n \tag{6.2.1a}$$

If one expresses P in terms of F, r, and n, the following expression is obtained

$$P = \frac{F}{(1+r)^n} \tag{6.2.1b}$$

P may be thought as the principal that will give the required amount F after (n) years assuming a constant interest rate r, so P is the present value of a capital F spent in year n at a discount rate r. The exponential relationship between the present and future values means that high DRs will have a considerable impact on the expenditures that will be incurred in the far future. A cash-flow diagram illustrating the relationship between P and F is shown in Fig. 6.2.1.

The LCC of a concrete structure or facility is expressed as follows

$$\mathrm{LCC} = \mathrm{DC} + \mathrm{CC} + \mathrm{IMC} + \mathrm{RRC} + \mathrm{UC} \pm \mathrm{RV} \tag{6.2.1c}$$

where DC is design cost; CC is construction cost; IMC is inspection and maintenance cost; RRC is repair and rehabilitation cost; UC is user cost; and RV is residual or salvage value. For the (RV) cost, the symbol (±) is used because in most cases, the residual value is a negative cost for the owner (for example, it is like an income); however, there are some situations where it is an additional cost for the owner—that is, the owner has to incur expenses to remove the salvage.

The costs shown in Eq. (6.2.1c) are incurred at different points in time within the life cycle of a concrete structure. Using Eq. (6.2.1b) and assuming a constant discount rate (DR), the PVLCC of a concrete structure over a given life cycle (or analysis period) T, with expenditures C_i at times t_i ($i = 1, 2, ..., T$) is given by

$$\mathrm{PVLCC} = C_0 + \sum_{t_i=1}^{T} \frac{C_i(t_i)}{(1+\mathrm{DR})^{t_i}} - R_v^d \tag{6.2.1d}$$

wherr PVLCC is single-payment present-value life cycle cost; C_0 is initial design and construction costs; $C_i(t_i)$ is i-th expenditure at time t_i (for example, for inspection, repair, rehabilitation, or all of these); DR is discount rate; T is life cycle; and R_v is discounted residual value (assumed negative) at the end of the life cycle.

Table 6.2.1 shows the impact of DR on the present value of costs incurred at different points in time.

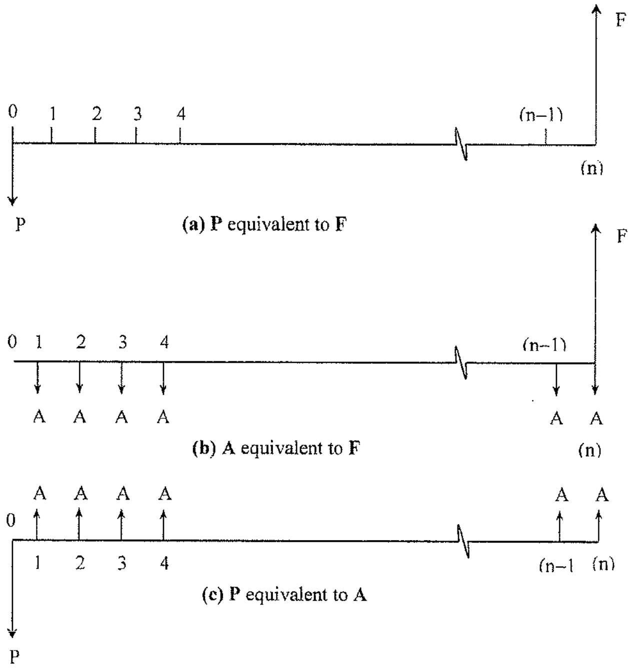

Fig. 6.2.1—Cash-flow diagrams of relationships between future, present, and annual values.

6.2.2 *Equivalent uniform annual cost (EUAC)*—The equivalent uniform annual cost (EUAC) is a uniform amount equivalent to the project net costs, considering the time value of money throughout the period of analysis. This technique is used to compare competing alternatives having replacement cycles that are not exact multiples of the period of analysis (ISO/DIS 15686-5). The annual equivalent cost is the regular annual cost that, when discounted, equals the PVLCC of the alternative (Grant et al. 1990). The selection of the lowest EUAC option is equivalent to the selection of the option with the lowest PVLCC. If a capital (A) is invested at the end of each year for (n) years, the total amount at the end of (n) years is the sum of the compound amounts of the individual investments. Assuming a constant interest rate r, the capital invested at the end of the first year will earn interest for (n – 1) years; its amount will thus be $A(1 + r)^{n-1}$. The second year's payment will amount to $A(1 + r)^{n-2}$, the third year's to $A(1 + r)^{n-3}$ and so on until the last payment made at the end of n years, which earns no interest. The total amount F is $A[1 + (1 + r) + (1 + r)^2 + (1 + r)^3 + ... + (1 + r)^{n-1}]$. The relationship between F and A is shown in Fig. 6.2.1(b). This expression of F may be simplified as follows

$$F = A\sum_{t=1}^{n}(1+r)^{t-1} \quad (6.2.2a)$$

After modification, Eq. (6.2.2a) yields

$$F = \frac{A[(1+r)^n - 1]}{r} \quad (6.2.2b)$$

From Eq. (6.2.2b), the equivalent annual uniform cost (A) is obtained as follows

Table 6.2.1—Present value (PV) of \$10,000 at different times with varying DRs

Year	Cost	PV @ 1 percent	PV @ 5 percent	PV @ 8 percent
1	\$10,000	\$9900	\$9523	\$9260
20	\$10,000	\$8195	\$3769	\$2145
50	\$10,000	\$6050	\$872	\$213

$$\text{EUAC} = A = \frac{rF}{(1+r)^n - 1} \quad (6.2.2c)$$

The uniform end-of-period payment A, which is secured for (n) years from a present investment P, is given by

$$A = \frac{rF}{(1+r)^n - 1} = \frac{r(1+r)^n P}{(1+r)^n - 1} \quad (6.2.2d)$$

The relationship between P and A is shown in Fig. 6.2.1(c).

6.2.3 *Cost per year of service life*—The cost per year of service life is a simplified method that has been used to determine the economic benefit of adding durability-enhancing materials to concrete. Although present value techniques are considered more appropriate by financiers to calculate economic benefits, it is difficult to predict interest rates 100 years into the future. As an alternative way of determining economic benefit, some engineers are simply dividing today's cost of concrete materials by the service life predicted from service life modeling techniques. There are, however, some limitations to this approach. One assumption made is that future costs are similar among all the alternatives, at least on an annual basis. This might be valid, depending on the situation that is being considered and the definition that is being used for end of design life. For example, if the end of service life is defined as the time to first corrosion, no repairs are being considered, and all the alternatives have the same maintenance requirements, this approach is valid. If these conditions are not met, then another LCC comparison method would be needed. This would be the case if, for example, the costs of repairs at different ages were considered.

This method has been used in the literature, however, to evaluate the impact of silica fume and other supplementary cementitious materials (SCMs) on the performance of concrete in the Middle East (Smith 2001). When engineers began specifying durability-enhancing materials, particularly silica fume in structures subject to extreme chloride exposure conditions of the Arabian Gulf in the early 1990s, the most immediate concern of the client and engineers was the high cost of the silica fume that was sold for eight times the cost of portland cement. The end of service life was defined as the time to the start of corrosion; maintenance during that period could be considered similar for all the alternatives under evaluation (Table 6.2.3). Table 6.2.3 also shows the estimated times to corrosion initiation and calculated costs per year of design life for the different mixture proportions considered (Smith 2001).

This analysis allowed the inclusion of initial costs and expected life in the decision-making in a simplified manner. Although the 100 percent portland cement mixtures have the lowest initial cost, they had the shortest predicted service life in this scenario, making them among the most expensive options considered. Slag cement, with a replacement level of 50 percent, was not the least costly option for this region, although it had the longest predicted service life in this scenario. Silica fume with lower replacement levels of 5 to 10 percent was the most popular choice for durable concrete in the Gulf Region in the 1990s.

6.3—Governing parameters in LCC analysis

Key parameters that govern the LCC and economic performance of a given concrete structure or facility include the service life, DR, construction and maintenance costs, social costs, and selected life cycle or analysis period.

6.3.1 *Service life*—Service life is the most relevant parameter in the LCCA of a given concrete structure and in comparing the economic performance of different design or rehabilitation alternatives. Reliable predictions of service life are critical for identifying optimal maintenance and rehabilitation (M&R) strategies that will enable the structure to achieve the selected life cycle.

6.3.2 *Discount rate (DR)*—The discount rate (DR) is used to convert the costs incurred at different times to equivalent present value costs (Eq. (6.2.1d)). It reflects the rate of interest that makes the investor indifferent to paying, or receiving a dollar now or at some point in time (ASTM E917). In the private sector, DRs reflect the interest rate that must be paid for a loan or interest forgone on another financial security. The DR, which represents an estimate of the average rate of return on private investment before taxes and after inflation, has a direct impact on the cost of any design or maintenance strategy. If the DR is low, greater significance is given to future expenditures. To the opposite, if DR is high, lesser significance is given to future expenditures. For government projects, the DR depends on several factors, such as the magnitude of investment returns, tax rates, and capital market conditions (Haveman and Margolis 1983). Treasury departments usually set the values of the DR. Examples are the Treasury Board in Canada or Government Treasury Securities in the United States. For social and environmental impact costs, however, there is a great uncertainty and serious debate on the appropriate DR to use and if these costs should be viewed the same way as other costs. Values of the DR used for assessing the owner's costs can vary between 3 and 10 percent. For social costs, the DR can vary between 0 and 10 percent. The large variability of the DR on social costs reflects the intangible nature of these costs.

6.3.3 *Construction and rehabilitation costs*—Over the life cycle of a given concrete structure or facility, different types of costs are incurred at different points in time, as shown in Eq. (6.2.1d), which depend greatly on the life cycle performance of the concrete structure. The LCC is the sum of all these discounted costs. These costs can include the initial costs of materials acquisition, transportation, construction, energy, and future costs of operation, maintenance, repair, rehabilitation, demolition, and disposal or salvage value, if any. In some instances, the environmental costs or benefits resulting from using different materials should be appropriately included in the LCC (as positive or negative costs). These costs are categorized using two different classifications according to: 1) their time of incurrence, which includes the initial costs of materials acquisition, transportation, construction, and future costs of operation, maintenance, repair, rehabilitation, replacement, demolition, and disposal or salvage value, if any; or 2) the entity that incurs the costs, which include owners, users, and third-party costs—for example, social and environmental costs.

Table 6.2.3—Cost and benefit of adding durability-enhancing materials to concrete

Mixture design	Mixture cost, $/m^3$	Time to start of corrosion, year	Cost/year of design life, $/m^3/year
OPC + 10 percent silica fume (SF)	$67	110	$0.6
OPC + 5 percent SF	$59	55	$1.1
OPC + 50 percent slag cement	$165	200	$0.8
OPC + 25 percent PF ash	$78	140	$0.55
OPC Type 1	$48	25	$1.9
SRC, Type 5	$49	15	$3.3

Notes: All mixtures have total cementitious content of 657 lb/yd^3 (390 kg/m^3). Cover to reinforcement = 3 in. (75 mm). Exposed to seawater in splash zone. Local costs as quoted by concrete supplier in August 1994.

Construction and rehabilitation costs are those associated with the design, construction, inspection, routine maintenance, repair, rehabilitation, and replacement of the concrete facility. They consist primarily of the in-place costs of materials and labor for the different actions performed on a structure within its life cycle. These are tangible costs, which are usually directly observable, and unambiguously measurable with monetary values. The design cost includes the costs of all studies, environmental and other reviews, and consultant contracts prior to solicitation of construction bids for a new facility or major rehabilitation of an existing one. The construction costs include the administrative and contract costs of the facility and ancillary facilities. Unit costs and bills of quantities are the most widely used basis for developing construction costs estimates.

The inspection and maintenance costs include the costs of periodic (visual) and nonperiodic (special) inspections of the structure and the cost of routine maintenance. Repair and rehabilitation costs refer to the costs of major repair and rehabilitation of the structure to strengthen it and improve its safety, serviceability, and functionality. These actions involve demolition, disposal, and construction activities, which are quite substantial. Replacement cost includes the cost of demolition and removal, in addition to costs similar to the costs discussed previously—for example, costs of design and construction.

6.3.4 *Social and environmental costs*—Social or user costs are those costs incurred by the users or society at large due to the poor performance or deterioration of the structure, as well as disruption of use or operation due to inspection, maintenance, repair, rehabilitation, and replacement of the structure. These costs can include accident costs (fatalities and injuries), costs of disruption of facility use, and environmental costs. Note that for some facilities, these user or social costs could be substantially larger than the construction costs. User costs and social costs are often difficult to quantify due to lack of data, inadequate understanding of true cause and effects, and difficulty to express in monetary terms. The use of multicriteria decision methods to select the design or maintenance alternative is possible when considering different types of costs that cannot be expressed in monetary terms.

6.3.5 *Life cycle*—The life cycle is the period over which the cost analysis is performed. The life cycle is selected as: 1) design life of the facility (for example, T = 75 years for a highway bridge or T = 50 years for a building); 2) design life of a given structure or facility component (for example, T = 40 years for a bridge deck); 3) a period determined by a contractual liability; 4) period of foreseeable use or occupation of the built facility before it becomes functionally obsolete; or 5) a specified investment analysis period adopted in a given organization.

6.3.6 *Residual/salvage value*—At the end of the life cycle or analysis period, some design or rehabilitation alternatives of concrete structures may not reach their end of life; for example, they still have residual service lives. The residual life is simply the difference between the service life of the design/rehabilitation alternative and the analysis period. This residual life has a residual value, which reflects the remaining useful life of the concrete structure. There can also be a terminal value of its constituents (for example, recyclable reinforcing steel/concrete) that can be added to determine the total residual value. The determination of this residual value is not a straightforward task; however, simplified estimates have been suggested (ASTM E917). A typical approach used to estimate the residual value is based on an estimate of the current replacement cost of the structure and its residual service life.

6.3.7 *Scheduling maintenance and rehabilitation (M&R) activities*—The specific timing of the different maintenance, repair, rehabilitation, and replacement actions will influence the LCC, unless the DR is very low (close to zero), in which case the present value and future value of the incurred become quasi-similar. Budgetary crises, political priorities, and other factors can delay or speed up the scheduling of specific actions on the concrete structure.

6.3.8 *Uncertainty in LCCA*—The basic principles of LCCA are applied using estimates of future performance and service life of the facility or structure, as well as estimates of present and future expenditures on construction, transportation, maintenance, repair, demolition, disposal, and user and environmental costs. As a result, the final LCC will also be an estimate. If considerable uncertainty is associated with all or some of the parameters that govern the LCC, there may be serious limitations with the use of a deterministic approach to LCCA (ASTM E1369). To overcome these limitations and consider the uncertainty of the LCCA parameters, two approaches are suggested: 1) a sensitivity analysis within a deterministic framework to assess the impact on the LCC

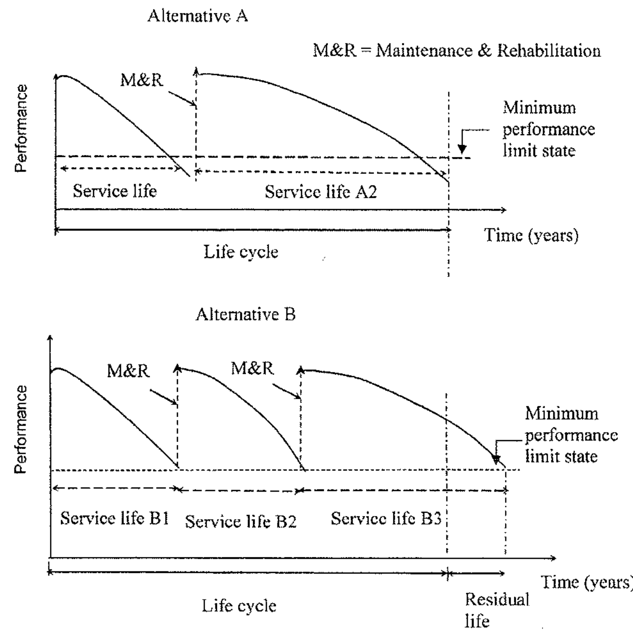

Fig. 6.4—Schematic representation of service lives and life cycle of two competing alternatives.

of varying the different parameters; or 2) a probabilistic or stochastic analysis in which the parameters are modeled as random variables with mean values, coefficients of variations, and probabilistic distributions that are estimated from available data. In such analyses, the LCC is computed as a probability distribution with expected or mean value of LCC and coefficient of variation (COV).

6.4—Service life prediction and LCCA of concrete structures

Determining reliable estimates of the LCC of a concrete structure depends greatly on the availability of reliable service life prediction models. These predict when the structure will reach unacceptable levels of deterioration that require maintenance, rehabilitation, or both. The predicted service life of the constructed or rehabilitated concrete structure should be at least as long as the design life. However, the life cycle assumed for the economic analysis is generally much longer than the service life of the structure, thus making it necessary to undertake, maintain, repair, rehabilitate, or replace the structure or some of its components to achieve the expected design life and assumed life cycle. Figure 6.4 shows two design and rehabilitation strategies that are used to achieve a specific life cycle for a given concrete structure or facility.

Figure 6.4 illustrates the importance of a reliable evaluation of the service life of a concrete structure for the planning of different M&R alternatives.

6.5—Example

In this example, the LCCA approach is used to a select a cost-effective design by comparing the LCC of two alternatives for the design of a reinforced concrete highway bridge deck that will be built in a corrosive environment due to

Table 6.5a—General information on investigated highway bridge

Bridge width	41 ft (12.57 m)
Bridge length	155 ft (47.5 m)
Deck thickness	8.9 in. (225 mm)
Isotropic reinforcement percentage for both mats	0.3 percent
Annual average daily traffic (AADT)	22,000
Annual average daily truck traffic (AADTT)	4500
Normal traffic speed	62 mph (100 km/h)

Table 6.5b—Service life parameters

Parameter	Mean value	COV*, %
Concrete cover depth	2.75 in. (70 mm)	25
Bar spacing	7.1 in. (180 mm)	5
Bar diameter	0.63 in. (16 mm)	—
Surface chloride content	10.1 lb/yd^3 (6 kg/m^3)	25
Chloride (apparent) coefficient of diffusion – NPC	0.062 in.^{2}/yr (0.40 cm^2/yr)	25
Chloride (apparent) coefficient of diffusion – HPC	0.029 in.^{2}/yr (0.19 cm^2/yr)	25
Threshold chloride content	1.18 lb/yd^3 (0.70 kg/m^3)	20
Corrosion rate	3.2 μA/in.^{2} (0.5 μA/cm^2)	20

*COV is coefficient of variation.

Table 6.5c—In-place costs of materials

Material	Unit cost
Normal concrete (NPC)	\$352 per yd^3 (\$460 per m^3)
High-performance concrete (HPC)	\$474 per yd^3 (\$650 per m^3)
Conventional (black) steel	\$0.82 per lb (\$1.80 per kg)

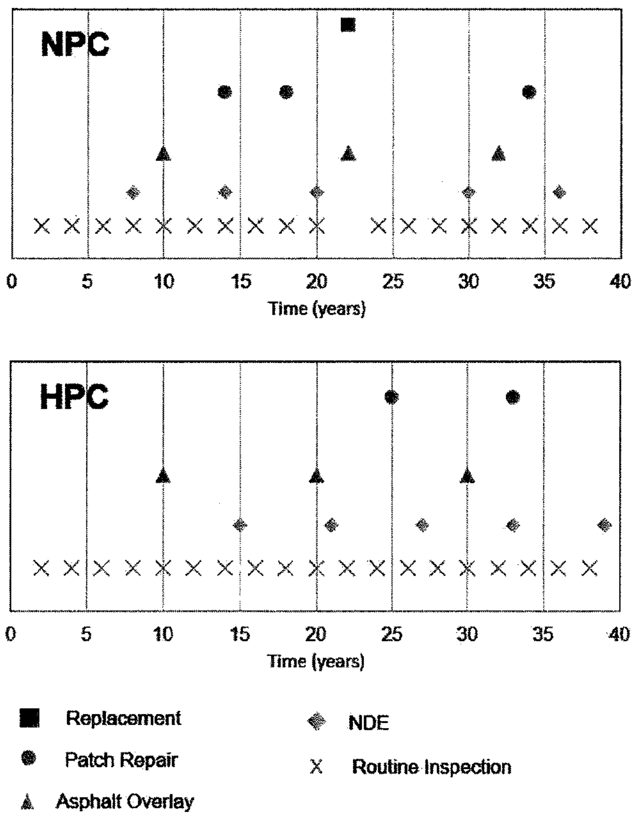

Fig. 6.5—Costs and schedule of inspection and repair activities for NPC and HPC decks over 40 years (Lounis and Daigle 2008). (Note: \$1/m^2 = \$10.7/ft^2.)

the use of deicing salts in winter months. Two bridge deck designs with two different types of concrete are compared: 1) normal-performance concrete (NPC) deck; and 2) HPC deck (Lounis and Daigle 2008, 2013). Details of the bridge are given in Table 6.5a.

The first design alternative consists of NPC with a water-cement ratio (w/c) of 0.4 and a 28-day compressive strength of 4350 psi (30 MPa). Reinforcement consists of No. 15M conventional black steel reinforcing bars with a yield strength of 58 ksi (400 MPa). The second design alternative consists of the use of HPC containing 25 percent of fly ash with a 28-day compressive strength of 6500 psi (45 MPa). Data for the service life prediction of both alternatives are given in Table 6.5b. The analysis period, or life cycle, is taken as 40 years and the DR used is 3 percent. Components of the initial construction costs (in-place costs for the steel and concrete mixtures) of the two alternatives are listed in Table 6.5b. Additional construction costs are not specified for the two alternatives, as they are considered approximately the same.

Using the data in Table 6.5b, the service life of the NPC deck is 22 years, while the service life of the HPC deck is 40 years. It is assumed that after 22 years, the damaged NPC deck is replaced with a similar type of deck; for example, normal concrete with black steel reinforcement is replaced in both top and bottom mats. Replacement costs include those for initial construction and demolition and disposal costs assumed equal to \$6.50/ft^2 (\$70/m^2). Because the end of life for the HPC deck is equal to the analysis period, its replacement is not included in the LCCA. At the end of the analysis period, the HPC alternative will have no residual service life or value. For the NPC alternative, the schedules of M&R activities for the replacement deck (Year 22 and after) are similar to those of the initial deck construction (up to Year 22). Using the data in Table 6.5c, the initial unit construction cost of NPC deck is \$12.36/ft^2 (\$133/m^2), while the initial unit construction cost of the HPC deck is \$13.57/ft^2 (\$146/m^2); for example, the HPC deck is 9.7 percent more expensive than NPC deck in terms of initial costs. However, by using the costs and times of occurrence of inspection, nondestructive evaluation (NDE), asphalt overlay, patch repair and replacement as shown in Fig. 6.5, the total unit LCC of the NPC deck is found to be \$39.68/ft^2 (\$427/m^2), while the total unit LCC of the HPC deck is \$28.25/ft^2 (\$304/m^2). The HPC deck, therefore, is 40 percent cheaper than the NPC deck.

CHAPTER 7—EXAMPLES OF SERVICE LIFE TECHNIQUES

7.1—Introduction

Ten examples representing applications of service life techniques to concrete structures or structural components are discussed in this chapter. Examples 1 through 5 were selected because of their usefulness in approach and their application to actual structures. Example 6 was selected because it illustrates the application of time-dependent reliability methods described briefly in 4.4. Examples 7 through 10 demonstrate the advancing state-of-the-art in-service life prediction, including the adaptation of available software data for use in a modeling exercise (Example 9). It is not the intent of these examples to be all-inclusive or fully comprehensive, but instead to provide guidance on how service life techniques have been used (for example, establishment of in-service inspection and maintenance strategies). Not all methods described are of equal appropriateness for all types of service life questions; consult the original papers for full details and limitations of the methods summarized. One important limitation common to many service life prediction models is whether they consider the effects of cracking, both related and unrelated to the deterioration. Neglect of the influence of cracking can severely overestimate the predicted service life, and the users should account for this when interpreting the results. Not all the predictions made in the following examples have been verified in practice. Insight from service life estimations is essential to establish life cycle costs (LCCs) for a structure and to evaluate the benefits of materials with enhanced performance characteristics. Also, decisions on using protection systems, repair materials, or demolition and reconstruction should be based on LCC estimates.

Examples 1 through 10 are summarized as follows:

Example 1—Illustrates the technique of comparing cumulative steel corrosion to concrete spalling to obtain the service life.

Example 2—Describes the challenge in evaluating the many measurements needed to characterize the condition of a structure and predict its service life.

Example 3—Describes how treating each process individually answers questions, such as when to repair and when to rehabilitate.

Example 4—Describes how an aggressive sewage environment is characterized and modeled based on the reaction efficiency of an environment with the concrete.

Example 5—Provides an illustration of calculations used to estimate service life and maintenance demands of a diaphragm wall exposed to saline groundwater.

Example 6—Illustrates the application of time-dependent reliability concepts for service life predictions.

Example 7—Demonstrates the stochastic modeling technique to predict the service life of a reinforced concrete bridge deck built in an aggressive environment.

Example 8—Describes how deterministic data are manipulated to provide probabilistic service life predictions.

Example 9—Demonstrates the use of a model to predict chloride ingress in a marine environment for concrete with different binder types.

Example 10—Demonstrates how a multi-ionic service life prediction modeling is used to estimate the chloride ingress into a concrete parking structure exposed to deicing salts.

Many service life computer programs have been developed in recent years providing convenient user-friendly analysis methods. These include Life-365™ (Ehlen et al. 2009; Ehlen and Kojundic 2014), STADIUM™ (Samson and Marchand 2007), AGEDDCA (Concrete Society 2004), and DuraCrete (Schiessl et al. 2004).

7.2—Example 1: Relationship of amount of steel corrosion to time of 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). This approach is based on an analysis that calculates the corrosion rate of steel reinforcement for each year based on several parameters, and calculates the total amount of corrosion, comparing it to the amount necessary for concrete cracking to occur. This example is based upon empirical relationships that were developed using metric units. Thus, this example is presented in metric units only.

The year that predicted cracking occurs is conservatively defined as the end of the service life of the structure.

The life of a reinforced concrete structure or structural member is 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,

$$t = Q_{cr}/q \tag{7.2a}$$

where t is the life of the structure or member; Q_{cr} is amount of corrosion to cause cracking of the concrete cover; and q is corrosion rate.

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

$$Q_{cr} = 0.590(1 + 2(c/d))^{0.85}d \tag{7.2b}$$

The corrosion rate q of the reinforcement is a function of the corrosion rate q_1 of reinforcement in concrete with a known chloride content exposed to a specified condition, corrosion rate of concrete q_2 of reinforcement in a concrete containing a known chloride 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.

$$q = q_1q_2/q_3 \tag{7.2c}$$

where

$$q_1 = [(-0.50) - 7.45N + 44.1(W/C)^2 + 66.64N(W/C)^2]d/c^2 \tag{7.2d}$$

$$q_2 = 2.54 - 0.05T - 6.76H - 22.43O - 0.97N + 0.14TH + 0.50TO + 0.01TN + 59.63HO \tag{7.2e}$$

$$q_3 = 0.55435 + 1.4027N \tag{7.2f}$$

where N is NaCl by mass of mixing water (percent) = 165 × Cl^-/W; Cl^- is chloride content in concrete, kg/m^3; C is cementitious material content, kg/m^3; W is water content per unit volume of concrete, kg/m^3; w/cm = water-cementitious materials ratio, percent/100; T is temperature, °C; H is humidity [H = (RH − 45)/100], where RH is relative humidity, percent; and O is oxygen concentration, percent/100.

A reference condition of 15°C and 69 percent RH with 20 percent 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 chloride content, determine concrete compressive strength, and estimate mixture proportions through chemical analyses.

Under conditions of constant chloride content, Eq. (7.2a) is used to estimate the life of the structure or member. For the current conditions, however, the chloride content increases with time. The life-prediction procedure used to address this includes:

Calculate corrosion rate at each year q_i, based on average chloride content at each year;

Calculate cumulative amount of corrosion at n-th year Q_{nyear} by summing q_i to n-th year, as follows

$$Q_{nyear} = \sum_{i=1}^{n} (q_i t') \tag{7.2g}$$

where for the present study, t' = 1 and n = 30. 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 percent) between the model prediction and the observed environmental condition, depending on the concrete and surrounding environmental condition, such as splash or intertidal zone.

7.3—Example 2: Use of multiple inputs to calculate life of structure

The second example describes the examination and analysis of several tunnels that are part of 162 miles (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 excessive 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.

Sixteen miles (26 km) of the tunnels were inspected between 1989 and 1993 to evaluate the general condition of the tunnels. Laboratory (chloride 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 groundwater 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 percent (by mass of cement). Carbonation depths ranged from 0.08 to 1.42 in. (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 during corrosion propagation influenced the service life of roof panels, such as structural behavior.

L is depth of concrete cover, in. (mm); D is diffusion coefficient, in.^{2}/year (mm^2/year); C_{ss} is concentration of chlorides in soil, percent; t_{co} is time of waterproofing failure, year; k is carbonation coefficient, in./year$^{1/2}$ (mm/year$^{1/2}$); d_{in} is initial diameter of steel reinforcing bars, in. (mm); r_{ch} is corrosion rate without chlorides, in./year (mm/year); r_{Cl} is corrosion rate with chlorides, in./year (mm/year); r is corrosion rate in air, in./year (mm/year); R_s is strength of the steel reinforcement, ksi (MPa); R_b is compressive strength of the concrete, psi (MPa); and M is applied bending moment of the roofing panel, ft-lb (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: 1) longitudinal cracking due to reinforcement corrosion; and 2) ultimate flexural resistance. Variations in estimates of service life for the tunnel roof panels were attributed to variations in both external conditions for time of waterproofing failure, chloride concentration in ground, operational loads, and temperature and humidity inside tunnels; and internal conditions including 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 flowchart 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 flowchart algorithm include:

1. Obtain service life estimations for all roof panels of a tunnel by calculating the probability of failure due to longi-

tudinal cracking or loss of bearing capacity due to reinforcement corrosion for each roof panel.

—For each section of steel reinforcement in a roof panel:

(a) Input data from measurements and observations

(b) Calculate values for remaining parameters identified previously (for example, L, D, C_{ss}, and k)

(c) Determine initial diameters of steel reinforcement

(d) Calculate probability of corrosion and corrosion rate due to carbonation, chloride ions, or the atmospheric conditions based on:

i. t_{sp}, time for longitudinal cracking and spalling

ii. t_{ct}, time to reach chloride threshold at the steel reinforcement section

iii. 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):

(a) Make a histogram of estimates for longitudinal cracking and spalling, and average service life and minimum service life for the tunnel

(b) Use histograms to indicate when first roof panel fails and estimate the mean life of the tunnel

2. 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.

7.4—Example 3: When to repair or rehabilitate

The third 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. This example is based upon empirical relationships that were developed using metric units. Thus, this example is presented in metric units only.

The premise of these studies lies in the fact that for a structure to degrade, several subsequent processes occur that are 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) is calculated by determining the time-to-corrosion initiation (T_i) and the time after corrosion has initiated to significant corrosion (T_{cor})

$$T_m = T_i + T_{cor} \tag{7.4a}$$

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

$$T_{rehab} = T_i + T_{cor} + T_{det} \tag{7.4b}$$

7.4.1 *Methodology development*—The amount of chloride at a given depth X and time t in a semi-infinite slab with constant surface chloride concentration C_0 is expressed as a solution to Fick's second law

$$C(x,t) = C_0\left\{1-\mathrm{erf}\left(\frac{x}{2\sqrt{Dt}}\right)\right\} \tag{7.4.1a}$$

where $C(x,t)$ is the chloride concentration at the surface of the steel at the time of interest, t. For concrete, $C(x,t)$ is 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, is 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 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 1 to 5 μA/cm² and cover depths of 51 and 76 mm, the time to cracking ranged from 5 to 1 year(s), and 10 to 2 years, respectively.

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

$$\mathrm{T} = \mathrm{ESFL} - \left(\frac{\mathrm{ID}}{\mathrm{DR}}\right) \tag{7.4.1b}$$

where ID is noticeable initial surface damage resulting from the initiation of corrosion, and DR is 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 based on surface damage.

Using data from 50 bridge decks in New York, 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

$$C_0 = 0.0110 \times \text{MAS} - 0.000189 \times \text{AADTL} + 3.349 \quad (7.4.1c)$$

where C_0 is in kg/m^3, MAS is in mm, 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 using 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 using

$$L = d - \alpha\sigma \quad (7.4.1d)$$

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. Therefore, L represents the effective cover depth for an amount of reinforcement at or less than d. Values of α result from a normal distribution and would equal 1.65 for the case where only 5 percent of cover measurements would be less than the calculated value. These results are used to calculate an effective cover depth L for varying amounts of reinforcement 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: 1) damage from spalling; and 2) total damage from spalls, delamination, and patches. From a survey of historical data on a variety of structures, it was determined that 1.4 to 5 percent spalls

Table 7.4.2a—Diffusion-cracking-deterioration model (DCDM) processes

DCDM processes	Equations to quantify process
Early damage related to construction defects	Eq. (7.4.1d)
Diffusion of chloride through concrete and initiation of corrosion at a depth of reinforcing steel equal to the initial observable level of damage, 2.5 percent	Eq. (7.4.1a) and (7.4.1c)
Corrosion of 2.5 percent of steel and subsequent spalling	Eq. (7.4a)
Damage of concrete until cumulative damage results in end of functional life (EFSL)	Eq. (7.4.1b)
Level of cumulative damage at EFSL	Eq. (7.4b)

Table 7.4.2b—Diffusion-spalling model (DSM) processes

DSM processes	Equations to quantify process
Diffusion of the chloride to a depth of reinforcing steel whose corrosion defines the effective functional service life (EFSL)	Eq. (7.4a), (7.4.1a), (7.4.1c), and (7.4.1d)
Corrosion of reinforcing steel at the critical depth resulting in surface damage defining EFSL	Eq. (7.4b) and (7.4.1b)

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 percent of the pavement in the worst traffic lane, or 5.8 to 10 percent of the pavement in the entire deck area, was reached. The study also showed that other components of a bridge, such as concrete piles, could have total damage levels between 20 and 40 percent at the end of service life.

7.4.2 *Application of the methodology*—Using the historical data, two primary approaches to predict the service life of bridge decks were evaluated: 1) diffusion-cracking-deterioration model (DCDM) (Table 7.4.2a); and 2) diffusion-spalling model (DSM) (Table 7.4.2b) (Cady and Weyers 1984). The DCDM predicts the service life based on five processes, listed in Table 7.4.2a. The DSM predicts service life based on two processes, listed in Table 7.4.2b.

For these two approaches, Eq. (7.4.1a) to (7.4.1d) are used with the primary difference between the DCDM and DSM methods being the definition of L in Eq. (7.4.1d). 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 percent of the deck surface was used. From the same data set, EFSL was defined as 40 percent 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 percent damage in the worst traffic lane is the EFSL (Weyers et al. 1993, 1994). For the substructures, 40 percent 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 these models, the degree of correlation was dependent on being able to accurately define the corrosion rate and the chloride diffusivity.

Another example of the use of Fick's second law in a hot marine environment is provided in Smith (2001).

7.5—Example 4: Use of reaction rate to calculate life of sewer pipe

The forth example addresses a variety of concrete sewer pipes that were studied in California from 1962 to 1976 (J.B. 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

$$C = 11.43k\Phi_{sw}(1/A) \tag{7.5a}$$

where C is average rate of corrosion (chemical dissolution) of concrete by acid, mm/year; 11.43 = experimentally derived constant; k is acid efficiency coefficient; Φ_{sw} is flux of hydrogen sulfide (H_2S) gas to the pipe wall; and A is alkalinity of the concrete.

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

$$\Phi_{sw} = 0.44(sv)^{3/8}j[DS](b/P') \tag{7.5b}$$

where 0.44 = empirical constant; $(sv)^{3/8}$ is energy term; s is energy gradient of waste stream; v is velocity; j is fraction of dissolved sulfide present as H_2S, a function of pH ($j = 1.0$ for pH < 4); $[DS]$ is concentration of dissolved sulfide in waste stream; and b/P' is the ratio of surface width of waste stream to exposed perimeter of pipe wall above water surface.

The flow characteristics of wastewater moving through concrete pipes of different diameters (2.26 ft to 4.9 ft [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. (7.5a) and (7.5b) 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. (7.5a) 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.

7.6—Example 5: Estimating service life and maintenance demands of diaphragm wall exposed to saline groundwater

The fifth example provides information on calculations that estimate the service life and maintenance demands of a diaphragm wall exposed to saline groundwater (9 g Cl^-/L) on one side and air on the other (Rostam and Geiker 1993; Geiker et al. 1993). The calculations assume homogeneous concrete in a 8°C environment, and were made in connection with a large Scandinavian traffic link. This example is based upon empirical relationships that were developed using metric units. Thus, this example is presented in metric units only.

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) considering the requirements of the maintenance budget.

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. It was assumed by the authors that the concrete was initially unsaturated. The saline groundwater 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.

7.6.1 *Time until steady-state moisture transport* t_l—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 (Herholdt et al. 1985). The time t_c for saturation of the pore system with saline water by capillary suction is calculated from the following (Herholdt et al. 1985)

$$t_c = z^2M \tag{7.6.1a}$$

where z is depth of penetration, assumed 100 mm, and M is the resistance number, assumed 7×10^7 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

$$t_p = (L - z - x)\rho\Delta u/q \quad (7.6.1b)$$

where L is wall thickness, 0.8 m; z is depth of penetration by capillary suction, assumed 0.1 m; x is distance between air-exposed side and evaporation zone; ρ is density of concrete, assumed 2300 kg/m^3; Δu is the difference in moisture content between saturated and nonsaturated concrete, assumed 40 percent of a total moisture content at 5 percent by mass of concrete; and q is rate of water transfer according to D'Arcy's law as follows.

The value of x was estimated to be 7 mm from

$$x = L/(1 + K_c(-p_h, 95\%/(K_p p_s(0.95 - \phi))) \quad (7.6.1c)$$

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

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

$$q = K_c(-p_h, 95\%)/(L - x) \quad (7.6.1d)$$

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 such time, chloride accumulation occurs.

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

7.6.2 *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. Considering the chloride concentration in the groundwater, 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

$$C_x = C_s(t) - (C_s(t) - C_i)\operatorname{erf}\left(\frac{x}{2\sqrt{D \cdot t}}\right) \quad (7.6.2)$$

where C_x is chloride concentration at depth x; C_s is chloride concentration at surface; C_i is initial chloride concentration; D is diffusion coefficient; x is depth; t is time; and erf is error function.

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

7.6.3 *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.

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

$$t_{sl} = t_1 + t_2 + t_3 = 17 + 30 + 20 = 67 \text{ years} \quad (7.6.4)$$

For a service life of 100 years to be achieved, remedial action should be anticipated.

7.7—Example 6: Application of time-dependent reliability concepts to concrete slab

Time-dependent reliability concepts are illustrated in example six for a hypothetical reinforced concrete slab. The results presented are drawn from research on aging of concrete structures in nuclear power plants (Mori and Ellingwood 1994b,c). The slab was designed using the requirements for flexure strength found in ACI 318-99

$$0.9R_n \; 1.4D_n + 1.7L_n \quad (7.7a)$$

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 are 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. 7.7a.

The behavior of the resistance over time should be obtained from mathematical models describing the degradation mechanism(s) present (Chapter 5). With the assumption that load occurrence is a Poisson process, the reliability function (Eq. (4.4c)) becomes (Ellingwood and Mori 1993)

$$L(0,t) = \int_0^\infty \exp\left(-\lambda t\left[1 - t^{-1}\int_0^t F_s(rg)dt\right]\right) f_R(r)dr \quad (7.7b)$$

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. 7.7a). The limit state probability or probability of failure during the interval $(0,t)$ is determined as $F(t) = 1 - 1 - L(0,t)$.

Figure 7.7b presents a comparison of limit-state probabilities for intervals $(0,t)$ for t ranging up to 60 years. Three cases are presented (Fig. 7.7a):

1. No degradation in strength (that is, $R(t)$ is a random variable—this case is analogous to what has been done

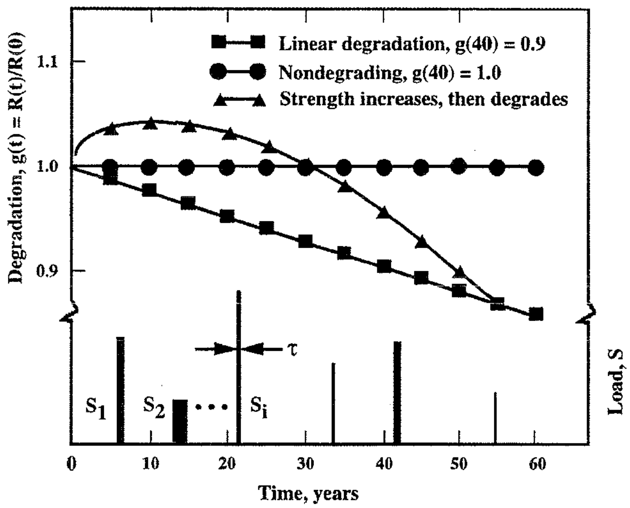

Fig. 7.7a—Mean degradation functions of one-way slab (Naus et al. 1996).

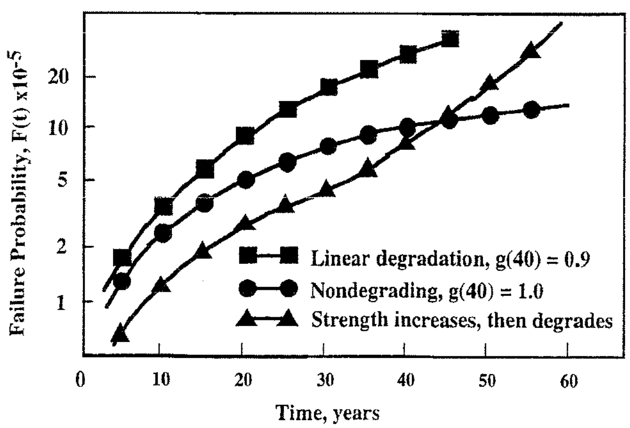

Fig. 7.7b—Failure probability of one-way slab (Naus et al. 1996).

in probability-based code work to date) (Ellingwood and Galambos 1982)

2. $R(t)$ is initially increasing with concrete maturity and then decreasing

3. $R(t)$ is decreasing linearly over time to 90 percent 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 is considered unconservative, depending on the nature of the time-dependent behavior.

Forecasts of reliability of the type illustrated in Fig. 7.7b enable an analyst to determine the 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 is 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 are developed (Mori and Ellingwood 1994b,c).

7.8—Example 7: Use of stochastic cumulative damage models to predict service life of concrete structures

The seventh example illustrates the application of the probabilistic cumulative damage model described in 5.2.5.2 to predict the service life of a reinforced concrete bridge deck built in an aggressive environment. The damage mechanisms are induced by the effects of traffic loading; corrosion; and freezing-and-thawing cycles including cracking, scaling, delamination, spalling, and localized failure. Deterioration of the deck affects the quality of the riding surface and traffic safety, deck stiffness, load distribution characteristics, deformation, and load-carrying capacity. The main factors that affect the performance and service life of concrete bridge decks include the traffic loading, quantity of deicing salts used, deck thickness, girder spacing, quality of concrete, depth of concrete cover, type and amount of reinforcing steel used, and existence of protective systems (membrane). Note that most failures of concrete bridge decks are due to loss of serviceability and functionality and not loss of strength and collapse. Depending on the maximum acceptable risk of failure and socioeconomic concerns, several definitions and values of service life are possible, which can vary from the time of damage initiation to the time of deck failure.

Most highway agencies assess the condition of bridge decks using discrete damage rating systems. These consist of mapping the observed/measured damage or distresses using visual inspection and nondestructive evaluation methods. Examples include spalls, cracks, measurements of potentials, chloride content, resistivity, and ground-penetrating radar (GPR) using a 1-to-9, 1-to-5, or 1-to-7 rating scale that describes the type, severity, and extent of different distresses. In this example, the condition of concrete bridge decks is discretized into seven damages states $D(i)$, $i = 7, \ldots 1$, which is compatible with the current practice of inspection and rating of concrete bridge decks in North America. A brief description of these ratings is given in Table 7.8. More detailed descriptions of condition ratings are found elsewhere, for example, reports from the Federal Highway Administration (FHWA 1995).

Bogdanoff's "duty cycle" is defined as the exposure of bridge decks to the damaging effects of deicing salts, freeze-and-thawing cycles, traffic load, and dead load over 1 year. For practical and simplification purposes, a constant severity duty cycle is assumed throughout the service life of the deck; hence, the transition matrix becomes time-invariant and equal to P, which yields a stationary stochastic process (Bogdanoff 1978; Lounis 2000). In addition, a unit-jump cumulative damage model is assumed. For example, no multiple state jumps are deemed possible within a year. These assumptions should be checked using the field data collected by bridge inspectors over the years for the governing parameters such as highway class, traffic, and environment. Therefore, the transition probability matrix is greatly simplified and contains only two elements per row. For example, $P_{k,k}$ and $P_{k,k-1}$, where $P_{k,k-1} = 1 - P_{k,k}$. The diagonal elements of the transition matrix are: $p_{11} = 0.7$, $p_{22} = 0.765$, $p_{33} = 0.85$, $p_{44} = 0.9$, $p_{55} = 0.98$, $p_{66} = 0.98$, and $p_{77} = 1$ (Lounis 2000, 2006).

Table 7.8—Qualitative condition rating of concrete bridge decks

Condition rating	Description
1	No damage in deck
2	Minor cracks with no spalling, scaling, delamination, or leaching; low risk of corrosion
3	Minor distresses; no spalling; medium to high risk of corrosion on less than 50 percent of deck area
4	Excessive cracking; spalling of 2 to 5 percent of the deck; heavy scaling or 20 to 40 percent of the deck is deteriorated or chloride-contaminated
5	Advanced section loss, deterioration, or spalling on more than five percent of the deck
6	Excessive full depth cracking, spalling, delamination, and active corrosion
7	Full deck failures over much of the deck area

Assuming that the initial condition of the bridge decks is given by the vector $\mathbf{P}_0$

$$\mathbf{P}_0 = [0.06\ 0.34\ 0.31\ 0.19\ 0.08\ 0.01\ 0.01]$$

Then, using Eq. (5.2.5.1), the condition of the bridge decks after 10, 30, and 50 years is shown in Fig. 7.8a.

For example, after 30 years, the deck condition is given by the following vector:

$$P30 = [0.000\ 0.000\ 0.008\ 0.089\ 0.547\ 0.273\ 0.082]$$

This example illustrates that after 30 years, 90 percent of the concrete decks are in the damage state 5 or higher (for example, damage levels 6 or 7). Using Eq. (5.2.4.1a), the time variation of the expected damage is shown in Fig. 7.8b. If the end-of-life criterion is assumed as the decks reaching the damage state 5, then the average service life of these concrete decks is approximately 21 years.

7.9—Example 8: Probabilistic service life prediction

There are many parameters that determine the service life of concrete structures. This example describes the probabilistic treatment of parameters that are used to predict the service life of concrete structures exposed to chloride. Previous work focused on the development of deterministic models based on one-dimensional (1-D) diffusion that use the concrete cover, an effective chloride diffusion coefficient, and a chloride threshold concentration for corrosion to determine the time-to-corrosion initiation. The solution to Fick's second law for diffusion in one dimension is given by (Crank 1975)

$$C(x,t) = C_0\left(1 - \operatorname{erf}\frac{x}{2\sqrt{D_c t}}\right) \tag{7.9a}$$

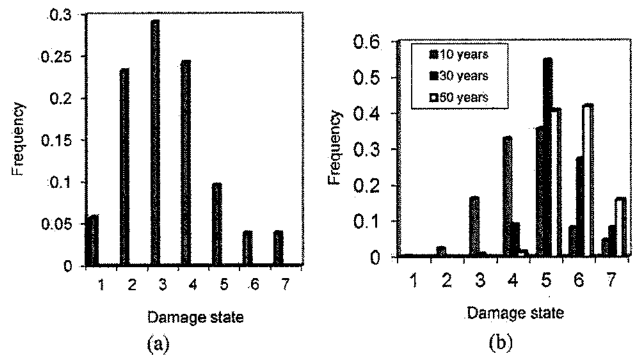

Fig. 7.8a—Prediction of bridge deck deterioration using stochastic cumulative damage model: (a) current condition of bridge decks; and (b) condition of bridge decks after 10, 30, and 50 years.

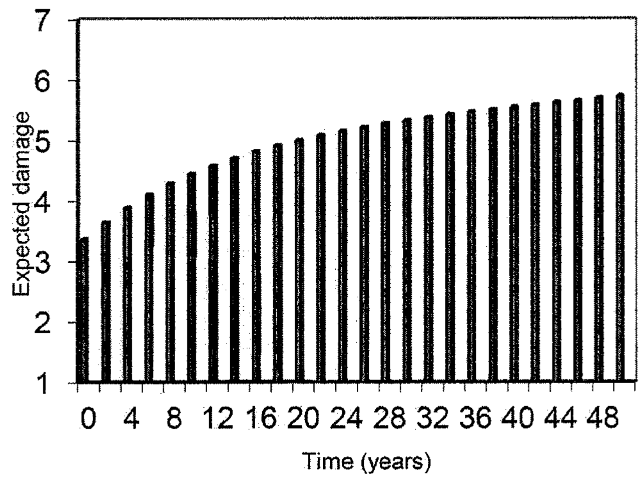

Fig. 7.8b—Time variation of average damage state of concrete bridge decks.

which forms the basis for most deterministic analyses. In most cases, a propagation period of 1 to 7 years is used to calculate or estimate the service life. The problem, at least for a Fickian model, is that the diffusion coefficient, the cover, and the threshold concentration are not uniform. The diffusion coefficient and cover vary with the geometry of the structure and exposure conditions. This variation is represented as set of values that are randomly distributed and is described, defined, or represented by a mean value and standard deviation. The question is how to use this variation to predict service life.

Schiessl et al. (2004) describes the implementation of DuraCrete, a probabilistic service life design procedure, to evaluate the service life of a tunnel structure subject to carbonation. DuraCrete (2000) is a service life design process that allows designers to perform a probabilistic service life analysis related to limit state functions. The work was the result of a consortium of 12 European Union member states.

Schiessl et al. (2004) used DuraCrete to predict the service life of the concrete cover on a cut-and-cover tunnel subjected to carbonation. The model assumes that the impact of carbonation is uniform over the entire tunnel structure and randomly distributed, and that resistance parameters are randomly distributed. A target service life of 100 years was chosen or selected based on Eurocode 1 (1999).

To avoid intervention, the reliability index β was set equal to 1.5, which corresponds to a failure probability of approximately 7 percent.

According to these criteria a limit state function is defined as

$$p\{\text{failure}\} = p_f = p\{d_c - x_c(T) < 0\} < p_0 \qquad (7.9b)$$

where p_f is failure probability, percent; d_c is concrete cover (in. [mm]); $x_c(T)$ = carbonation depth at time T (in. [mm]); T is target service life (in this example, T = 100 years); and p_0 is target failure probability, percent (in this example, p_0 = 7 percent).

The parameters d_c and $x_c(T)$ are represented by the following equations or relationships

$$d_c = d_{c,meas} + \Delta d_c \text{ and } x_c(T) = x_{c,0}(T) + \varepsilon_{xc} \qquad (7.9c)$$

where $d_{c,meas}$ is measured concrete cover (in. [mm]); Δd_c is uncertainty in the measured concrete cover; ε_{xc} is error term that represents the non-uniform carbonation process; and $x_{c,0}(T) = \sqrt{2k_e k_c (k_t R_{ACC,0}^{-1} + \varepsilon_t) C_S} \cdot \sqrt{T} \cdot W(T)$, where k_e, k_c, k_t are functions that consider the influence of the environment, including results obtained under accelerated (ACC) and natural conditions (NAC); $R_{ACC,0}^{-1}$ is inverse effective carbonation resistance of dry concrete, determined at a certain point of time t_0 on specimens with the accelerated carbonation test (ACC); ε_t is error term considering inaccuracies that occur from the ACC test method; C_s = CO_2 concentration at the surface; and W is weather function that considers the effect of mesoclimatic conditions.

To implement the analysis, each of the variables or parameters should be determined statistically. The quantification of the parameters is described in Schiessl et al. (2004) and DuraCrete (2000).

Kirkpatrick et al. (2002a) developed a probabilistic model for determining the service life of bridge decks exposed to chloride. The model uses the following input parameters that are randomly distributed: concrete cover x; surface chloride concentration C_0; chloride apparent diffusion coefficient D_c; and chloride concentration as a function of depth and time, $C(x,t)$. To determine the service life, a constant is added to the time-to-corrosion t_{crack}, which varies from 1 to 7 years depending on the reinforcement and conditions. The approach is relatively simple. The random variables are used to solve Fick's second law for diffusion in one dimension.

The concentration is determined using a Monte Carlo simulation where the desired response is determined by repeatedly solving the mathematical model. Kirkpatrick et al. (2002a) also implemented a resampling method called bootstrapping that uses data from the field measurements of bridge decks to determine the parameters for distributions used in the model. Two bootstrapping techniques were used: 1) simple; and 2) parametric. The simple bootstrap assumes that the shape of the population distribution is represented by the sample distribution. The parametric bootstrap technique uses the sample data to determine the parameters for the assumed distributions. Distributions are selected to fit the total populations of the input variables, and the values are randomly sampled from distributions. The probabilistic nature of the input parameters was determined using chloride data from 10 bridge decks in Virginia (Kirkpatrick et al. 2002a). The analysis was used to determine the time-to-corrosion and a deterministic propagation period added to estimate the time to first repair.

In a separate publication, Kirkpatrick et al. (2002b) implemented their model to determine the time to first repair and subsequent rehabilitation of concrete bridge decks exposed to chloride deicing salts. The analysis showed that the results were highly sensitive to the diffusion coefficient values, which is predictable considering the diffusion coefficient is a square root term in the denominator of the error term function for the solution to Fick's second law. The time-to-corrosion was not as sensitive to the concrete cover. For concrete covers greater than 1.5 in. (38 mm), the time-to-corrosion showed a linear dependence with cover. When the surface chloride concentration and chloride concentration for corrosion initiation or chloride threshold value are equal, the predicted time for corrosion initiation is large.

7.10—Example 9: Predicting chloride ingress in a marine environment for concrete with different cementitious materials

This example illustrates the use of a diffusion model built into a computer program that also predicts temperature profiles and thermal cracking risks in concrete components. The program was developed by Riding et al. (2013) at the University of Texas at Austin, with input for the diffusion module provided by Michael Thomas of the University of New Brunswick. Models capable of somewhat similar predictions are available for use by engineers in daily practice. One example is Ehlen et al. (2009). This example is based upon empirical relationships that were developed using metric units. Thus, this example is presented in metric units only.

The Riding et al. (2013) model in this example considers diffusion to be a function of both time and temperature. The initial (28-day) diffusion coefficient D_{28}, and the exponent m, that controls the change in diffusion with time are a function of the W/CM and type of cement used. The model uses the following series of equations with material inputs defined in percentage of the weight of the concrete

28-day diffusion coefficient D_{28} as a function of w/cm:

$$D_{28} = 2.17 \times 10^{-12} e^{(-W/CM)/-0.279} \qquad (7.10a)$$

Ratio of the diffusion coefficients for silica fume (SF) concrete and portland cement concrete D_{SF}/D_{PC}, as a function of SF content:

$$\frac{D_{SF}}{D_{PC}} = 0.206 + 0.794 e^{(-SF/2.51)} \qquad (7.10b)$$

Ratio of the diffusion coefficients for ultra-fine fly ash (UFFA) concrete and portland cement concrete D_{UFFA}/D_{PC}, as a function of UFFA content:

$$\frac{D_{UFFA}}{D_{PC}} = 0.170 + 0.829e^{(-\mathrm{UFFA}/6.07)} \tag{7.10c}$$

Ratio of the diffusion coefficients for metakaolin (MK) concrete and portland cement concrete D_{MK}/D_{PC}, as a function of MK content

$$\frac{D_{MK}}{D_{PC}} = 0.191 + 0.809e^{(-MK/6.12)} \tag{7.10d}$$

Diffusion coefficient D_t, at time t, as a function of D_{28} (or D_{SF}, D_{UFFA}, D_{MK}), the ultimate diffusion coefficient D_{ult}, and the decay coefficient m:

$$D_t = D_{28}\left(\frac{28}{t}\right)^m + D_{ult}\left(1-\left(\frac{28}{t}\right)^m\right) \tag{7.10e}$$

D_{ult} as a function of D_{28} and m:

$$D_{ult} = D_{28}\left(\frac{28}{36{,}500}\right)^m \tag{7.10f}$$

m as a function of the content of fly ash (FA) and slag (SG):

$$m = 0.26 + 0.4\left(\frac{\mathrm{FA}}{50} + \frac{\mathrm{SG}}{70}\right) \tag{7.10g}$$

Diffusion coefficient D_T at temperature T as a function of the diffusion coefficient D_{ref}, at T_{ref} = 293 K:

$$D_T = D_{ref} \cdot \exp\left[\frac{U}{R}\cdot\left(\frac{1}{T_{ref}} - \frac{1}{T}\right)\right] \tag{7.10h}$$

Large concrete blocks (0.3 x 0.3 x 0.9 m) were placed at the midtide level of the marine exposure site at Treat Island, ME. The blocks were produced from four concrete mixtures all with w/cm = 0.40, but with four levels of ground-granulated blast furnace slag (0. 25, 45 and 65 percent by mass of total cementitious materials). The blocks were retrieved after an exposure period of 25 years and chloride concentration profiles were established for each block (Thomas et al. 2008). The concentration profiles are shown in Fig. 7.10.

The seasonal variation in the average temperatures were taken from climate data for Eastport, ME, the exposure site. For tidal exposure, it is assumed that surface concentration is constant and C_s = 0.8 percent by mass of concrete, but for other conditions such as marine splash zone or highway structures, C_s will increase with age.

Table 7.10 shows the values of D_{28} and m predicted by the model for the four different concrete mixtures. The chloride profiles predicted by the model are compared in Fig. 7.10 with the experimentally-derived profiles. There is generally a good agreement between the model and experimental data.

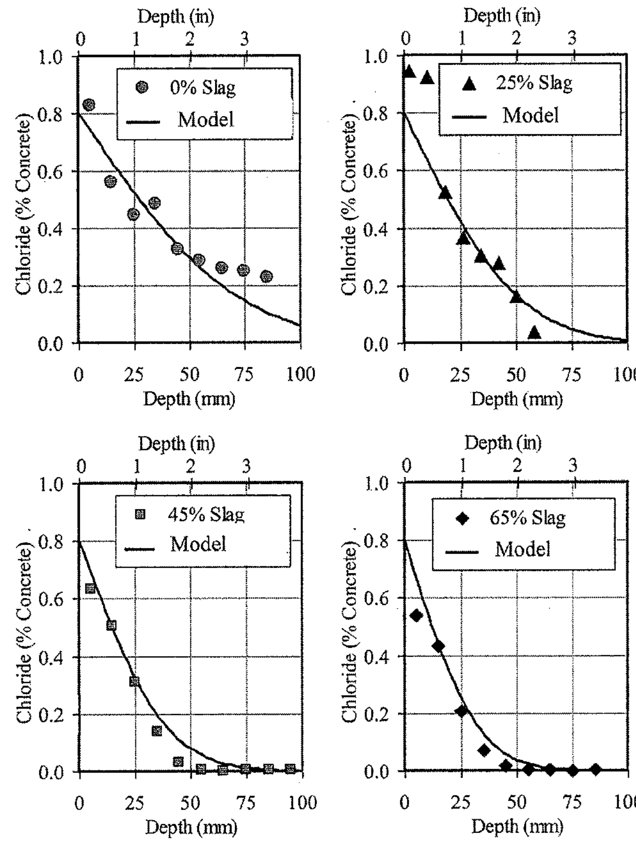

Fig. 7.10—Chloride profiles predicted by model versus experimental data for 25-year-old concrete blocks with w/cm = 0.40. (Note: 1 in. = 25.4 mm)

Table 7.10—Selected D_{28} and m values for concrete with w/cm = 0.40

Slag, percent	D_{28}, 10^{-12} in.^{2}/y (m^2/s)	m
0	0.445 (9.10)	0.260
25	0.445 (9.10)	0.403
45	0.445 (9.10)	0.517
65	0.445 (9.10)	0.600

7.11—Example 10: Multi-ionic finite element transport model (parking structure)

This section presents a durability analysis performed on a multilayered parking structure exposed to severe deicing salts. The objective is to demonstrate the accuracy of chloride ingress calculations made with a multi-ionic finite element transport model based on a sequential split operator approach that separates ionic movement and chemical reactions. The approach by Samson and Marchand (2007) is briefly summarized in 5.5.

At the time of the analysis, the structure was 20 years old. Concrete cores were extracted from the structure at different locations to measure the total chloride content in the material (ASTM C1152/C1152M). These profiles, shown on Fig. 7.11a, were taken from 7.9 in. (200 mm) thick concrete slabs. The differences between the two profiles indicate a strong variation in the exposure condition depending

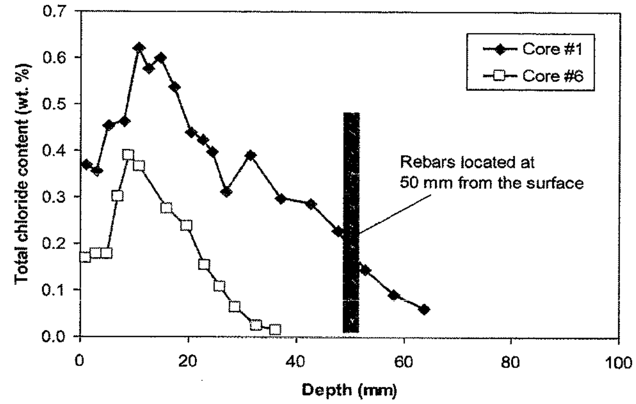

Fig. 7.11a—Chloride profiles in the parking structure after 20 years of exposure. (Note: 1 in. = 25.4 mm)

Table 7.11—Properties of concrete: 0.45 *w/cm*, ASTM Type I cement

Properties	Values	Properties	Values
Cement type	ASTM Type I	Diffusion coefficients	(E-11 m^2/s)
w/c	0.45	OH^-	17.0
		Na^+	4.3
Mixture proportions, kg/m^3		K^+	6.3
Cement	375	SO_4^{2-}	3.4
Water	169	Ca^{2+}	2.6
Coarse aggregates	925	$Al(OH)_4$	1.4
Fine aggregates	815	Cl^-	6.6
		Water diffusivity	
Cement composition, % mass		A (E-14 m^2/s)	20.0
CaO	64.5	B	80.0
SiO_2	20.8		
Al_2O_3	5.3	Hydration parameters	
SO_3	2.8	a	0.6
		α (1/s)	5.0E-03
Initial solid phases, g/kg			
Portlandite	41.4	Initial pore solution, mmol/L	
C-S-H	83.0	OH^-	114.7
Monosulfate	44.0	Na^+	83.2
Ettringite	0.1	K^+	95.9
		SO_4^{2-}	8.9
Conductivity, W/m/°C	2.0	Ca^{2+}	1.1
Heat capacity, J/kg/°C	1000.0	$Al(OH)_4^-$	0.1
		Cl^-	18.7
Porosity, %	11.8		

Note: 1 kg/m^3 = 1.69 lb/yd^3; 1 g/kg = 0.1%; 1 m^2/s = 1550 $in.^2/s$.

on the location and traffic in the structure. The top of the reinforcing steel in the concrete were located at approximately 2 in. (50 mm) from the surface. No apparent signs of corrosion could be observed on the slab at the time.

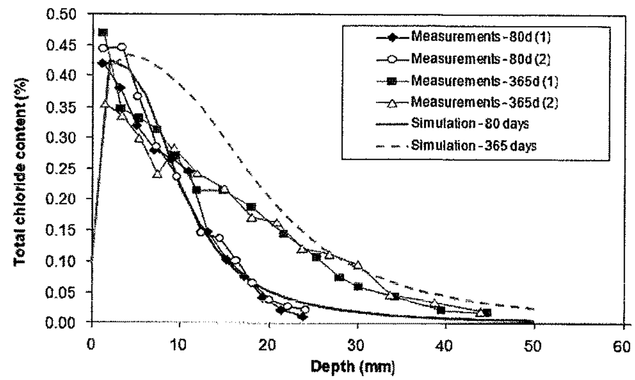

Fig. 7.11b—Validation of the transport parameter on the basis of chloride profiles at 80 and 365 days from immersion tests. (Note: 1 in. = 25.4 mm)

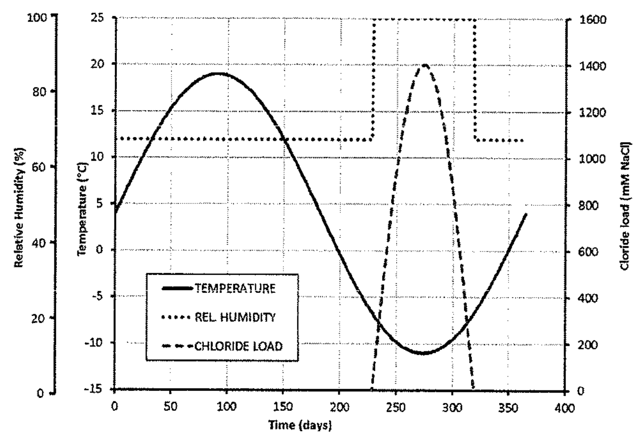

Fig. 7.11c—Boundary conditions on top of the concrete slab for 1 year.

Based on civil engineering practices at the time, it was assumed that the concrete was made at a 0.45 water-cementitious materials ratio (*w/cm*) using a standard ASTM Type I cement without SCMs. The parameters used for the chloride ingress calculations are summarized in Table 7.11 and were evaluated on lab-prepared concrete samples. The test methods used to obtain these parameters are detailed in the work by Samson and Marchand (2007). The data were validated by reproducing the measurements from an immersion test, where concrete samples are exposed for various durations to sodium chloride (NaCl) solutions. The results of the validation simulations are shown in Fig. 7.11b.

The temperature and humidity boundary conditions were obtained from the local weather service. The average temperature for the location is 39°F (4°C) with 29°F (16°C) average amplitude. The average RH is 70 percent and was considered constant. The exposure to chloride was more problematic because no data could be found on this topic. To circumvent this, brine was collected on top of slabs on a regular basis during two consecutive winters in the parking structure. Data obtained illustrated the irregular distribution of chloride concentration. To simulate the application of NaCl, it was assumed that chloride has been applied for 50 days during winter, for example, when temperature reaches

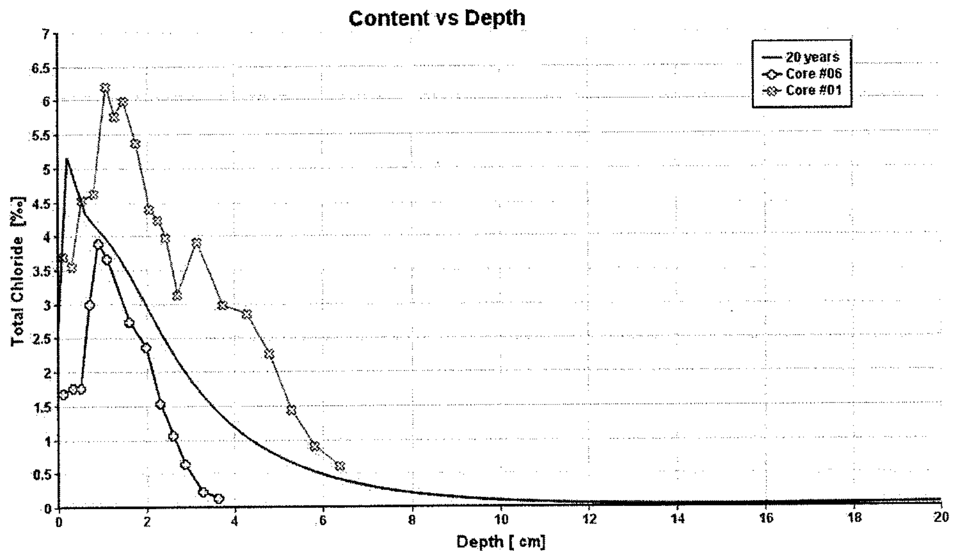

Fig. 7.11d—Chloride ingress simulations after 20 years. (Note: 1 in. = 2.54 cm)

its minimum value. It is also assumed that during the period, the top of the parking slab is wet (100 percent RH). During the remainder of the year, the concrete slab surface is free of chloride and exposed to the ambient 70 percent humidity, which causes the material to dry. The boundary conditions for one full year are summarized in Fig. 7.11c.

The simulation results are shown on Fig. 7.11d and 7.11e. Using the parameters in Table 7.11, the simulation provides a chloride profile that falls between the measured data for the two cores. The simulation result can thus be attributed to an average exposure case. To estimate the time-to-corrosion, the chloride content at the reinforcement depth (2 in. [50 mm]) were also determined. Given a chloride threshold of 0.05 percent by mass of concrete for black steel (McDonald et al. 1998), a simulation made with the multi-ionic approach predicts that the corrosion process will begin after 15 years for this type of steel. Given that the first signs of corrosion are apparent 2 to 7 years after corrosion is initiated and that no signs of corrosion were observed on the structure when the coring was performed, the predictions made with the multi-ionic approach provide a realistic view of the structure's state after 20 years (Weyers et al. 1994). The predictions indicate that corrosion signs are about to be apparent in certain locations of the structure.

This example presented a multi-ionic model to predict the ingress of chloride in concrete structures exposed to deicing salts. Contrary to a simplified approach based on a single chloride conservation equation, the model incorporates the effect of ionic coupling, wetting/drying cycles, temperature variations and multiple chemical reactions. Also, time-dependent boundary conditions based on a one-year exposure cycle are considered. The analysis of a 20-year old parking structure case showed that this new approach provides an accurate estimation of the service life of concrete structures.

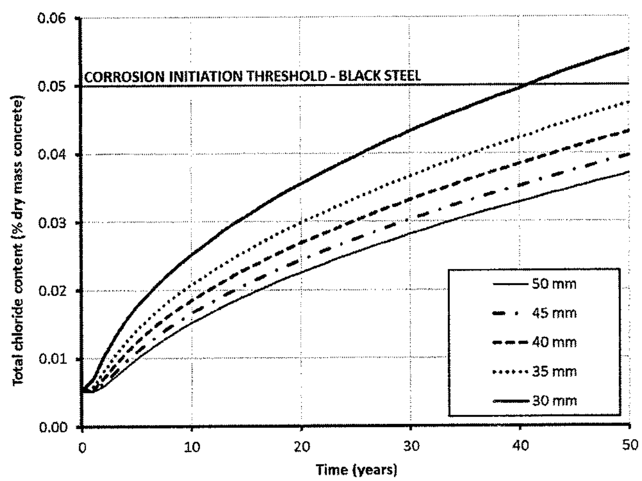

Fig. 7.11e—Chloride content at various reinforcement depths. (Note: 1 in. = 25.4 mm)

CHAPTER 8—ONGOING WORK AND NEEDED DEVELOPMENTS

8.1—Introduction

In the engineering of modern concrete structures, it is no longer unusual that design service life is included as a requirement in the design process. Thus, rather than simply specifying that the structures should be durable and of low cost, ISO 2394 specifies that the service life of a building of average importance should be 50 years. Other owners may require longer service lives (Strategic Highway Research Program (SHRP) (SHRP 2; RILEM TC 230PSC 2015). Significantly longer design service lives have also been specified for some high-importance structures (Connal and Berndt 2009; Nanukuttan et al. 2015). In addition to specified requirements, the ability to accurately predict service life is important for judging the sustainability of different engineering solutions on a level playing field. It is within this context that this report has been written.

Design and construction currently consists of seven components (Sommerville 1986; Polder et al. 2011; RILEM TC 230PSC 2015):

(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
(7) Minimum levels of maintenance

Provisions for durability in the past have primarily been addressed under components (5) and (6). With few exceptions, performance criteria have not often 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 service life prediction (Frohnsdorff and Masters 1990; Nanukuttan et al. 2015).

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, refining mathematical models depicting aging and degradation, improving the understanding of the effects of microclimates on long-term behavior, synthesizing the interaction of physical loading and environmental degradation, and incorporating the beneficial contributions of prudent inspection, maintenance, monitoring, and rehabilitation into the service life prediction process. In addition, more work is needed on modeling the mechanical process of corrosion, developing more reliable propagation period estimates, and recommending service life with the use of nonmetallic reinforcement.

The following is information regarding the durability aspects of service life prediction for the repair of concrete structures and the design of new ones.

8.2—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; Stipanovic et al. 2010; Andrade et al. 2004). 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; Gulikers 2011; Alexander et al. 2001), and progressive changes in the physical and chemical nature of concrete are well understood under such conditions. Using this information to develop criteria for service life prediction is far from complete.

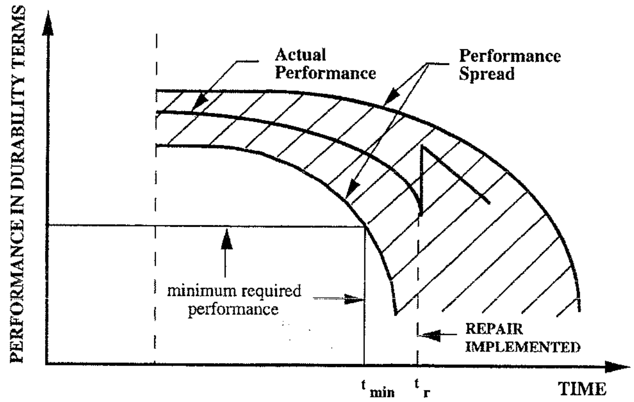

Fig. 8.2—Loss of durability with time (Sommerville 1986).

A problem with a comparative approach such as this is that each concrete structure is unique because of variability in materials, geometry, environmental exposure and construction practices. Also, over the years, the properties of the concrete materials have changed. Feedback from assessments of performance in practice increases the validity of this approach. An important aspect in the development of designs for durable structures is that a database be developed on measurements of performance of service and environmental influences. The database should 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.

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 account for uncertainties. Figure 8.2 presents an illustration of the relationship between performance, minimum required performance, and time (Sommerville 1986; Bickley et al. 2012; Bognacki et al. 2012). Relationships of these types would permit a systematic approach 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 to repair or rehabilitate the structure and what procedures to use. The effect of a repair or rehabilitation procedure on service life is also illustrated in Fig. 8.2.

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/EURAM 1998; RILEM TC 230-PSC 2015). 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;

Basheer et al. 2000) and pavements (AASHTO 1985), and in the assemblage of performance data (Philipose et al. 1991; Parrott 1987; Castellote et al. 2001). Work was initiated to develop 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, 1994c). 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 service life prediction (Frohnsdorff and Masters 1990; Gulikers 2011; Andrade 2006).

8.3—Needed developments

Modeling the service life of reinforced concrete structures has a long history and a vast body of knowledge developed. Work is still needed to improve state-of-the-art and more widely applicable models. Some areas that require further development are:

(a) The influence of cracks on deleterious material transport within the concrete

(b) Effect of the nature (width, depth, propagation) of cracks on corrosion rates and type (pitting versus general) and propagation process

(c) Influence of macro- and micro-environment on behavior

(d) Modeling the ongoing influence of repairs to the structure

(e) Effect of very long design lives (for example, 300 years) on the applicability and validity of models

(f) Further field validation of models

(g) Interaction of multiple deterioration mechanisms occurring simultaneously, including possible synergistic behaviors

(h) Modeling alkali aggregate reaction and its influence on service life

(i) Improvements in corrosion rate modeling for various exposure conditions

(j) Incorporation of sustainability and green parameters in durability design using service life modeling

(k) Development of practical tools for practicing engineers to evaluate service life and implement in specifications

CHAPTER 9—REFERENCES

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