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Report on Service Life Prediction

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Report on Service Life Prediction

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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 conjunction with life cycle cost analyses (LCCA) (Zhang 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 (CO₂) 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 similar service conditions without repairs.

2.2—Notation

A = annual capital invested (6.2.2)	$F_i(t)$ = life distribution at the i -th elevated stress level
A = alkalinity of concrete (7.5)	H = humidity
A_d = amount of accumulative deterioration	ID = noticeable initial surface damage resulting for initiation of corrosion
A_{df} = amount of damage at failure	i_{corr} = corrosion rate
B = linear strain caused by a concentration of sulfate reacted in a specific volume of concrete	j = fraction of dissolved sulfide preset as H_2S , as a function of pH
C = concentration of dissolved material (5.2.4.3)	j_i = flux of an ion i in solution
C = cementitious material content (7.2)	j_i^{adv} = flux of an ion i in solution due to advection
C = average rate of corrosion of concrete by acid (7.5)	j_i^{diff} = flux of an ion i in solution due to diffusion
C_0 = concentration of reacted sulfate in the form of ettringite (5.2.4.2)	K = experimentally obtained dissolution-rate constant
C_0 = initial design and construction costs (6.2.1)	K_c = transport coefficient for concrete
C_0 = surface chloride concentration (7.4.1)	K_p = transport coefficient for pasts
Cl^- = chloride content in concrete	k = acceleration factor (5.2.3.1)
C_s = solution potential of water (5.2.4.3)	k = carbonation coefficient (7.3)
C_s = chloride concentration at surface (7.6.2)	k = acid efficiency coefficient (7.5)
C_s = CO_2 concentration at surface (7.9)	k_e, k_c, k_t = functions that consider the influence of the environment, including results obtained under accelerated and natural conditions
C_{ss} = concentration of chloride in soil	k_f = coefficient related to environmental conditions
C_t = time-dependent chloride concentration	L = thickness of concrete element (5.2.4.1)
$C(x,t)$ = chloride concentration as a function of depth and time	L = depth of concrete cover (7.3)
c = concrete cover	L = amount of reinforcement at or below a given cover depth (7.4.1)
c_b = bound chloride ion concentration	L = wall thickness (7.6.1)
c_f = free chloride ion concentration	L_n = code-specified live load
c_i = chloride ion concentration at the depth of reinforcement (5.2.4.1)	M = mass loss in time t from an area A (5.2.4.3)
c_i = concentration of species i in solution (5.5)	M = applied bending moment of the roofing panel (7.3)
$c_i(t_i)$ = i -th expenditure at time t_i	M = resistance number (7.6.1)
c_s = sulfate concentration in bulk solution	m = change in chloride apparent diffusion coefficient (decay coefficient)
c_0 = chloride ion concentration at outside surface of concrete	N = number of freezing-and-thawing cycles damaging a laboratory specimen (5.2.3.2)
D = apparent diffusion coefficient (5.2.4.1)	N = NaCl mass of mixing water (7.2)
D_{28} = 28-day diffusion coefficient	n = number of years (5.2.5.2)
D_c = apparent diffusion coefficient (7.9)	n = time order (5.4)
D_i = intrinsic diffusion coefficient of sulfate ions	o = oxygen concentration
D_i^0 = diffusion coefficient of species i in free water	P = freezing-and-thawing resistance index obtained by the Deutscher Beton Verein (DBV) freeze-salt test (5.2.3.2)
D_{MK} = diffusion coefficient for metakaolin concrete	P = principal or capital, present value (6.2.1)
D_n = code-specified dead load	p_f = probability of failure
D_{PC} = diffusion coefficient for portland-cement concrete	p_i = time transformation function
DR = discount rate (6.2.1)	p_o = target failure probability
DR = deterioration rate (7.4.1)	p_s = saturated vapor pressure
D_{SF} = diffusion coefficient for silica fume concrete	Q_{cr} = amount of corrosion to cause cracking of the concrete cover
D_T = diffusion coefficient at temperature T	Q_{year} = cumulative amount of corrosion
D_{UFFA} = diffusion coefficient for ultra-fine fly ash concrete	q = corrosion rate (7.2)
D_{ULT} = ultimate diffusion coefficient	q = rate of water transfer (7.6.1)
$D(i)$ = damage state	R = ideal gas constant
$[DS]$ = concentration of dissolved sulfide in waste streams	$R_{ACC,O}^{-1}$ = inverse effective carbonation resistance of dry concrete, determined at a certain point of time t_0 on specimens with the accelerated carbonation test
d = diameter of reinforcing bar (7.2)	R_{AT} = rate of degradation in accelerated tests
d = design cover (7.4.1)	R_b = compressive strength of concrete
d_c = concrete cover	R_s = strength of steel reinforcement
$d_{c,meas}$ = measured concrete cover	R_d = overall rate of degradation
d_{in} = 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	

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 concrete body.

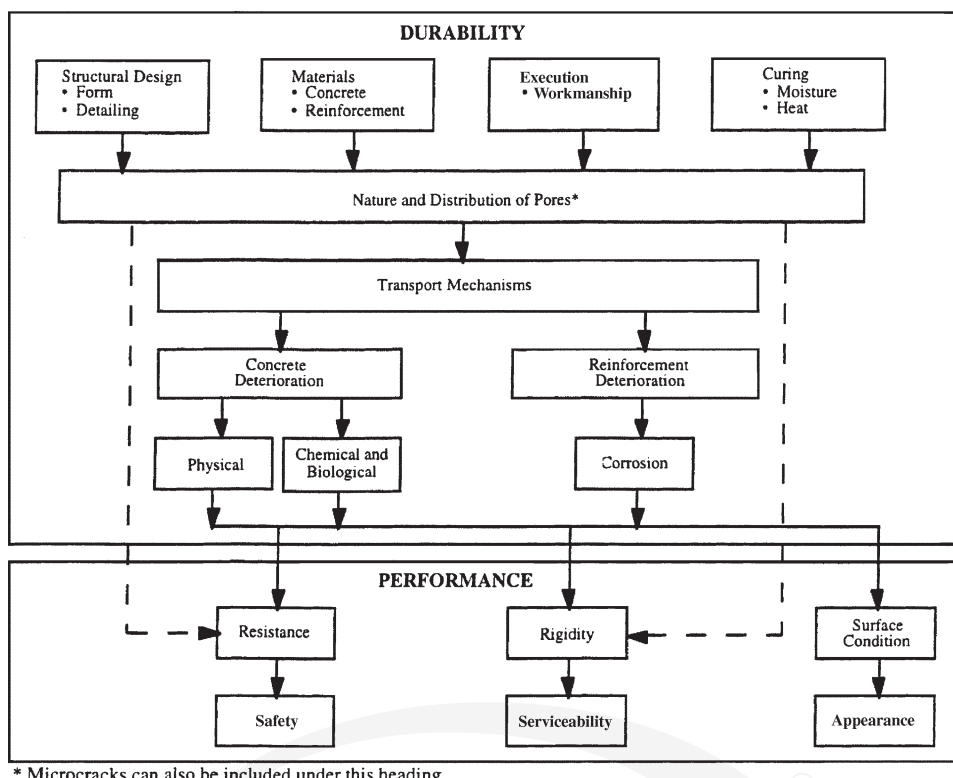


Fig. 3.2—Relationships between the concepts of concrete durability and performance (CEB 1992).

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/yd ³ (kg/m ³)				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 (C₃A) 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 C₃A is attacked by sulfates, the concrete vulnerability can be reduced by using cements low in C₃A, such as **ASTM C150/C150M** Types II and V sulfate-resisting cements. Under extreme conditions, supersulfated slag cements such as **ASTM C595/C595M** Type III 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. 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

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

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; Sturup 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; Sturup 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 structure. Mercury-intrusion porosimetry is one method that deter-

mines 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

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.

and form placement; reinforcement placement; and concrete material procurement, batching, mixing, delivery, and placement.

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.

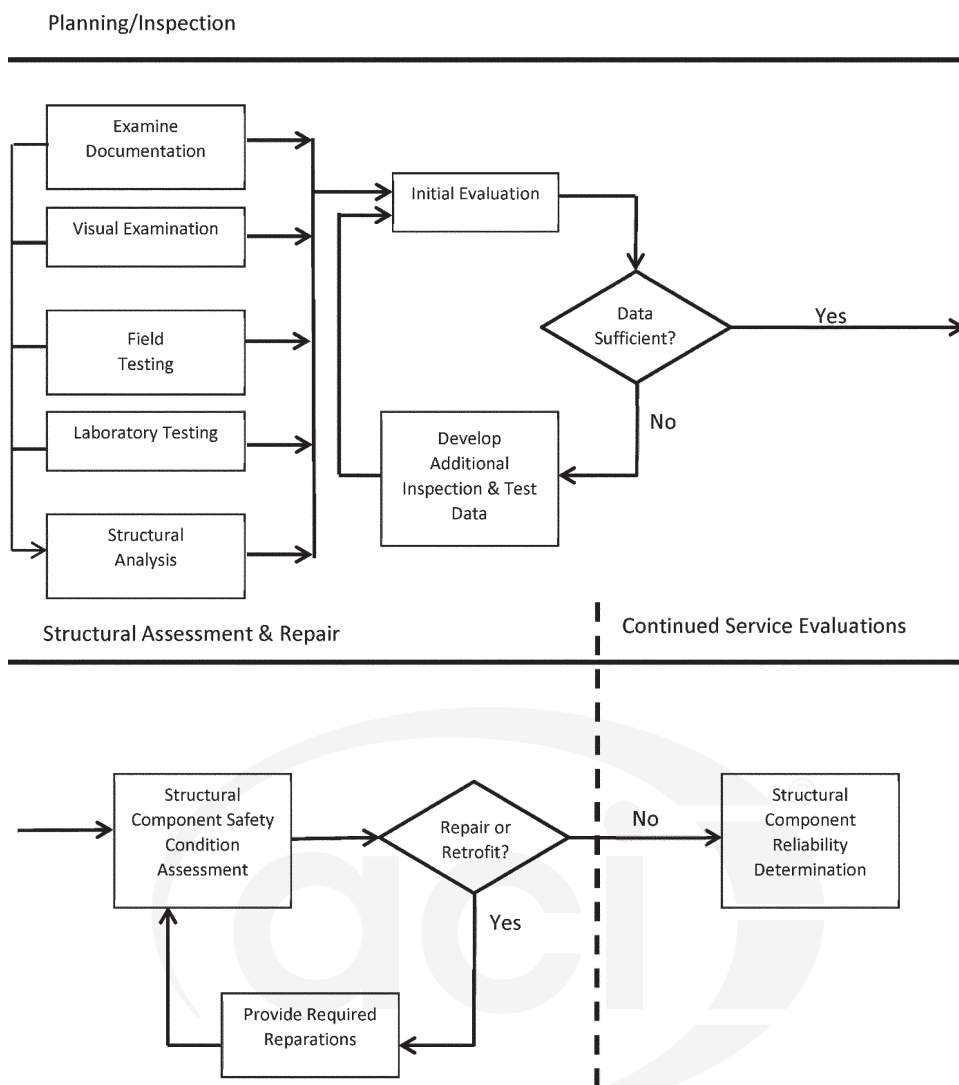


Fig. 4.1—Concrete component evaluation methodology (Naus and Oland 1994); adaptation of a procedure presented in Rewerts (1985).

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 current 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		Comment
	Primary	Secondary	
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).