Draft

Service Life Design Calculations

Example

October 27, 2015

The second Strategic Highway Research Program (or SHRP2) is a national partnership of key transportation organizations: the Federal Highway Administration, American Association of State Highway and Transportation Officials, and Transportation Research Board. Together, these partners are deploying products that will help the transportation community enhance the productivity, boost the efficiency, increase the safety, and improve the reliability of the nation's highway system.

This report is a work product of the SHRP2 Solution, *Service Life Design for Bridges* (R19A). The product leads are Matthew DeMarco, Federal Highway Administration, and Patricia Bush, American Association of State Highway and Transportation Officials. This report was authored by Anne-Marie Langlois, PE, COWI North America, with support from Mike Bartholomew, CH2M *Service Life Design Team* Leader.

Contents

Figures

Tables

Definitions

^oC	degrees Celsius
^oF	degrees Fahrenheit
AAR	alkali-aggregate reactions
AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
ASTM	American Society for Testing and Materials
Cl^-	Chloride ions
Cls	Chloride surface concentration
CO_2	Carbon Dioxide
DEF	delayed ettringite formation
FA	fly ash
ft	feet
GGBS	ground granulated blast furnace slag
in^2/s	square inches per second
kg/m^3	kilograms per cubic meter
lb/yd^3	pounds per cubic yard
m^2/s	square meters per second
mg/L	milligrams per liter
mm	millimeters
MPa	megapascal
OPC	ordinary Portland cement
O_2	Oxygen
psi	pounds per square inch
SI	International System of Units

1.0 Introduction

This document presents the background information for the service life design charts accompanying this report. Calculations were performed in accordance with *fib* Bulletin 34, *Model Code for Service Life Design* [1]. The choice of input parameters is specific to the SHRP2 *Service Life Design for Bridges* (R19A) project.

The input parameters used in the *fib* model are expressed in the International System of Units (SI units), and therefore, this report includes both English units and SI units, where applicable.

This document is not meant to teach the reader how to do a full durability assessment. The reader should refer to the *fib* Bulletin 34, *Model Code for Service Life Design* for additional background information.

2.0 Methodology

2.1 Probabilistic Design Basis

The basic requirement of performance-based service life design is that the "predefined limit states should not be exceeded during the design life with an adequate degree of reliability."

The procedure to check such a requirement can be subdivided into a number of steps. First, the limit states to be considered should be formulated. Next a model describing the degradation process and the exceedance of the limit state in terms of structural and environmental properties is used. Finally the various uncertainties are quantified and a target for the failure probability is chosen. The target depends on the consequences of exceeding the limit state: collapse requires a higher safety target than cracking or spalling.

In the theory of structural reliability, the limit state function Z is usually formulated as the difference between a resistance R and a load effect S:

$$Z = R - S \qquad (1)$$

The resistance R may be, for instance, the fully plastic moment of a beam and the load effect S the bending moment in the central cross section of the beam due to self-weight and imposed load. In durability design, the resistance may refer to the critical chloride concentration (start of corrosion) and the load effect to the actual concentration at the reinforcement bars.

In most cases R and S vary in time (see Figure 1):

$$Z(t) = R(t) - S(t) \qquad (2)$$

Failure occurs if the load exceeds the resistance at one or more points in time: *no failure occurs* if Z is positive for all points in time t in the interval [0,T] under consideration (the reference period):

$$\{\text{no failure in } [0,T]\} = \{Z(t) > 0 \;\forall\, t \in [0,T]\} \tag{3}$$

Where "$\forall t \in$" means "for all points t in." As the probability of failure is one minus the probability of no failure, the following equation can be established: $P_f, T = 1 - P\{R(t) > S(t) \;\forall\, t \in [0,T]\}$

(4)

If Z is a monotonic increasing function of time, only the value of Z(t) at the end of the interval is important equation (4) can be simplified to:

$$P_f, T = P\{R(T) - S(T) < 0\}$$

(5)

In order to check whether the design has an adequate reliability, the failure probability has to be compared with some target value: So finally the durability requirement can be formulated as:

$$P_f, T = P\{R(T) - S(T) < 0\} < P_{target}$$

(6)

Formula (6) may next be simplified into a design requirement based on characteristic values and load and material margins, similar to the design formulas for standard structural design.

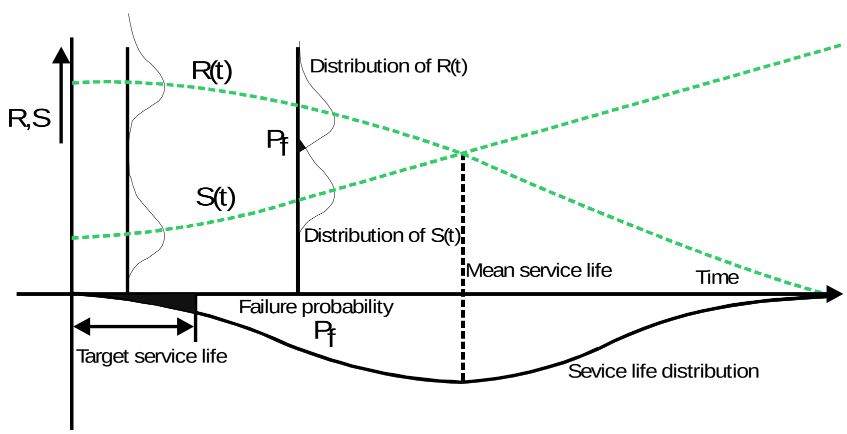

Figure 1: Failure Probability and Target Service Life (Illustrative Presentation)

2.2 Deterioration Modeling for Chloride Penetration

fib Bulletin 34 uses Fick's 2nd law of diffusion to model the chloride ingress into the concrete. The limit state equation is defined as:

$$c(x,t) = c_s - (c_s - c_0)\left(1 - \mathrm{erf}\left[\frac{x - \Delta x}{2\sqrt{D_{app,c}t}}\right]\right), D_{app,c} = \left(\frac{t_0}{t}\right)^a D_{rcm,0} \tag{7}$$

where c(x,t) denotes the chloride concentration at the time t at the distance from the surface x, where c_s denotes the chloride surface concentration, c_0 is the initial chloride concentration, Δx is the depth of the convection zone (transfer function), erf is the error function, a is an age factor. $D_{app,c}$ is the apparent diffusion coefficient of chlorides through concrete diffusion coefficient at the time t = t_0 and $D_{rcm,0}$ is the chloride migration coefficient measured using NT Build 492 [2].

2.2.1 Limit State Function

The defined limit state is the depassivation of the reinforcement (corrosion initiation), i.e., corrosion is initiated when the chloride concentration around the reinforcement exceeds a critical chloride concentration. The limit state function, which is defined as less than or equal to zero if, and only if, corrosion initiation occurs, can be written

$$g(z,t) = c_{cr} - c(x,t) \tag{8}$$

where x denotes the cover thickness, c_{cr} is the chloride threshold of the reinforcement, and z is the vector of stochastic variables, i.e., the concrete cover thickness, surface concentration, diffusion coefficient, etc.

The limit state modeling the risk of chloride-induced corrosion is:

$$g(x,t) = d_c - 2\mathrm{erf}^{-1}\left(1 - \frac{c_{cr}}{c_s}\right)\sqrt{k_t k_e k_c D_{rcm,0}\left(\frac{t_0}{t}\right)^a t} \tag{9}$$

d_c : Concrete cover
k_t : Test factor (set as 1.0, constant)
k_e : Environmental factor (set as 1.0, constant)
k_c : Curing factor (set as 1.0, constant)
t_0 : Reference time (28 days)
$D_{rcm,0}$: Chloride migration coefficient
t: Time
a: Age factor

2.2.2 Reliability Index

The reliability index can be defined based on Table A2-2 in *fib*. For serviceability limit states, the reliability index is set to 1.3, i.e., a probability of failure P_f ~ 10%.

2.2.3 Probabilistic Analysis

An analysis of the probability of initiation of corrosion is based on a probabilistic model of the variables, e.g., all variables in the limit state function have to be statistically quantified (mean, standard deviation, and type of distribution function). The probabilistic analysis is performed using a structural reliability analysis program system.

The different design steps and related input parameters of *fib* Bulletin 34 modeling are illustrated in Figure 2 :

i. Identification and quantification of the environmental exposure of the different structural members and their location.

   It is assumed that the main potential deterioration risk for the bridges is chloride induced reinforcement corrosion. With regard to chloride induced corrosion the following different exposure classes have been investigated:

   - Splash / Deicing Salts Spray;
   - Buried / Submerged; and,
   - Atmospheric.

ii. Determination of the design quality of the concrete with respect to its design penetrability for the aggressive substances and their concentrations, as identified from the environmental exposure.

iii. Definition of service life and acceptance criteria.

Corrosion initiation is defined as the nominal end of service life. The corrosion propagation period is not taken into account. In the present case, a 100-year service life is targeted.

This means there is 90% probability that corrosion has not initiated before 100 years have passed. This probability corresponds to the chosen reliability index ($\beta = 1.3$).

The following steps involve determining the combination of concrete covers and concrete chloride migration coefficients that will achieve the target service life. The design tables accompanying this report were calculated using a Monte-Carlo algorithm.

Steps iv to xii are the calculation process:

iv. The limit state modeling the risk of chloride induced corrosion is defined.

v. The required lifetime is specified.

vi. The requirement to the reliability for the lifetime is specified. The target is set as a minimum β-value. This value is equivalent to an acceptable probability of initiation of corrosion due to chlorides after the specified lifetime.

vii. The input parameters to the limit state are modelled either as stochastic variables (if they are assumed uncertain) or as constants. The stochastic variables are modelled by specifying a statistical distribution (i.e., a normal distribution) and corresponding statistical parameters (i.e., a mean value and a standard deviation). Each parameter is defined in the analysis software in the stochastic model.

viii. The program is run for different time values (t). Output is corresponding β-values.

ix. The β-value corresponding to the lifetime requirement is checked against the target-β, β_{target}.

x. If β corresponding to the lifetime is equal to $\beta_{target,}$ the concrete composition modelled by the input parameters in item iv (above) is fulfilling the requirements to the durability against chloride-induced corrosion.

If β corresponding to the lifetime is larger than β_{target}, the concrete composition modelled by the input parameters in item iv may be considered revised, setting less-restrictive demands to the diffusion properties of the concrete.

If β corresponding to the lifetime is lower than β_{target}, the concrete composition modelled by the input parameters in item iv should be revised, setting more restrictive demands to the diffusion properties of the concrete.

The procedure from iv to vi is repeated until $\beta \sim \beta_{target}$.

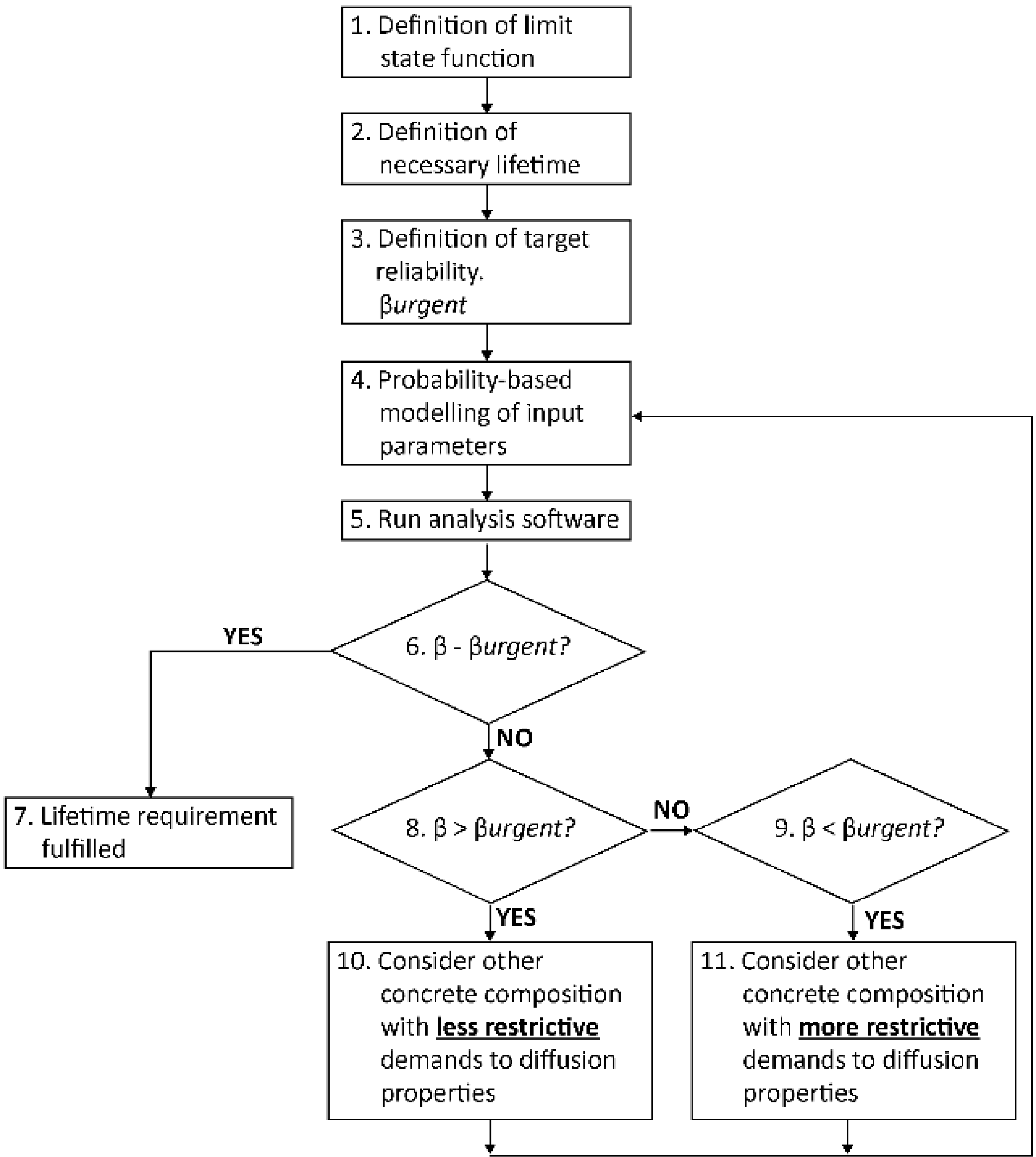

Figure 2: Decision Flow for Determining the Lifetime for Chloride-Induced Corrosion According to the *Fib* Bulletin 34 [1] Approach

3.0 Input Parameters

3.1 Probabilistic Methodology

The target service life is 100 years with a probability of failure of 10%; this is equivalent to a reliability index of 1.3 (β = 1.3). A Monte-Carlo method is used to compute the probability of failure.

3.2 Exposure

3.2.1 Exposure Zones

The following exposure zones are assumed:

- Splash / Deicing Salts Spray;
- Buried / Submerged; and,
- Atmospheric.

Splash refers typically to areas at water level (river, seawater). Deicing salts spray refers to areas exposed to deicing salts. Buried and submerged are for areas permanently buried in soil or submerged in water. Atmospheric refers to areas not included previously that are typically exposed to air. Figure 3 shows an example of exposure zones for the main towers of a cable supported bridge.

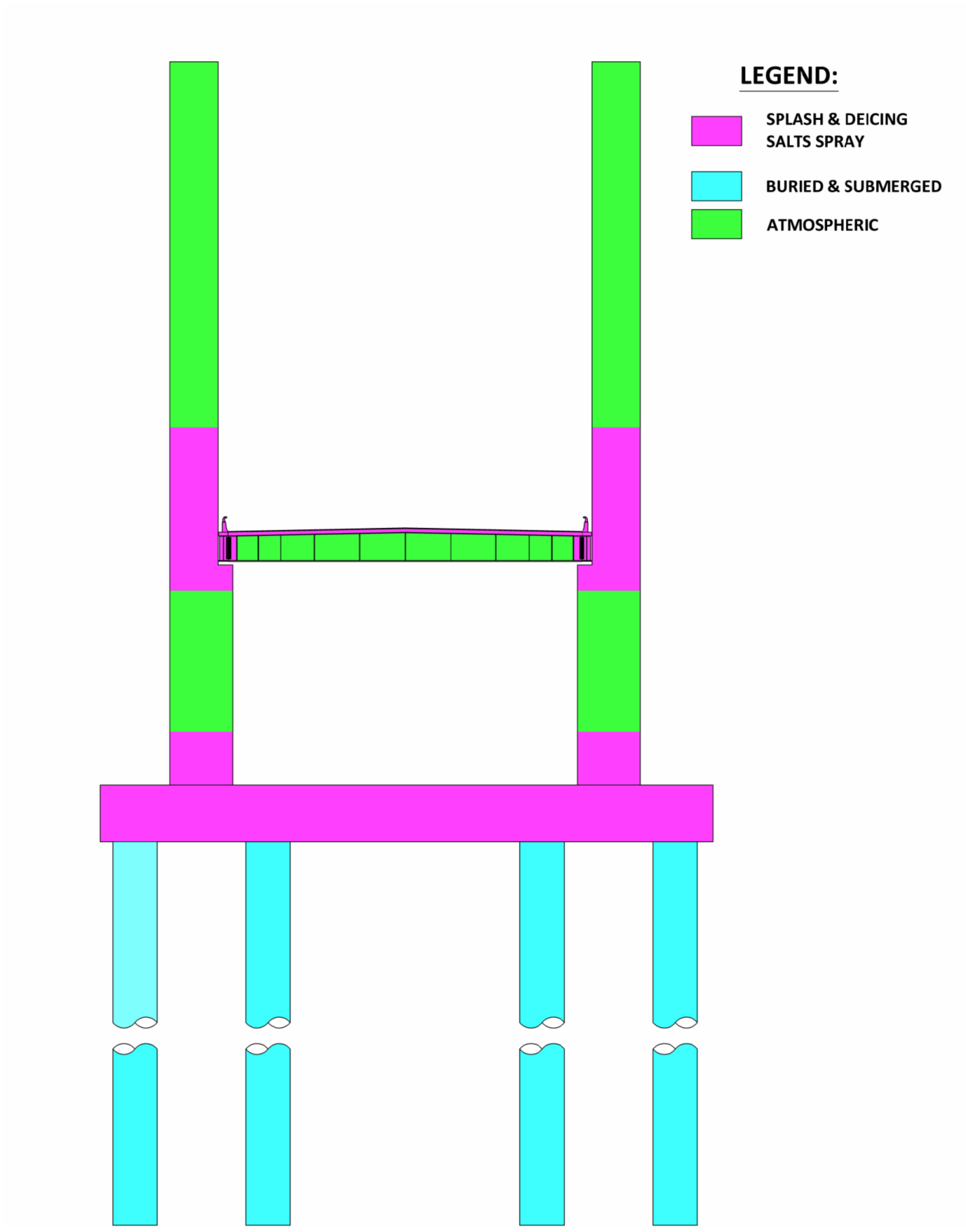

Figure 3: Example of typical exposure zones for main towers of a cable supported bridge

3.2.2 Surface Chloride Concentrations

The surface chloride concentration (Cl^-_s) depends on the different environments and material properties. The chloride concentration can be assessed based on data from structures in a similar environment and having similar material properties.

The design tables were developed for the following chloride surface concentrations:

Mean Values μ : 1.0%, 2.0%, 3.0%, 4.0% by mass cementitious

Standard Deviation: 0.5μ

Probability Density Function: Lognormal

3.2.3 Temperature

The temperature is based on project data or data from nearby meteorological stations.

In this case, the average monthly temperature was available. The twelve monthly mean values are used to determine the annual average temperature and standard deviation.

The data for Prineville, OR, is available here:
http://www.usclimatedata.com/climate/prineville/oregon/united-states/usor0281

Table 1 shows the calculated monthly average, annual average, and annual standard deviation.

Table 2: Temperature Data for Prineville, OR

	avg high °F	avg low °F	Avg °F
Jan	43	25	34
Feb	48	27	37.5
Mar	55	30	42.5
Apr	61	33	47
May	69	39	54
Jun	76	44	60
Jul	86	48	67
Aug	85	47	66
Sep	78	40	59
Oct	65	34	49.5
Nov	50	29	39.5
Dec	41	24	32.5
Annual Average			49.0
Annual Standard Deviation			12.2

The design tables were developed for the mean annual air temperatures based on public data as shown in Table 2.

Table 3: Temperature Data

Parameters	**Prineville, OR**	**HI**	**PA**	**VA**	**IO**
Mean values μ (°F)	49.0	TBD	TBD	TBD	TBD
Standard deviation (°F)	12.2	TBD	TBD	TBD	TBD
Probability density function	Normal	Normal	Normal	Normal	Normal

3.2.4 Transfer Function

Concrete elements exposed to chlorides and moisture with interruptions by dry periods are subject to capillary suction: the solution in the concrete pores close to the surface will evaporate during the dry periods and any re-wetting will provoke a capillary action. This effect leads to a rapid transport of chlorides into the concrete up to a depth Δx where the chlorides can accumulate with time until they reach a concentration equal to the surface concentration. Beyond this depth, chloride ingress is controlled by diffusion. The use of the transfer function effectively neglects any benefit from a thickness of Δx of the provided cover.

The transfer function Δx has been taken as specified in the *fib* Bulletin 34 for splash zone and zones with deicing salts subject to frequent wet-dry cycles: mean value of 8.9 mm, standard deviation of 5.6, beta distribution with parameter a = 0 and parameter b = 50.

The transfer function has a deterministic value of 0 for atmospheric zone and submerged/buried as specified by *fib*.

3.3 Materials

3.3.1 Chloride Migration Coefficient

The chloride migration coefficient ($D_{rcm,0}$) is typically the design transport parameter measured by a standard test (NT Build 492 [2]) at 28 days. The chloride migration coefficient will be a requirement with which the Contractor has to comply.

The chloride migration coefficient is modelled following *fib*:

Mean Values μ: as defined in the design tables

Standard Deviation: 0.2μ

Probability Density Function: Normal

3.3.2 Aging Factor

The aging factor (a) represents the ability of the concrete to develop an increased denseness with time. It is represented by decreasing diffusion coefficient with increasing age. It is found that the aging factor is strongly related to the type of cementitious material and the environmental condition. Aging determined with the rapid chloride migration method (NTBuild 492) represents only a certain portion of the total effect of the increase chloride penetration resistance due to ongoing hydration of concrete.

The age factor has been determined as per *fib* for each exposure zone and combinations of cementitious materials:

Table 4: Age factors

Concrete Mixes	Splash/Deicing Salts, Submerged/Buried			Atmospheric	
	Distribution	Parameters	Mean (μ)	Parameters	Mean (μ)
OPC+ 20-35%FA	beta	σ=0.15, a=0, b=1	0.60	σ=0.15, a=0, b=1	0.65
OPC	beta	σ=0.12, a=0, b=1	0.30	σ=0.15, a=0, b=1	0.65

3.3.3 Background Chloride Concentration

The background chloride concentration foreseen in the concrete mix (Cl^-_0) at the time of construction. A deterministic value of 0.1% by mass of cementitious materials is assumed.

3.3.4 Critical Chloride Concentration

The critical chloride concentration (Cl^-_{cr}) is the chloride concentration at the level of reinforcement, which triggers corrosion on the reinforcement. The critical chloride concentration used for the calculation is given by the assumed critical chloride concentration reduced by the background chloride concentration, i.e., Cl^-_{cr} - Cl^-_0. It is known that the critical corrosion-inducing chloride content depends among other factors on the environmental condition and the concrete quality.

The critical chloride concentration for uncoated carbon steel reinforcement (black steel) assumed in *fib* is:

Mean Values μ : 0.6% by mass of cementitious materials

Standard Deviation: 0.15

Probability Density Function: Beta with Parameter a = 0.2 and Parameter b = 2.0

3.3.5 Concrete Cover

Concrete cover is defined as the concrete thickness measured from the concrete surface to the outermost steel reinforcement.

All concrete covers are modeled using a normal distribution with a standard deviation of 6 mm to account for the variability of as-constructed cover. The standard deviation is based on guidance provided by *fib*. It should be noted that the standard deviation suggested by *fib* is based on typically observed accuracy of reinforcement placement and is distinct from specified placement tolerances. The standard deviation could be changed to accommodate the actual placement tolerance observed during construction.

The concrete cover assumed is:

Mean Values μ : 25.4, 50.8, 76.2, 101.6 mm (1.0, 2.0, 3.0, 4.0 inch)

Standard Deviation: 6 mm

Probability Density Function: Normal Distribution

3.3.6 Summary of Input Parameters

Table 4 presents a summary of the input parameters used to generate the design charts.

Table 5: Summary of Input Parameters

Category	Input Parameter	Symbol	Units	Distribution	Mean (μ)	Parameters	Reference / Notes
Exposure	Surface Chloride Concentration	Cls	%cem.	Lognormal	1.0, 2.0, 3.0, 4.0	$\sigma = 0.5\mu$	Splash Zone
					1.0, 2.0, 3.0, 4.0	$\sigma = 0.5\mu$	Atmospheric Zone
					1.0, 2.0, 3.0, 4.0	$\sigma = 0.5\mu$	Buried/Submerged Zone
	Temperature	T	^{o}C	Normal	Table 2	Table 2	
	Transfer Function	Δx	mm	Deterministic	0	-	*fib* for Buried/Submerged and Atmospheric Zones
				Beta	8.9	$\sigma = 5.6$, a=0, b=50	*fib* for Splash Zone
Materials	Chloride Migration Coefficient (28-day)	$D_{crm,0}$	x10^{-12} m^2/s	Normal	output to determine	$\sigma = 0.2\mu$	*fib*
	Age Factor	a	-	Beta	Table 3	Table 3	*fib*
	Initial Chloride Concentration	Co	%cem.	Deterministic	0.1	-	*fib*, consistent with ACI 318
	Chloride Threshold Concentration	Clcr	%cem.	Beta	0.6	$\sigma = 0.15$, a=0.2, b=2.0	*fib* distribution section B2.2.6
	Cover	-	mm	Normal	25.4 mm (1inch) to 101.6 mm (4 inch) in 0.5 inch increment	s = 6 mm	mean based on AASHTO, std based on *fib*

4.0 References

[1] *fib* (2006). *Model Code for Service Life Design*. *fib* Bulletin 34. International Federation for Structural Concrete (fib), Lausanne, Switzerland, 1st edition. 126 pp.

[2] NT BUILD 492 (1999). Concrete, mortar and cement-based repair materials: Chloride migration coefficient from non-steady-state migration experiments. NORDTEST method 492. NORDTEST, Espoo, Finland.