IBRACON Structures and Materials Journal
Revista IBRACON de Estruturas e Materiais
ISSN 1983-4195 ismj.org

ORIGINAL ARTICLE

Proposed durability parameters for reinforced concrete structures with design service life between 50 years and 100 years in Brazil

Proposição de parâmetros para durabilidade de estruturas de concreto armado com vida útil de projeto entre 50 e 100 anos no Brasil

Francine Barcellos Mumberger[a]
Bernardo Fonseca Tutikian[a]
Fabrício Longhi Bolina[a]

[a]Universidade do Vale do Rio dos Sinos – UNISINOS, Programa de Pós-graduação em Engenharia Civil, São Leopoldo, RS, Brasil.

Received 02 March 2022
Accepted 20 September 2022

Abstract: The main system of a construction is the structure. Its replacement is most of the times unfeasible and its repair or demolition generates waste that is often difficult to recycle, reuse or dispose of. In this way, structures with longer design service life (DSL) will generate lower environmental impacts, in addition to being financially more interesting for their users. Reinforced concrete is one of the most used types of structure, and commonly suffers with attacks of chloride ions and carbon dioxide that can facilitate the corrosion of the reinforcement. Concrete structures also suffer effects from the passage of time, as probability of accidental load increase, creep and shrinkage. The objective of this study was to determine durability and time effect parameters for DSLs between 50 and 100 years. The durability study was conducted through a review of reference studies, a selection of DSL models based on characteristic forms of environmental aggressiveness and comparison with international standards, using DSLs between 50 and 100 years and the following parameters: w/c ratio, compression strength, minimum cement usage and minimum cover. The time effect study considered Brazilian standards and their probability for accidental loads, creep, shrinkage and variations of the compressive strength, using DSLs between 50 and 100 years. The durability results were compiled in a table with practical recommended dimensional parameters. Despite some proposed parameters being higher or lower than standard values, the differences in performance were accounted through other parameters in order to maintain safety levels and to obtain minimum cover thicknesses. Variable vertical loads presented increments of 4.29% for 75 years and 7.22% for 100 years and wind velocity demonstrated a variation of 5.08% increase at 75 years and 9.84% at 100 years. Compression strength of concrete, creep coefficient and specific shrinkage deformation did not present significant variations. FBM: conceptualization, formal analysis, methodology, writing; BFT e FLB: data curation, formal analysis.

Keywords: reinforced concrete, durability, design service life.

Resumo: O principal sistema de uma construção é a estrutura. A sua substituição é muitas vezes inviável e a sua reparação ou demolição gera resíduos muitas vezes de difícil reciclagem, reutilização ou descarte. Dessa forma, estruturas com maior vida útil do projeto (VUP) gerarão menores impactos ambientais, além de serem financeiramente mais interessantes para seus usuários. O concreto armado é um dos tipos de estrutura mais utilizados, e comumente sofre com ataques de íons cloreto e carbonatação que podem facilitar a corrosão da armadura. As estruturas de concreto também sofrem efeitos com a passagem do tempo, como aumento da probabilidade de carga acidental, fluência e retração. Este trabalho tem como objetivo propor parâmetros para vidas úteis de projeto superiores a 50 anos, relativos à durabilidade e ao efeito do tempo. O estudo da durabilidade foi conduzido a partir de uma revisão bibliográfica e escolha de modelos de previsão de vida útil baseados em formas características de agressividade ambiental e comparação com normas internacionais, com

Corresponding author: Francine Barcellos Mumberger. E-mail: barcellosfrancine@gmail.com
Financial support: None.
Conflict of interest: Nothing to declare.
Data Availability: The data that support the findings of this study are openly available in Unisinos Library Digital Repository at http://www.repositorio.jesuita.org.br/handle/UNISINOS/9344, reference number 9344.

VUPs entre 50 e 100 anos e utilizando os seguintes parâmetros: relação a/c, resistência à compressão, consumo mínimo de cimento e cobrimento mínimo. O estudo do efeito do tempo considerou as normas brasileiras e sua probabilidade para cargas acidentais, fluência, retração e variações da resistência à compressão, utilizando VUPs entre 50 e 100 anos. Os resultados de durabilidade foram compilados em uma tabela com parâmetros dimensionais práticos recomendados. Apesar de alguns parâmetros propostos serem superiores ou inferiores aos valores de normas, as diferenças de desempenho foram contabilizadas através de outros parâmetros para manter os níveis de segurança e obter espessuras de cobertura mínimas. As cargas verticais variáveis apresentaram incrementos de 4,29% para 75 anos e 7,22% para 100 anos e a velocidade do vento demonstrou uma variação de 5,08% de aumento aos 75 anos e 9,84% aos 100 anos. A resistência à compressão do concreto, o coeficiente de fluência e a deformação específica de retração não apresentaram variações significativas. FBM: conceituação, análise formal, metodologia, redação; BFT e FLB: curadoria de dados, análise formal.

Palavras-chave: estrututas de concreto, durabilidade, vida útil de projeto.

How to cite: F. B. Mumberger, B. F. Tutikian, and F. L. Bolina, "Proposed durability parameters for reinforced concrete structures with design service life between 50 years and 100 years in Brazil," *Rev. IBRACON Estrut. Mater.*, vol. 15, no. 6, pp. e15603, 2022, https://doi.org/10.1590/S1983-41952022000600003

1 INTRODUCTION

The durability of a building relates both to its performance and variation over time, that is, how well it maintains its designed performance over its designed service life (DSL) [1]. In the case of structures, durability is a fundamental issue since being the main support system of a building, partial or total replacement is not viable due to high direct costs, risks in conducting repairs, indirect costs and inconvenience to users [2], [3].

From a financial point of view, a structure designed aiming durability and longer service life will incur less cost. Consequently, durability is a general worldwide concern for project managers, structural engineers and suppliers. There is a tendency among governments and builders, over the last decades, to design longer service lives for economical and practical reasons due to the possibility of unexpected durability issues [4].

In 2013, standard NBR 15575 [5], regarding structural performance, came into effect in Brazil. This standard defined structural safety parameters based on performance and durability of elements as well as deformations and fissuring. For residential constructions, required minimum, intermediate and superior performance levels must met service lives of 50 years, 63 years and 75 years, respectively. For other constructions that contained reinforced concrete such as bridges, overpasses, walkways, dams and sculptures, durability and performance were of even higher importance and DSLs might become as long as 100 years [6].

Parameters for minimum concrete cover, water/cement ratio and minimum compressive strength are specified in standard NBR 6118 [7]. However, parameters to determine performance and durability of structures with DSL of more than 50 years are not listed. Consequently, a workaround combination of international standards and extensive reference works are currently used to perform calculations on reinforced and pre-stressed concretes with designed service lives longer than 50 years [8].

According to FIB Model Code [3], the performance of a reinforced concrete structure was related to its behavior due to applied or self-generated loads throughout its DSL. In order to achieve adequate performance, the structure must be able to fully resist expected combined loads throughout its DSL; have safe structural integrity to ensure global stability; allow adequate deformations and have adequate mechanical strength. From this definition, it was possible to identify design parameters that had significant influence on performance throughout the design service life of a structure: loads and combined loads applied to the structure and concrete strength, creep and shrinkage.

The general purpose of this study was to analyze design parameters regarding reinforced concrete to formulate further parameters for a DSL between 50 years and 100 years for structures to be built in Brazil. This was a necessity since, as noted previously, most Brazilian standards list durability and performance parameters only for a DSL of up to 50 years. The analysis was conducted through predictive models which considered structural corrosion mainly from carbonation and chloride ion aggression. Results from this study were compiled and compared to international standards that consider DSLs longer than 50 years in table format to be used as an aid for projects with similar DSLs.

This study also performed an analysis on the effect of time on variable vertical loads and wind actions, creep, shrinkage and variations in compressive strength. Standards and reference studies were used as a basis to determine the degree of influence of time on these factors. Thus, results would allow structural engineers to determine if more specific calculations should be needed in order to reach a DSL between 50 years to 100 years.

2 DURABILITY PARAMETERS FOR REINFORCED CONCRETE

2.1 Carbonation

The most common form of deterioration is corrosion of reinforcements caused by the destruction of the passivating layer. This may be a result of carbonation, chloride ion aggression or both, to the extent that Brazilian standards regarding environmental aggressiveness is based solely on these two agents [8], [9].

Carbonation of the concrete cover is the main cause of the destruction of the passivating layer on the outer surface of reinforcements and results in their corrosion. It is a slow, non-linear process that decelerates down over time: as cement becomes more hydrated and products from carbonation are formed, pores in the concrete clog up and carbon dioxide penetration becomes more difficult [1], [10].

Local carbon dioxide concentration affects carbonation depth and number of chemical reactions since the diffusion process is driven by concentration. Relative humidity (RH) also affects this process as low humidity limits the amount of soluble calcium hydroxide while too high humidity clogs the pores and prevents carbon dioxide transport. The ideal RH range for carbonation falls in the 50% to 70% range [1].

2.2 Chloride Ion Penetration

Chloride ions do not affect concrete directly but rather the reinforcement as it dissolves the iron oxide protective layer and allows corrosion, being one of the most common forms of deterioration [11]. Destruction of this passivating layer occurs once the chloride ion concentration reaches a high enough threshold [12], [13].

The main factor in corrosion is the water/cement (w/c) ratio, which affects the porosity and permeability of concrete and, consequently, the diffusion velocity and penetration of chloride ions. An increase in w/c ratio from 0.4 to 0.6 can increase the diffusion velocity by a factor of 4. High ambient temperatures also lead to higher molecular mobility, which increases ion penetration [13], [14].

2.3 Permeability and Water/Cement Ratio

Permeability is defined as the degree to which liquids or gases can permeate concrete through capillary pores, leading to chemical aggressions in the concrete and cover. Permeability depends on w/c ratio, hydration level of the cement paste and aggregate porosity. Higher w/c ratios result in higher permeability [15].

In theory, a w/c ratio of 0.25 should be enough to hydrate cement but the resulting mixture would lack workability and a substantial portion of water would be lost to the environment through evaporation. In practice, without the use of additives, the minimum w/c ratio is in the order of 0.40 [16], [17].

2.4 Cement Usage

The minimum amount of cement usage is a parameter defined in standards in accordance to expected environmental aggressiveness to ensure a certain durability level to the structure. It is considered to have a minimum effect by itself in some studies and relegated to a second tier of importance [18], [19].

The amount of carbon dioxide absorbed in the carbonation process is driven by the amount of calcium hydroxide present in the pores rather than its total amount. It is not affected by the amount of cement in the mixture either, despite cement being the source of calcium hydroxide [20]. With regards to chloride ion aggression, it is usual to recommend a higher amount of cement in the mix ratio since it is believed that tricalcium aluminate in the cement reacts with chloride ions to form calcium chloroaluminate and prevents corrosion. However, the current understanding is that this chemical reaction only occurs when chloride ions are present at the moment of mixing and becomes irrelevant after curing, thus negating any possible advantage [14], [21].

2.5 Design Service Life (DSL) Estimation Models

A DSL can be generally defined as the time period in which a structure maintains its performance without the need of corrective maintenance [20]. In the case of reinforced concrete, DSL estimates must account for environmental parameters, materials and types of chemical aggressions which may befall on the structures – which are difficult to determine in the initial phases of a project.

Table 1 presents the main estimation methods along with the input and output parameters. Carbonation and chloride ion penetration effects are considered in all methods.

Table 1. DSL estimation models

Model	Input parameter	Output parameter
Helene [20] and Tuutti [22]	Chloride penetration depth	- Chloride ion concentration within concrete; - Chloride ion concentration in the surroundings; - Time of exposure; - Maximum water absorption by the concrete; - Concrete specific mass; - Cement usage.
Clear and Hay [23]	Chloride penetration depth	- Time of exposure; - w/c ratio; - Chloride ion concentration in the surroundings.
Andrade [24]	Chloride penetration depth	- Average RH of the surroundings; - Average temperature of the surroundings; - Surface chloride concentration; - Correction factor for the type of cement; - Characteristic compression strength of concrete; - Correction factor for the mix ratio; - Level of additives in the concrete; - Exposure time.
Morinaga [25]	Carbonation depth	- Carbon dioxide concentration; - Carbonation coefficient of the protective layer; - Temperature of the surroundings; - RH of the surroundings; - w/c ratio; - Exposure time.
Possan [26]	Carbonation depth	- Characteristic axial compression strength of concrete; - Exposure time; - Level of pozzolans in the concrete; - Average RH of the surroundings; - Carbon dioxide levels in the surroundings; - Type of cement; - Rainfall exposure levels of the structure.

The Tuutti [22] model is the most used individually and as a basis for other models. The Possan [26] and Andrade [24] models are also used since their input values are easier to obtain. The estimation models of Table 1 are only a portion of available models in references and can be used individually or together. In addition to the models presented here, we can also mention the following models: Saetta and Vitaliani [27], Lorensini [28], Nilsson et al. [29], Bob [30], Liu [31], Cady and Weyers [32], Bažant [33], Güneyisi et al. [34].

3 REINFORCED CONCRETE PERFORMANCE AND DESIGN PARAMETERS

3.1 Variable actions under imposed loads

Brazilian standards NBR 6120 [35] and NBR 8681 [36] consider variable actions under imposed loads the live loads from furniture, vehicles and individuals. Minimum loads were defined in accordance with the use of the building while the final load was defined from statistical combinations of loads over a period of 50 years.

Bolina et al. [37] presented a methodology to determine variable vertical loads for a DSL longer than 50 years. This methodology consisted of a statistical model based on performance levels for DSLs of 50 years, 63 years and 75 years contained in standard NBR 15575 [5]. Data were taken from standard NBR 8681 [36] with a 35% probability of excess loads in a period of 50 years and assuming variable vertical loads as a random variable with standard distribution. A general criterion was proposed in this study which resulted in a 2.7% increase in variable loads for a DSL of 63 years and a 4.4% increase for a DSL of 75 years. Based on these results, extrapolations were possible to cover DSLs between 50 years and 100 years.

3.2 Wind static actions

Wind static actions were determined from standard NBR 6123 [38] through dynamic pressure calculations based on characteristic wind velocity. The characteristic wind velocity was the multiplication of the base wind velocity with topographic features such as terrain roughness, construction dimensions and statistical factors.

The base wind velocity was represented by a 3-second wind gust, which had a statistical probability of being exceeded once in a period of 50 years. A statistical factor was defined that accounted for the safety of the building under the probability of the base wind velocity being exceeded within the DSL period. Standard NBR 6123 [38] presented a table of this statistical factor with a 63% probability of base wind velocity being exceeded in a DSL of 50 years. However, Annex B of NBR 6123 [38] also presented a mathematical equation which allowed the calculation of the statistical factor with different safety levels and DSL.

3.3 Increase in strength over time

Despite the increase in concrete strength over time related to the hydration level of the cement paste, long-term loads tend to have a cumulative detrimental effect on strength [11]. This decrease in strength is the same regardless of the value of compression strength but changes in accordance with the time at which initial loads are applied: the later the load, the less strength reduction [39]. This behavior is incorporated in national and international standards through coefficients applied to strength calculations. The *fib* Model Code [3] contains equations to calculate strength gains and losses more precisely so that a single coefficient may be determined.

Brazilian standards contained the same equations for gains of strength in concrete as *fib* Model Code [3]. These indicated that the coefficient for the decrease in strength under normal design conditions should be 1 since standards assumed that the increase in strength after 28 days should compensate for long-term load application. In the European standard EN 1992-1 [40], this same coefficient was recommended as 1 but variations between 0.8 and 1 were allowed depending on the country.

3.4 Concrete creep

Concrete creep is characterized by a non-linear increase in deformation under constant stress over time [3], [39]. The main factors affecting it are age of the concrete, length of time of load application, stress/strength relation, geometry of the structure, humidity, temperature, type of cement and additives, curing conditions and characteristics of aggregates [41], [42]. Factors that contribute negatively to creep are increase in centerline deflection, loss of prestress in concrete structures and increase in the curvature of columns. These factors introduce additional flexural moments which could increase second order effects, fissuring and corrosion of reinforcement [43]–[45].

Equations to determine creep are mostly obtained empirically and calibrated in laboratory tests. These make use of the difference in deflection between two equal elements, one loaded and another unloaded for the same length of time. Results are determined as specific creep, which represents creep deflection per unit of applied stress [3].

Comparing Brazilian and international standards, NBR 6118 [7] contained the same criteria to calculate the creep coefficient as *fib* Model Code [3], ACI 209 [24] and EN 1992-1 [40] standards despite the use of different equations.

3.5 Concrete Shrinkage

Shrinkage is a deformation of concrete over time caused by water loss which, in theory, does not depend on the applied stress. Shrinkage can generate internal stresses in the structure due to design restrictions and lead to fissuring [43]. The main consequences of concrete shrinkage are cracking, decrease in compression strength and decrease in durability.

Factors that affect shrinkage the most are: age of concrete, type of aggregate, water/cement (w/c) ratio used in the paste, relative air humidity (RH) and characteristic thickness of the slab. Aggregates have the most effect due to their sheer volume in concrete. Cement paste is affected by the w/c ratio, with increasing ratio also increasing shrinkage. The characteristic thickness of the slab is the ratio between volume and exposed surface of the structure, with smaller thicknesses associated with larger shrinkages [41], [43].

Comparison of Brazilian and international standards yielded similar results, despite not presenting the same formulation, the Brazilian standard used the same parameters as *fib* Model Code [3], ACI 209 [24] and EN 1992-1 [40] standards.

4 METHODOLOGY

The analysis of durability utilized the following suggested parameters: w/c ratio, compression strength, minimum cement usage and minimum cover as these were considered the most important for estimates for DSL. The time increment of the analyses was of 5 years.

The durability study was conducted through a review of reference studies and a selection of DSL models based on characteristic forms of environmental aggressiveness. These followed the four-tier environmental aggressiveness classification (EAC) of standard NBR 6118 [7]. Meteorological stations provided nation-wide temperature and humidity data which, coupled with the references, allowed the calibration of the DSL models and the calculation of CO_2 and Cl^- concentrations at each EAC. The calibrated models were applied to DSLs in 5 year increments and results were compared to international standards which contained DSLs longer than 50 years. Some of these standards were the Australian AS 3600 [46] with a DSL of 60 years and British BS 8500 [47] with a DSL of 100 years.

Temperature and RH data were obtained from 309 Brazilian meteorological stations measurements by the National Meteorological Institute (INMET) between 1981 and 2010. The average values were of 23.53 ºC and 74.2%, respectively. Since higher temperatures promote chloride ion corrosion, a temperature of 25 ºC was considered for all EAC as a form of rounding, simplifying of data and as margin of safety, as this temperature occurs in all Brazilian regions. Table 2 presents RH values used for each EAC and considerations based on the specific degradation of each classification. Note that RH values were selected to be as close as possible to the average INM value.

Table 2. Relative humidity values and considerations for each EAC

EAC	RH	Considerations
I	80%	Classification I represents an insignificant risk of deterioration. Consequently, the selected RH level optimizes carbonation, which is the main form of aggression.
II	70%	Classification II represents a small risk of deterioration. Consequently, the selected RH level was at the limit of optimum carbonation, which is the main form of aggression.
III	70%	Classification III represents an elevated risk of deterioration. Consequently, the selected RH level allows the transport of chloride ions.
IV	80%	Classification IV represents a high risk of deterioration. Consequently, the selected RH level was higher than for EAC III and approached chloride ion saturation.

The DSL estimation models were selected considering durability parameter analyses and carbonation or chloride ion corrosion as the main causes of deterioration of reinforced concrete structures [8], [12]. It should be noted that a combined carbonation and chloride ion aggression was not considered, nor were models that included sulfate corrosion or other possible chemical aggressions. Table 3 presents the selected models used at each EAC and output durability parameter.

Table 3. DSL estimation models, classification level and output durability parameters

Model	EAC	Output durability parameter
Possan [26]	I and II	Compression strength
Morinaga [25]	I and II	w/c ratio
Andrade [24]	III and IV	Compression strength
Helene [20] and Tuutti [22]	III and IV	Minimum cement usage
Clear and Hay [23]	III and IV	w/c ratio

Model calibration was conducted with parameters determined from standard NBR 6118 [7] for a DSL of 50 years. Control variables considered were concrete cover thickness, compression strength, w/c ratio, cement usage, temperature and RH. Environmental concentrations of CO_2 and chloride ions were given as output variables for each EAC and structural element. This calibration defined environmental conditions for the EAC of each model and allowed their application in distint designs with different designed DSLs. Results are shown in Table 4.

Table 4. Output environmental parameters

Model	EAC	Structural element	Temperature (°C)	RH (%)	Concentration of CO_2 (for EAC I and II) or Cl^- (for EAC III and IV)	Cover (mm)
Possan	I	Slab	-	80	0.0005%	12
		Beam / column	-	80	0.95%	15
	II	Slab	-	70	1.11%	15
		Beam / column	-	70	5.80%	20
Morinaga	I	Slab	25	80	0.0009%	9.8
		Beam / column	25	80	0.0021%	14.9
	II	Slab	25	70	0.0031%	14.9
		Beam / column	25	70	0.0056%	20
Andrade	III	Slab	25	70	0.38%	25
		Beam / column	25	70	0.43%	30
	IV	Slab	25	80	0.81%	35
		Beam / column	25	80	0.98%	40
Helene, Tuutti	III	Slab	-	-	0.39%	25
		Beam / column	-	-	0.47%	30
	IV	Slab	-	-	0.62%	35
		Beam / column	-	-	0.71%	40
Clear, Hay	III	Slab	-	-	80 mg/L	25
		Beam / column	-	-	150 mg/L	30
	IV	Slab	-	-	470 mg/L	35
		Beam / column	-	-	755 mg/L	40

Concrete structures analyzed were molded *in situ* with adequate quality control following current standards. The type of cement used was CP V with no chemical additives.

A maximum compression strength value of 50 MPa was proposed, which was common for most commercial applications. However, for each analysis, lower compression strength limits and minimum concrete cover thicknesses were used. The w/c ratios proposed were within the range between the maximum value for each EAC in standard NBR 6118 [7] and a minimum of 0.35. For each time interval of analysis, the higher w/c ratio mixtures were combined with the least cover thicknesses.

Cement usage was planned with a hard upper limit of 450 kg/m^3. Cement substitutions such as fly ash or blast furnace slag were included within this limit. For EAC I and EAC II, the minimum cement usage at each stage of analysis was extrapolated between the usage recommended for 50 years in standard NBR 12655 [40] and 100 years in standard BS 8500 [47]. For the Helene [20] and Tuutti [22] models, a maximum water absorption of 17% was considered for a concrete with specific mass of 2,400 kg/m^3.

The minimum concrete cover thickness for each EAC was determined based on all parameters of the DSL models with the intent to use the smaller thickness at each time interval of analysis. Changes in cover were rounded off to 5 mm increments in accordance with standard NBR 6118 [7]. Following the definition of durability parameters, the average, maximum and minimum cover values were analyzed in order to determine a minimum safety value. It should be noted that the minimum safety cover proposed in this study must be increased by a practical margin of safety as to become a nominal cover.

This part of the study focused on the effect of time on parameters of reinforced concrete projects, namely, variable vertical loads, creep deflections, shrinkages and compression strength. Variable loads were vertical or wind action, defined as a function of the probability of occurrence within a period of time. The mathematical formulas were defined to consider the same probabilities as listed in standards but with different periods of time. Time effects on compression strength, creep coefficient and shrinkage coefficient were conducted in accordance with equations presented in standards.

Variable vertical loading increments were determined using Bolina, Perrone and Tutikian [37] as a basis. It was assumed that the variable loads listed in standard NBR 6120 [35] were random variables with average normal distribution (μ) equal to 0 and standard deviation (σ) equal to 1. As shown in Bolina et al. [37], the variable vertical loading increments are established by consensus and have a probability of 25% to 35% of being exceeded during a period of 50 years, in Brazil. Thus, a conservative estimate would result that the risk of these loads being matched or exceeded would be of 35%.

Wind action was accounted for in Annex B of standard NBR 6123 [38]. It was assumed that the probability of the base wind velocity be matched or exceeded at least once in the period of time analyzed would be 63%.

The variation in compression strength (f_{ck}) over time was analyzed following the methodology of *fib* Model Code [3], which prescribed that the increase in strength was due to delayed cement hydration and decrease in creep.

Creep deflection and shrinkage were calculated in accordance with standard NBR 6118 [7] with wet curing instead of steam curing. This standard was selected for being more thorough than the Australian and Indian standards and equivalent to *fib* Model Code [3].

Minimum age for initial loading was taken to be 28 days with CPV cement and slump between 100 mm and 150 mm. For a simulated age, air temperature and RH were considered to be the same as the durability parameters at each EAC. A cement factor (*s*) of 0.165 was used to determine the creep coefficient which accounted for the increase in strength over time.

The simulated thickness was calculated for a structural element 20 cm in width and 30 cm in height for a resulting surface area of 600 cm^2.

5 RESULTS AND DISCUSSION

5.1 Compression Strength

Table 5 shows proposed values for compression strength (f_{ck}) between DSLs of 50 years and 100 years with the Possan [26] model and EAC I and EAC II. Also shown are the minimum cover ($C_{mín}$) needed for the specific strength at each age analyzed.

Table 5. Compression strength and minimum cover with the Possan [26] model

EAC	Structural element	f_{ck} (MPa)	$C_{mín}$ (mm)	f_{ck} (MPa)	$C_{mín}$ (mm)	f_{ck} (MPa)	$C_{mín}$ (mm)
		50 years		55 years		60 years	
I	Slab	20	10.0	20	12.7	20	13.3
	Beam / column	20	15.0	20	15.6	25	11.3
II	Slab	25	15.0	30	11.4	30	12.0
	Beam / column	25	20.0	30	15.0	30	15.7
		65 years		70 years		75 years	
I	Slab	25	9.6	25	10.0	25	10.4
	Beam / column	25	11.7	25	12.2	25	12.6
II	Slab	30	12.5	30	12.9	35	10.2
	Beam / column	30	16.3	30	16.6	35	13.2
		80 years		85 years		90 years	
I	Slab	25	10.7	25	11.0	25	11.4
	Beam / column	25	13.0	25	13.4	30	10.2
II	Slab	35	10.6	35	10.9	35	11.2
	Beam / column	35	13.7	35	14.1	35	14.5
		95 years		100 years			
I	Slab	25	11.7	25	12.0		
	Beam / column	30	10.5	30	10.8		
II	Slab	35	11.5	35	11.8		
	Beam / column	35	14.9	35	15.3		

For EAC I, the increases in compression strength between 50 years and 100 years were of 5 MPa for slabs (from 20 MPa to 25 MPa) and 10 MPa for beam/columns (from 25 MPa to 35 MPa). For EAC II, this increase was of 10 MPa for both slabs and beam/columns (from 25 MPa to 35 MPa). Minimum cover thicknesses varied slightly only for slabs in EAC I since the prediction model started with a slightly thicker cover as specified in standards. The remaining EACs presented no changes in cover over time since increases in compression strength supplied sufficient durability.

The insubstantial increase in compression strength between 50 years and 100 years and the maintenance of cover thickness with the selected DSL method were due to the low risk of deterioration attributed to EAC I and EAC II in standard NBR 6118 [7]. This was attributed to carbonation being the main form of aggression in these two classifications – a slow process that attenuated over time. Another consideration would be the effect on durability from the relation between compression strength and porosity. Since carbonation products could fill available pores and prevent the ingress of aggressive agents, the risk of deterioration also tended to be lower.

Table 6 compares the values of compression strength from EAC I and EAC II with international standards. Results from this study are similar to standard AS 3600 [46] at 60 years and more conservative than standard BS 8500 [47] at 100 years.

Table 6. Comparison of compression strength between Possan [26] model and international reference standards

EAC	Structural element	f_{ck} (MPa)			
		60 years		100 years	
		Possan [26]	AS 3600	Possan [26]	BS 8500
I	Slab	20	20	25	20
	Beam / column	25	25	30	20
II	Slab	30	32	35	25
	Beam / column	30	32	35	40

Table 7 presents compression strengths and minimum cover thickness of several DSL for the Andrade [24] model with EAC III and EAC IV. The time interval considered included the DSL range of Table 6.

Table 7. Compression strength and minimum cover thickness with the Andrade [24] model

EAC	Structural element	f_{ck} (MPa)	C_{min} (mm)	f_{ck} (MPa)	C_{min} (mm)	f_{ck} (MPa)	C_{min} (mm)
		50 years		55 years		60 years	
III	Slab	30	25	35	22.4	35	23.4
	Beam / column	30	30	35	26.9	35	28.1
IV	Slab	40	35	45	32.6	45	34.1
	Beam / column	40	40	45	37.3	45	38.9
		65 years		70 years		75 years	
III	Slab	35	24.4	40	22.2	40	22.9
	Beam / column	35	29.2	40	26.5	40	27.5
IV	Slab	45	35.5	50	33.1	50	34.3
	Beam / column	45	40.5	50	37.8	50	39.2
		80 years		85 years		90 years	
III	Slab	40	23.7	40	24.4	45	22.3
	Beam / column	40	28.4	40	29.2	45	26.7
IV	Slab	50	35.4	50	36.5	50	37.6
	Beam / column	50	40.5	50	41.7	50	42.9
		95 years		100 years			
III	Slab	45	22.9	45	23.5		
	Beam / column	45	27.5	45	28.2		
IV	Slab	50	38.6	50	39.6		
	Beam / column	50	44.1	50	45.2		

The more substantial increases in compression strength were determined on slabs and beam/columns for EAC III: starting at 15 MPa, increasing to 30 MPa at 50 years and increasing further to 45 MPa at 100 years. Cover thickness did not change for EAC III within the period of this study due to the increase in compressive strength providing sufficient durability. For EAC IV, compression strength increases reached the upper limit of 50 MPa of this study at 70 years. For longer time periods, the prediction method compensated the locked compression strength with increases in minimum cover thickness.

Structures with EAC III and EAC IV had deterioration risks classified as elevated and high, respectively, in standard NBR 6118 [7] which explained the resulting predicted increase in compression strength. In these classifications, the main mechanism of aggression was chloride ion attack, which represented more significant corrosion than general types. This aggression could be airborne or from contact with seawater or industrial runoff, which stressed the need of a less porous structure and, by extension, one with higher compression strength when compared to other EACs.

Compression strength results from the Andrade [24] model are also compared to international standards as shown in Table 8. In this case the values obtained in this study are very much of the same order of magnitude as the standards.

Table 8. Comparison of compression strength between Andrade [24] model and international reference standards

EAC	Structural element	f_{ck} (MPa)			
		60 years		100 years	
		Andrade [24]	AS 3600	Andrade [24]	BS 8500
III	Slab	35	32	45	45
	Beam / column	35	40	45	45
IV	Slab	45	50	50	45
	Beam / column	45	50	50	45

5.2 Water/cement Ratio

Table 9 shows the proposed values for w/c ratio for EAC I and EAC II and relevant DSL intervals for the Morinaga [25] model. Also shown are the corresponding minimum cover thickness.

Table 9. w/c ratio and minimum cover thickness with the Morinaga [25] model

EAC	Structural element	w/c	C_{min} (mm)	w/c	C_{min} (mm)	w/c	C_{min} (mm)
		50 years		55 years		60 years	
I	Slab	0.65	10	0.60	8.5	0.60	8.9
	Beam / column	0.65	15	0.60	12.7	0.60	13.3
II	Slab	0.60	15	0.55	13.7	0.55	14.3
	Beam / column	0.60	20	0.55	18.4	0.55	19.2
		65 years		70 years		75 years	
I	Slab	0.60	9.3	0.60	9.6	0.60	9.9
	Beam / column	0.60	13.8	0.60	14.3	0.60	14.9
II	Slab	0.55	14.9	0.50	13.2	0.50	13.7
	Beam / column	0.55	20.0	0.50	17.8	0.50	18.4
		80 years		85 years		90 years	
I	Slab	0.55	9.0	0.55	9.3	0.55	9.6
	Beam / column	0.55	13.5	0.55	13.9	0.55	14.3
II	Slab	0.50	14.2	0.50	14.6	0.45	12.4
	Beam / column	0.50	19.0	0.50	19.6	0.45	16.6
		95 years		100 years			
I	Slab	0.55	9.8	0.50	8.6		
	Beam / column	0.55	14.7	0.50	12.9		
II	Slab	0.45	12.7	0.45	13.0		
	Beam / column	0.45	17.1	0.45	17.5		

Starting from the listed w/c ratio of 0.65 for EAC I at 50 years in standard NBR 6118 [7], the predicted value decreased to 0.50 at 100 years with little effect on cover thickness as their values remained lower than the standard value at 50 years. As for EAC II, the starting w/c ratio was defined as 0.60 at 50 years in standard NBR 6118 [7] and decreased to 0.45 at 100 years, with the same effect in cover thickness as observed for EAC I. These results demonstrated that the need of further protection for EAC I and EAC II was lower and, consequently, the w/c ratio could be kept the same for longer DSLs when compared to other EACs. The same observation could be made for minimum cover thickness.

Comparison of w/c ratios and international reference standards are shown in Table 10. However, direct comparison at 60 years was not possible since standard AS 3600 [46] did not include w/c ratio amongst its recommended durability parameters. A comparison at 100 years with standard BS 8500 [47] showed that both EAC recommendations from this study were more conservative. This result was not entirely surprising since standard BS 8500 [47] maintained the same w/c ratio recommendations at 50 years through 100 years.

Table 10. Comparison of w/c ratio between Morinaga [25] model and international reference standards

EAC	Structural element	w/c ratio			
		60 years		100 years	
		Morinaga [25]	AS 3600	Morinaga [25]	BS 8500
I	Slab	0.60	-	0.50	0.70
	Beam / column	0.60	-	0.50	0.70
II	Slab	0.55	-	0.45	0.65
	Beam / column	0.55	-	0.45	0.45

Table 11 presents the proposed value of w/c ratio at EAC III and EAC IV for the Clear and Hay [23] model and corresponding minimum cover thickness for the time periods evaluated in this study.

Table 11. w/c ratio and minimum cover thickness for the Clear and Hay [23] model

EAC	Structural element	w/c	C_{min} (mm)	w/c	C_{min} (mm)	w/c	C_{min} (mm)
		50 years		55 years		60 years	
III	Slab	0.55	25	0.55	26.9	0.50	26.7
	Beam / column	0.55	30	0.55	32.2	0.50	32.0
IV	Slab	0.45	35	0.45	37.8	0.40	36.8
	Beam / column	0.45	40	0.45	43.2	0.40	42.2
		65 years		70 years		75 years	
III	Slab	0.50	28.5	0.45	27.8	0.45	29.4
	Beam / column	0.50	34.1	0.45	33.3	0.45	35.2
IV	Slab	0.40	39.3	0.35	37.5	0.35	39.6
	Beam / column	0.40	45.0	0.35	42.9	0.35	45.4
		80 years		85 years		90 years	
III	Slab	0.40	28.2	0.40	29.6	0.40	31.0
	Beam / column	0.40	33.7	0.40	35.4	0.40	37.1
IV	Slab	0.35	41.8	0.35	43.9	0.35	46.0
	Beam / column	0.35	47.8	0.35	50.3	0.35	52.7
		95 years		100 years			
III	Slab	0.40	32.4	0.40	33.8		
	Beam / column	0.40	38.8	0.40	40.5		
IV	Slab	0.35	48.1	0.35	50.2		
	Beam / column	0.35	55.1	0.35	57.4		

Table 11 shows that the starting w/c ratio of 0.55 for EAC III at 50 years, which was also the maximum value recommended in standard NBR 6118 [7], decreased to 0.40 at 80 years and remained at this level until 100 years. This variation was also observed in other EACs but, only for EAC III resulted in an increase in minimum cover thickness to maintain required durability since it occurred in a shorter time span. The w/c ratio for EAC IV started at 0.45 at 50 years, reached the minimum allowed value of this study of 0.35 at 70 years and remained at this level until 100 years. This result suggests how highly aggressive environments classified as EAC IV are, because in addition to a decrease in the w/c ratio, they need an increase in the cover thickness to ensure durability levels.

Permeability is fundamental to chloride ion attack since the aggressive agent must penetrate concrete through humidity in the air or in liquid form. Consequently, the less permeable the surface, the less ingress of chloride ions, and this effect could be achieved by controlling the w/c ratio. This explained the model predictions of lower w/c ratios for EAC III and EAC IV compared to the other EACs at lower DSLs. However, a lower hard limit of w/c ratio of 0.35 was needed to allow the most quantity of cement to be hydrated without compromising mechanical strength.

Table 12 compares the w/c ratio results of this study with standard BS 8500 [47] at 100 years. For EAC III, the w/c ratios of this study were slightly higher while for EAC IV the w/c ratios were the same as the standard. As in the case of Table 10, comparison at 60 years was not possible since standard AS 3600 [46] did not report w/c ratio.

Table 12. Comparison of w/c ratio between Clear and Hay [23] model and international reference standards

EAC	Structural element	w/c ratio			
		60 years		100 years	
		Clear and Hay [23]	AS 3600	Clear and Hay [23]	BS 8500
III	Slab	0.50	-	0.40	0.35
	Beam / column	0.50	-	0.40	0.35
IV	Slab	0.40	-	0.35	0.35
	Beam / column	0.40	-	0.35	0.35

5.3 Minimum Cement Usage

Minimum cement usage was included as a parameter in this study as it was also listed in NBR 6118 [7] despite some disagreement in this field regarding its use and effect on the durability of reinforced concrete structures.

Table 13 presents the proposed values of minimum cement usage for EAC I and EAC II as extrapolated from standards NBR 12665 [40] and BS 8500 [47]. As DSL increased, the minimum cement usage increased almost linearly. For EAC I, cement usage was 260 kg/m^3 for the entirety of the period studied. For EAC II, cement usage started at 280 kg/m^3 at 50 years and increased to 340 kg/m^3 at 100 years.

Table 13. Minimum cement usage extrapolated from standards NBR 12665 [40] and BS 8500 [47]

EAC	Structural element	Cement usage (kg/m^3)					
		50 years	55 years	60 years	65 years	70 years	75 years
I	Slab	260	260	260	260	260	260
	Beam / column	260	260	260	260	260	260
II	Slab	280	285	290	295	300	305
	Beam / column	280	285	290	295	300	305
		80 years	85 years	90 years	95 years	100 years	
I	Slab	260	260	260	260	260	
	Beam / column	260	260	260	260	260	
II	Slab	310	315	325	335	340	
	Beam / column	310	315	325	335	340	

Table 14 presents proposed minimum values of cement usage and cover thickness over the DSL period of this study with the Helene [20] and Tuutti [22] models

Table 14. Minimum cement usage and cover thickness from the Helene [20] and Tuutti [22] models

EAC	Structural element	Cem. Use. (kg/m^3)	C_{min} (mm)	Cem. Use. (kg/m^3)	C_{min} (mm)	Cem. Use. (kg/m^3)	C_{min} (mm)
		50 years		55 years		60 years	
III	Slab	320	25	355	24.7	385	24,8
	Beam / column	320	30	355	29.7	385	29,9
IV	Slab	360	35	395	34.9	435	34,6
	Beam / column	360	40	395	39.8	435	39,4
		65 years		70 years		75 years	
III	Slab	415	24.9	450	24.8	450	26,5
	Beam / column	420	29.7	450	29.8	450	32,0
IV	Slab	450	36.2	450	39.0	450	41,8
	Beam / column	450	41.3	450	44.4	450	47,6
		80 years		85 years		90 years	
III	Slab	450	28.3	450	30.1	450	31,8
	Beam / column	450	34.1	450	36.2	450	38,4
IV	Slab	450	44.6	450	47.4	450	50,2
	Beam / column	450	50.8	450	53.9	450	57,1
		95 years		100 years			
III	Slab	450	33.6	450	35.4		
	Beam / column	450	40.5	450	42.6		
IV	Slab	450	53.0	450	55.8		
	Beam / column	450	60.3	450	63.5		

As classified in Brazilian standards, the minimum cement usages were 320 kg/m^3 fort EAC III and 360 kg/m^3 for EAC IV. In this study, the maximum stipulated usage of 450 kg/m^3 was reached for EAC III at 70 years and for EAC IV at 65 years. Since usage was not allowed to increase further, only increases in minimum cover were allowed to ensure durability for longer DSLs.

A comparison of cement usage between the Helene [20] and Tuutti [22] models and standard BS 8500 [47] are presented in Table 15. In this case, standard BS 8500 [47] maintained the same minimum cement usage for DSLs between 50 years and 100 years, which made the results of this study more conservative estimations.

Table 15. Comparison of minimum cement usage Helene [20] and Tuutti [22] and international reference standards

EAC	Structural element	Minimum cement usage (kg/m^3)			
		60 years		100 years	
		Helene [20] and Tuutti [22]	AS 3600	Helene [20] and Tuutti [22]	BS 8500
III	Slab	385	-	450	380
	Beam / column	385	-	450	380
IV	Slab	435	-	450	380
	Beam / column	435	-	450	380

For EAC III and EAC IV, the Helene [20] and Tuutti [22] models resulted in elevated minimum cement consumption, even reaching the maximum consumption considered for this study at 70 years and 65 years, respectively. These hard upper limits were set from safety concerns since the chemical effect of cement consumption on the passivating layer was still a topic of discussion in this field. Additionally, an elevated amount of cement could favor fissuring and shrinkage, which would promote chemical aggressions listed in the EACs. Consequently, the use of cement substitutes could be a technically and economically viable alternative.

5.4 Minimum cover

Table 16 presents the proposed average cover thickness over all the models of this study with respect to EACs and DSLs analyzed.

Table 16. Proposed average cover of all DSL models with respect to EAC and time period

EAC	Structural element	Average cover (mm)										
		50 years	55 years	60 years	65 years	70 years	75 years	80 years	85 years	90 years	95 years	100 years
I	Slab	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0
	Beam / column	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0
II	Slab	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0
	Beam / column	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0
III	Slab	25.0	25.0	25.0	25.0	25.0	30.0	30.0	30.0	30.0	35.0	35.0
	Beam / column	30.0	30.0	30.0	30.0	30.0	35.0	35.0	35.0	35.0	40.0	45.0
IV	Slab	35.0	35.0	35.0	40.0	40.0	45.0	45.0	50.0	50.0	55.0	55.0
	Beam / column	40.0	40.0	40.0	45.0	45.0	50.0	50.0	55.0	55.0	60.0	60.0

For EAC I and EAC II, average cover for slabs or beam/columns remained the same when considering only 5 mm increments in cover as prescribed by standard NBR 6118 [7] over all DSL periods. This was a direct result of the type of aggression associated with EAC I and EAC II, which acted slowly and decelerated over time. Thus, deterioration levels were considered insignificant or small and, while average cover varied little, changes to other parameters were sufficient to attain the desired durability.

For EAC III at 75 years, average cover had an increase of 5 mm which further repeated itself at 90 years. On the other hand, EAC IV had the most variation in cover: a 5 mm increase at 65 years which repeated every 10 years until a cover of 60 mm for beam/columns at 100 years. Since chloride ion aggression was predominant in EAC III and EAC IV, starting minimum cover values were already higher than other EACs.

Table 17 shows a comparison of the minimum covers proposed in this study and international reference standards. Minimum cover for a DSL of 60 years differed considerably with a Δc between 5 mm and 10 mm between the proposed values and Australian standard AS 3600 [46]. In fact, the proposed values for EAC I and EAC II at 60 years were closer to the British standard BS 8500 [47] of 100 years. For the remaining EAC classifications, the minimum cover listed in standards were more conservative than the proposed values of this study. For example, for EAC III and EAC IV with a DSL of 100 years, the proposed minimum cover values were similar to standard BS 8500 [47] with differences between 5 mm and 10 mm.

Table 17. Comparison of proposed average covers and international reference standards

EAC	Structural element	Minimum cover (mm)			
		60 years		100 years	
		Proposed	AS 3600	Proposed	BS 8500
I	Slab	10	20	10	15
	Beam / column	15	30	15	15
II	Slab	15	40	15	25
	Beam / column	20	40	20	30
III	Slab	25	40	35	45
	Beam / column	30	45	45	45
IV	Slab	35	50	55	60
	Beam / column	40	65	60	60

5.5 Compilation of results

As the objective of this study was to propose parameters of consultation for structural reinforced concrete projects, all durability parameters for DSLs between 50 years and 100 years were compiled and are presented in Tables 18 and 19.

Table 18. Compilation of durability parameters for DSL between 50 years and 75 years

EAC	Structural element	f_{ck} (MPa) / Cement usage (kg/m^3) w/c ratio / minimum cover thickness (mm)					
		50 years	55 years	60 years	65 years	70 years	75 years
I	Slab	20 / 260 0.65 / 10	20 / 260 0.6 / 10	20 / 260 0.6 / 10	25 / 260 0.6 / 10	25 / 260 0.6 / 10	25 / 260 0.6 / 10
	Beam / column	20 / 260 0.65 / 15	20 / 260 0.6 / 15	25 / 260 0.6 / 15	25 / 260 0.6 / 15	25 / 260 0.6 / 15	25 / 260 0.6 / 15
II	Slab	25 / 280 0.6 / 15	30 / 285 0.55 / 15	30 / 290 0.55 / 15	30 / 295 0.55 / 15	30 / 300 0.5 / 15	35 / 305 0.5 / 15
	Beam / column	25 / 280 0.6 / 20	30 / 285 0.55 / 20	30 / 290 0.55 / 20	30 / 295 0.55 / 20	30 / 300 0.5 / 20	35 / 305 0.5 / 20
III	Slab	30 / 320 0.55 / 25	35 / 355 0.55 / 25	35 / 385 0.5 / 0.25	35 / 415 0.5 / 25	40 / 450 0.45 / 25	40 / 450 0.45 / 30
	Beam / column	30 / 320 0.55 / 30	35 / 355 0.55 / 30	35 / 385 0.5 / 30	35 / 420 0.5 / 30	40 / 450 0.45 / 30	40 / 450 0.4 / 35
IV	Slab	40 / 360 0.45 / 35	45 / 395 0.45 / 35	45 / 435 0.4 / 35	45 / 450 0.4 / 40	50 / 450 0.35 / 40	50 / 450 0.3 / 45
	Beam / column	40 / 360 0.45 / 40	45 / 395 0.45 / 40	45 / 435 0.4 / 40	45 / 450 0.4 / 45	50 / 450 0.35 / 45	50 / 450 0.35 / 50

Table 19. Compilation of durability parameters for DSL between 80 years and 100 years

EAC	Structural element	f_{ck} (MPa) / Cement usage (kg/m^3) w/c ratio / minimum cover (mm)				
		80 years	85 years	90 years	95 years	100 years
I	Slab	25 / 260 0.55 / 10	25 / 260 0.55 / 10	25 / 260 0.55 / 10	25 / 260 0.55 / 10	25 / 260 0.55 / 10
	Beam / column	25 / 260 0.55 / 15	25 / 260 0.55 / 15	30 / 260 0.55 / 15	30 / 260 0.55 / 15	30 / 260 0.5 / 15
II	Slab	35 / 310 0.5 / 15	35 / 315 0.5 / 15	35 / 325 0.45 / 15	35 / 335 0.45 / 15	35 / 340 0.45 / 15
	Beam / column	35 / 310 0.5 / 20	35 / 315 0.5 / 20	35 / 325 0.45 / 20	35 / 355 0.45 / 20	35 / 340 0.45 / 20
III	Slab	40 / 450 0.4 / 30	40 / 450 0.4 / 30	45 / 450 0.4 / 30	45 / 450 0.4 / 35	45 / 450 0.4 / 35
	Beam / column	40 / 450 0.4 / 35	40 / 450 0.4 / 35	45 / 450 0.4 / 35	45 / 450 0.4 / 40	45 / 450 0.4 / 45
IV	Slab	50 / 450 0.35 / 45	50 / 450 0.35 / 50	50 / 450 0.35 / 50	50 / 450 0.35 / 55	50 / 450 0.35 / 55
	Beam / column	50 / 450 0.35 / 50	50 / 450 0.35 / 55	50 / 450 0.35 / 55	50 / 450 0.35 / 60	50 / 450 0.35 / 60

In the proposed durability parameters of Table 18 and Table 19, rounded values were used in accordance with Brazilian standards, which were considered important for practical design applications. In the case of a desired intermediate DSL within the intervals presented, it was recommended to select the highest closer value.

5.6 Increases in variable vertical loads

Table 20 presents increases of variable vertical loads for the DSLs of this study. The load increased, starting with the load for a DSL of 50 years had a parabolic behavior over the years, reaching a 7.22% increase for a DSL of 100 years.

Table 20. Increase of variable vertical loads

n	R	p	T	Q	Fk	Load increase coefficient	% load increase with respect to 50 years
50	0.35	0.00858	116.57	2.38	3.38		
55	0.35	0.00780	128.18	2.42	3.42	1.010	1.03%
60	0.35	0.00715	139.78	2.45	3.45	1.020	1.95%
65	0.35	0.00661	151.39	2.48	3.48	1.028	2.80%
70	0.35	0.00614	163.00	2.50	3.50	1.036	3.58%
75	0.35	0.00573	174.60	2.53	3.53	1.043	4.29%
80	0.35	0.00537	186.21	2.55	3.55	1.050	4.96%
85	0.35	0.00506	197.82	2.57	3.57	1.056	5.58%
90	0.35	0.00478	209.42	2.59	3.59	1.062	6.16%
95	0.35	0.00452	221.03	2.61	3.61	1.067	6.71%
100	0.35	0.00430	232.64	2.63	3.63	1.072	7.22%

5.7 Increases of variable wind actions

The variation in the statistical factor (S3) used to determine wind velocity for variable actions between 50 years and 100 years is presented in Table 21. The increase of S3, from its value at 50 years, was of 9.84% at 100 years. This was necessary as higher base wind velocity would produce corresponding increases for DSLs longer than 50 years.

Table 21. Values for statistical wind factor S3 for DSL between 50 years and 100 years

Age (years)	S3	Relative S3 increase
50	1.0000	
55	1.0018	0.18%
60	1.0153	1.53%
65	1.0279	2.79%
70	1.0397	3.97%
75	1.0508	5.08%
80	1.0613	6.13%
85	1.0713	7.13%
90	1.0808	8.08%
95	1.0898	8.98%
100	1.0984	9.84%

5.8 Variations in compression strength over time

The variations in compression strength over time are represented by coefficient α_c, with values shown in Figure 1 for ages between 50 years and 100 years.

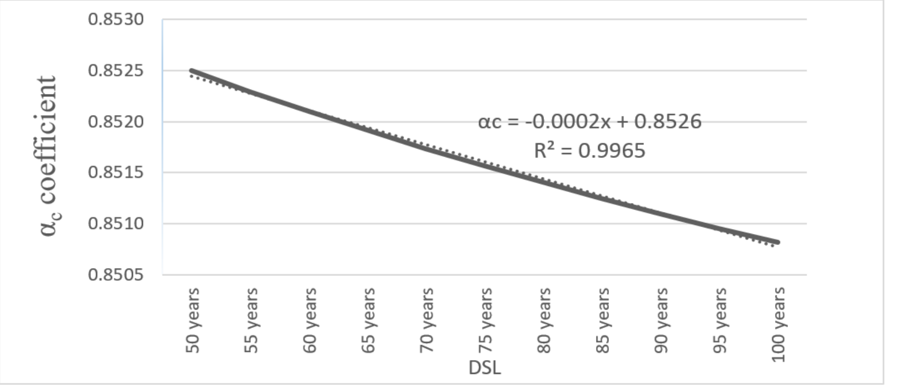

Figure 1 – Values of α_c coefficient between 50 years and 100 years

As noted from *fib* Model Code [3], the α_c coefficient had little variation after the first year and decreased progressively slower over time, to the point that variations past 50 years were almost negligible. Figure 1 attested that the relative variation between 50 years and 100 years was in the order of 0.17%. This behavior could be attributed to the hydration level reached by cement in the initial years, which consumed all available water inside the structure.

Since α_c did not present any significant variation, no analysis was necessary to evaluate its effect for the ages above 50 years under normal conditions. However, a factor not included in this study was the effect of different types of cement, which references showed could induce an increase on this coefficient. In this case, a positive effect would occur with an increase in compression resistance over time despite long-term loading.

5.9 Concrete deformations from creep and shrinkage

The variation of the coefficient of creep for DSLs between 50 years and 100 years is shown in Figure 2. Results are classified according to EAC and RH. It was determined that there was little variation in between ages and only some variation between EACs. The overall trend observed was that the smaller the RH, the larger the creep coefficient. This produced a higher deformation from creep in the structure and denoted that humidity was a more important factor than age for DSLs between 50 years and 100 years. The relationship between RH and shrinkage was related to creep from drying, which produced an exchange of humidity between the structure and the environment. Drying or loss of humidity to the environment under loading reduced water available to the hydration process of the paste and allowed increases in deformations. Thus, environments with low RH incurred higher creep coefficients and higher deformation in structures [42].

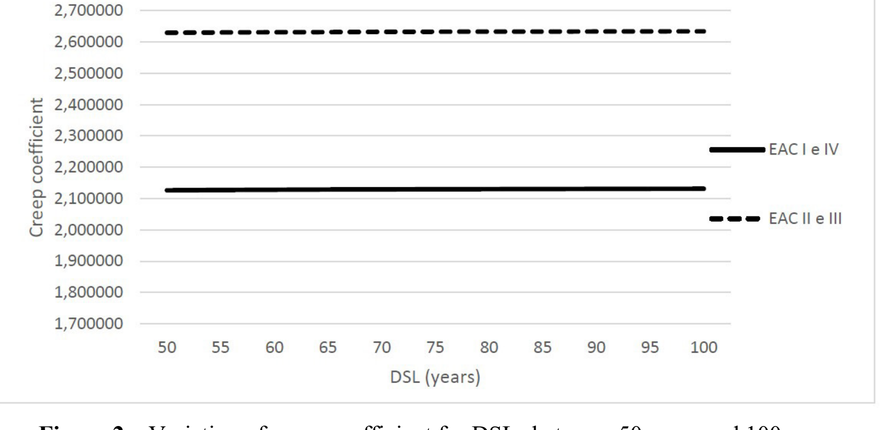

Figure 2 – Variation of creep coefficient for DSLs between 50 years and 100 years

Variations in specific shrinkage deformation for DSLs between 50 years and 100 years are shown in Figure 3. Similar to the coefficient of creep, some variations were observed but were not significant and the most changes were observed in between EAC classifications. The overall trend observed was that higher RH resulted in smaller shrinkages. Shrinkage from drying, also known as hydraulic shrinkage, was always a result of water loss from the inner portions of the concrete to an unsaturated environment.

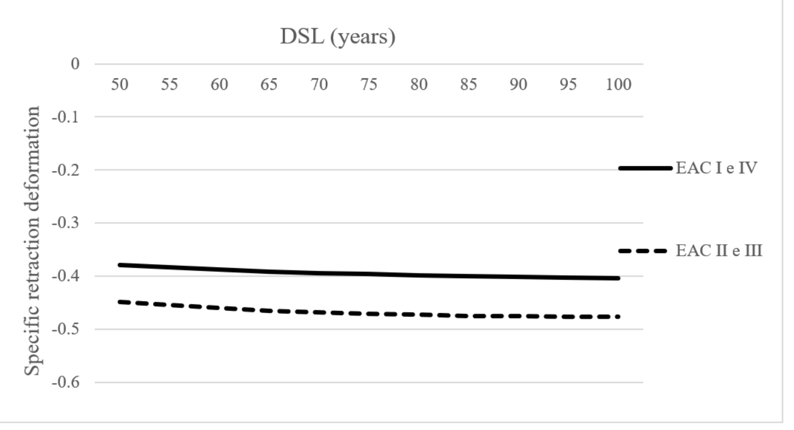

Figure 3 – Variation of specific shrinkage deformation for DSLs between 50 years and 100 years

It should be noted that creep and shrinkage results from this study were evaluated solely and relative to time. Ambient temperature, type of cement, specific thickness of the structure and loading age should also affect the creep coefficient $\varphi(t,t_0)$ and specific shrinkage deformation $\varepsilon_{cs}(t,t_0)$, but these factors vary according to the design of the project.

6 CONCLUSIONS

The objective of this study was to propose design parameters for reinforced concrete structures to ensure durability for DSLs between 50 years and 100 years. This was performed with DSL prediction models and reference standards. Despite some proposed parameters being higher or lower than standard values, the differences in performance were accounted through other parameters in order to maintain safety levels and to obtain minimum cover thicknesses. This was possible due to the superposition of effects that the parameters had with respect to each other and the preference to minimize cover thickness. Another cause of this effect was the flexibility of the British standard, which presented several combinations of parameters to ensure durability for each EAC. The possibility of achieving a desired performance through a flexible combination of parameters could be an interesting addition to Brazilian standards and would allow a designer to select the most parameters for a particular project.

Evaluation of the effect of time on design parameters did not produce relevant variations that could affect negatively structural performance for DSLs between 50 years and 100 years. Thus, no special consideration could be needed for these DSLs unless structural designers should deem them necessary.

Variable vertical loads presented small increments for the DSLs considered: 4.29% for 75 years and 7.22% for 100 years. In addition, these relatively small variations were based on a normalized vertical load for structural use and additional combinations of loads would further decrease their contribution to the total load.

The S3 factor used to determine the characteristic wind velocity demonstrated a slightly higher variation than variable vertical loads: 5.08% increase at 75 years and 9.84% at 100 years. This increase could be applied to base wind velocities between 30 m/s and 50 m/s. Thus, it was a factor as important as DSL period, especially for higher velocities above 40 m/s. However, it should be noted that wind action contribution to total load would also be diluted once additional combination of loads are incorporated.

Compression strength of concrete did not show considerable variation for DSLs between 50 years and 100 years, with the α_c coefficient remaining above 0.85. This value was recommended in standard NBR 6118 [7] for 50 years and further confirmed that no changes were needed to this coefficient for the DSLs of this study,

The creep coefficient and specific shrinkage deformation did not present significant variations for DSLs between 50 years and 100 years. For both parameters, variations were more evident in between EACs due to differences in RH.

Results indicated that deformations from these parameters were already stabilized at 50 years and further considerations were not necessary for ages of up to 100 years.

The main contribution of the durability study was in filling a niche gap in reference standards since the proposed durability parameters could be used as reference for designing reinforced concrete structures with DSLs longer than 50 years but less or equal than 100 years.

The evaluation of the effect of time on design parameters main contribution was to provide insight on which parameters should be analyzed more closely so that the workload engineers could be optimized.

REFERENCES

[1] K. Li, *Durability Design of Concrete Structures*, 1st ed. Singapura: Wiley, 2016.

[2] C. C. D. Dal Molin, A. B. Masuero, J. J. O. Andrade, E. Possan, J. R. Masuero, and M. M. Mennucci, *Contribuição à Previsão da Vida Útil de Estruturas de Concreto*, 1a ed. Porto Alegre: ANTAC, 2016.

[3] Comité Euro-Internacional Du Béton, *Model Code 2010 – Textbook on Behaviour, Design and Performance.* Lausanne: CEB-FIB, 2010.

[4] T. Dyer, *Concrete Durability.* Boca Raton: CRC Press, 2014.

[5] Associação Brasileira de Normas Técnicas, *Desempenho de Edificações Habitacionais*, NBR 15575, 2013.

[6] M. R. Penn and P. J. Parker, *Introdução a Infraestrutura.* Rio de Janeiro: LTC, 2017.

[7] Associação Brasileira de Normas Técnicas, *Projeto de Estruturas de Concreto – Procedimento,* NBR 6118, 2014.

[8] F. L. Bolina and B. F. Tutikian, "Especificação de parâmetros da estrutura de concreto armado segundo os preceitos de desempenho, durabilidade e segurança contra incêndio," *Rev. Concreto Construcoes*, vol. 76, pp. 133–147, 2014.

[9] P. Helene, "A nova NBR 6118 e a vida útil das estruturas de concreto," in *An. II Semin. Patol. Constr.* 2004.

[10] T. Dyer, *Concrete Durability.* Boca Raton: CRC Press, 2014.

[11] A. Poursaee, *Corrosion of Steel in Concrete Structures*. Duxford: Elsevier, 2016.

[12] C. L. Page and M. M. Page, *Durability of Concrete and Cement Composites*, 1st ed. Cambridge: Woodhead, 2007.

[13] A. Scott and M. G. Alexander, "Effect of supplementary cementitious materials (binder type) on the pore solution chemistry and the corrosion of steel in alkaline environments," *Cement Concr. Res.*, vol. 89, pp. 45–55, 2016.

[14] E. P. Figueiredo, "Ação dos cloretos no concreto," in *Concreto: Ciência e Tecnologia*, G. C. Isaia, Ed., São Paulo: Ibracon, 2011, pp. 887–902.

[15] A. M. Neville and J. J. Brooks, *Concrete Technology.* Harlow: Pearson Education Limited, 2010.

[16] S. M. Ashraf, *Practical Design of Reinforced Concrete Buildings.* Boca Raton: CRC Press, 2018.

[17] M. Setareh and R. Darvas, *Concrete Structures.* Springer, 2017.

[18] Y. Porras, C. Jones, and N. Schmiedeke, "Freezing and thawing durability of high early strength portland cement concrete," *J. Mater. Civ. Eng.*, vol. 32, no. 5, pp. 1–9, 2020.

[19] D. Saje and J. Lopatič, "The effect of constituent materials on the time development of the compressive strength of high-strength concrete," *Mag. Concr. Res.*, vol. 62, no. 4, pp. 291–300, 2010.

[20] P. Helene, "Contribuição ao estudo da corrosão em armaduras de concreto armado," Ph.D. dissertation, Dep. Eng. Constr. Civ., Univ. São Paulo, São Paulo, 1993.

[21] A. M. Neville, "Chloride attack of reinforced concrete: an overview," *Mater. Struct.*, vol. 28, no. 2, pp. 63–70, 1995.

[22] K. Tuutti, *Corrosion of Steel in Concrete.* Stockholm: Stockholm Royal Inst. Technol., 1982, 472 p.

[23] K. C. Clear and R. E. Hay, *Time to Corroslon of Reinforcing Steel In Concrete Slabs (Report FHWA/RD-73132).* Washington, D.C., 1983.

[24] J. J. O. Andrade, "Contribuição à previsão da vida útil das estruturas de concreto armado atacadas pela corrosão de armaduras: iniciação por cloretos," Ph.D. dissertation, Univ. Fed. Rio Grande do Sul, Porto Alegre, 2001.

[25] S. Morinaga, "Prediction of service lives of reinforced concrete buildings based on the corrosion rate of reinforcing steel," in *Proc. 5th Int. Conf. Durability Build. Mater. Componente*, 1990, 795 p.

[26] E. Possan, "Modelagem da carbonatação e previsão de vida útil de estruturas de concreto em ambiente urbano," Ph.D. dissertation, Prog. Pós-grad. Eng. Civ., Univ. Fed. Rio Grande do Sul, Poto Alegre, 2010.

[27] A. V. Saetta and R. V. Vitaliani, "Experimental investigation and numerical modeling of carbonation process in reinforced concrete structures. Part I: theoretical formulation," *Cement Concr. Res.*, vol. 34, pp. 571–579, 2004.

[28] V. R. Lorensini, "Avaliação probabilística da deterioração de estruturas em concreto armado," M.S. thesis, Curso de Pós-grad. Eng Estrut., Univ. Fed. Minas Gerais, Belo Horizonte, 2006.

[29] L. Nilsson, A. Andersen, T. Luping, and P. Utgenannt, "Chloride ingress data from field exposure in a swedish road environment," in *Proc. 2nd Int. Rilem Workshop Test. Modell. Chloride Ingress Concr.*, 2000.

[30] C. Bob, “Probabilistic assessment of reinforcement corrosion in existing structures,” in *Proc. Int. Conf. Concr. Repair, Rehabil. Prot.*, 1996, pp. 17–28.

[31] Y. Liu, “Modeling the time-to-corrosion cracking in chloride contaminated reinforced concrete structures,” Ph.D. dissertation, Virginia Polytech. Inst. State Univ., Virginia, 1996.

[32] P. D. Cady and R. E. Weyers, "Deterioration ratesof concrete bridge decks," *J. Transp. Eng.*, vol. 100, no. 1, pp. 34–44, 1984.

[33] Z. Bažant, "Physical modelfor steel corrosion in sea structures – applications," *J. Struct. Div.*, vol. 105, no. 6, pp. 1155–1166, 1979.

[34] E. M. Güneyisi, K. Mermerdas¸, E. Güneyisi, and M. Gesoglu, "Numerical modeling of time to corrosion induced cover cracking in reinforced concrete using soft-computing based methods," *Mater. Struct.*, vol. 48, pp. 1739–1756, 2015.

[35] Associação Brasileira de Normas Técnicas, *Ações para o Cálculo de Estruturas de Edificações*, NBR 6120, 2020.

[36] Associação Brasileira de Normas Técnicas, *Ações e Segurança nas Estruturas – Procedimento*, NBR 8681, 2003.

[37] F. Bolina, V. Perrone, and B. Tutikian, "Discussão sobre as ações variáveis de projeto segundo os requisitos mínimo, intermediário e superior de desempenho da ABNT NBR 15575," *Rev. Conc. Constr.*, vol. 79, pp. 65–78, 2015.

[38] Associação Brasileira de Normas Técnicas, *Forças Devidas ao Vento em Edificações*, NBR 6123, 1988.

[39] H. Rüsch, "Researches toward a general flexural theory for structural concrete," *ACI J. Proc.*, no. 57-1, pp. 1–28, 1960.

[40] European Committee for Standardization, *Eurocode 2: Design of Concrete Structures*, EN 1992-1, 2004.

[41] J. Z. F. Diniz, J. F. Fernandes, and S. C. Kuperman, “Retração e fluência,” in *Concreto: Ciência e Tecnologia*, G. C. Isaia, Ed., São Paulo: Ibracon, 2011, pp. 673–703.

[42] A. M. Neville, *Properties of Concrete*, 5th ed. Essex: Trans-Atlantic Publications, Inc., 2012.

[43] J. M. Araújo, *Estruturas de Concreto – Modelos de Previsão da Fluência e da Retração do Concreto*, 4a ed. Rio Grande: Dunas, 2002.

[44] Z. P. Bažant and O. Buyukozturk, “Creep analysis of structures,” in *Mathematical Modeling of Creep and Shrinkage of Concrete*. New York: Wiley, pp. 217–273, 1988.

[45] L. T. Katakoa, “Estudo experimental e numérico da deformabilidade por fluência e sua utilização na monitoração de estruturas de concreto,” Ph.D. dissertation, Progr. Pós-grad. Eng. Civ., Univ. São Paulo, São Paulo, 2010.

[46] Australian Standard, *Concrete Structures*, AS 3600, 2009.

[47] British Standard, *Concrete Complementary British Standard to BS EN 206-1 – Part 1 : Method of Specifying and Guidance for the Specifier*, BSI 8500-1, 2006.

Author contributions: FBM: conceptualization, formal analysis, methodology, writing; BFT e FLB: data curation, formal analysis.

Editors: Edna Possan, Mark Alexander