

Prediction and Evaluation of Service Life of Concrete Structure in cold Regions*

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Abstract

With the growing amount of concrete in the capital infrastructure, the maintenance and management burden is increasing and the calculation of life-cycle-cost and service lifetime are gaining in importance. The design, construction, maintenance, and management of structures raises the following questions: (1) Will the concrete structure provide service over its lifetime without reinforcement and repair? (2) How will performance and safety deteriorate if the structure is left alone? (3) How can current durability designs be checked? Accordingly, there is a need to predict the deterioration and service life of concrete structures. A method of achieving this needs to be established as soon as possible. The purpose of this study is to develop a technique for predicting and evaluating the service life of concrete structure (reinforced concrete [RC] structures) design in cold regions through predicting deterioration. This paper describes the development of a technique and a case-study in which the technique is applied to a tentative model. The case-study results show that the water-cement ratio and depth of cover over reinforcing steel influenced deterioration over time due to neutralization and steel corrosion. Between some cases of choosing of the rate, several-fold variations in neutralization depth and a factor of two in the time until the onset of steel corrosion were recognized. As shown in Fig. 4, it was determined that the difference in synthetic degree of deterioration was 20% at 50 years after construction.

1. INTRODUCTION

With the growing amount of concrete in the capital infrastructure, the maintenance and management burden is increasing and the calculation of life-cycle-cost and service lifetime are gaining in importance.

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The design, construction, maintenance, and management of structures raises the following questions: (1) Will the concrete structure provide service over its lifetime without reinforcement and repair? (2) How will performance and safety deteriorate if the structure is left alone? (3) How can current durability designs be checked? Accordingly, there is a need to predict the deterioration and service life of concrete structures. A method of achieving this needs to be established as soon as possible.

The following problems are presented in the prediction and evaluation of these characteristics: (1) There are many kinds of deterioration which affect the life of a concrete structure, and many factors influence deterioration. It is a complex matter. (2) The relationship between design and service life is not well understood because almost no design and construction data for existing structure have been preserved. (3) The inspection data of structures are not being put to practical use. (4) Techniques of maintenance and management are not well established and those in the field do not take it seriously enough. (5) The durability assessment of concrete structures is only now being developed in Japan and abroad and is not yet mature enough to meet the demands of the fields.

The purpose of this study is to develop a technique for predicting and evaluating the service life of concrete structures (reinforced concrete [RC] structures) designed in cold region through predicting deterioration. This paper describes the development of a technique and a case-study in which the technique is applied to a tentative design model.

2. METHOD

2.1 Development of Prediction and Evaluation Methods

The method is able to predict the deterioration and evaluate the service life of RC structures designed for cold climates. Initially, the essential criteria for deterioration-prediction were studied. Because deterioration-prediction must take into consideration the importance of the structure and its component members, it was divided into two levels: the general level (level 1) and a level in which greater accuracy is demanded such as for structural members of particular importance, safety, function, or demanding maintenance-free installation (level 2). In this study, level 1 of deterioration prediction was considered.

Next, the following issues were studied to develop a system able to predict deterioration:¹⁾ (1) The choice of indicators of prediction and evaluation. (2) The procedure for prediction and evaluation (Fig. 1). (3) The collation and analysis of existing studies and experimental data. (4) A deterioration-prediction equation. (5) The grading of deterioration. (6) The weighting of predictions and evaluations. As shown in Table 1, this led to suggestions for the method of prediction and evaluation of deterioration of RC structure.

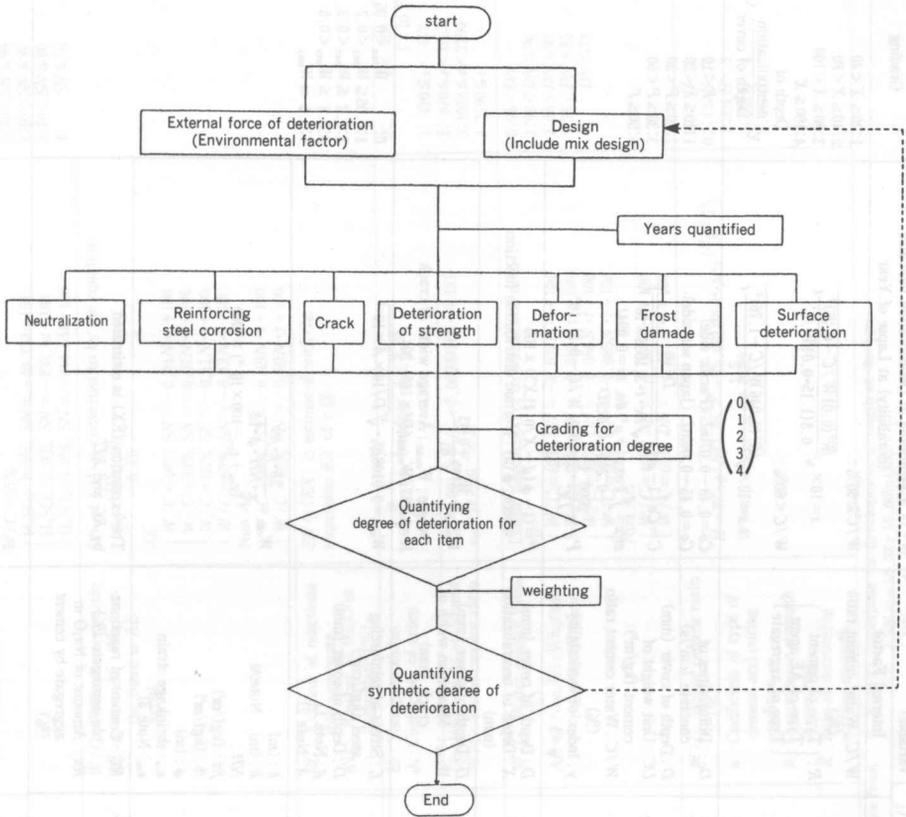


Fig. 1 The prediction and evaluation procedure for RC structures

Table 1(b) Continued

Quantitative Item	Schedule Indicator	Form II	Variable	Interval Point	External Factor	Form I
1. Chloride ion concentration (ppm)	Chloride ion concentration (ppm)	1	Chloride ion concentration (ppm)	0	Chloride ion concentration (ppm)	1
2. Sulfate ion concentration (ppm)	Sulfate ion concentration (ppm)	2	Sulfate ion concentration (ppm)	0	Sulfate ion concentration (ppm)	2
3. Free water/cement ratio	Free water/cement ratio	3	Free water/cement ratio	0	Free water/cement ratio	3
4. Air content (%)	Air content (%)	4	Air content (%)	0	Air content (%)	4
5. Maximum aggregate size (mm)	Maximum aggregate size (mm)	5	Maximum aggregate size (mm)	0	Maximum aggregate size (mm)	5
6. Water/cement ratio	Water/cement ratio	6	Water/cement ratio	0	Water/cement ratio	6
7. Cement content (kg/m³)	Cement content (kg/m³)	7	Cement content (kg/m³)	0	Cement content (kg/m³)	7
8. Slag content (kg/m³)	Slag content (kg/m³)	8	Slag content (kg/m³)	0	Slag content (kg/m³)	8
9. Fly ash content (kg/m³)	Fly ash content (kg/m³)	9	Fly ash content (kg/m³)	0	Fly ash content (kg/m³)	9
10. Silica fume content (kg/m³)	Silica fume content (kg/m³)	10	Silica fume content (kg/m³)	0	Silica fume content (kg/m³)	10
11. Admixture content (kg/m³)	Admixture content (kg/m³)	11	Admixture content (kg/m³)	0	Admixture content (kg/m³)	11
12. Concrete strength (MPa)	Concrete strength (MPa)	12	Concrete strength (MPa)	0	Concrete strength (MPa)	12
13. Concrete density (kg/m³)	Concrete density (kg/m³)	13	Concrete density (kg/m³)	0	Concrete density (kg/m³)	13
14. Concrete modulus of elasticity (GPa)	Concrete modulus of elasticity (GPa)	14	Concrete modulus of elasticity (GPa)	0	Concrete modulus of elasticity (GPa)	14
15. Concrete coefficient of thermal expansion (1/°C)	Concrete coefficient of thermal expansion (1/°C)	15	Concrete coefficient of thermal expansion (1/°C)	0	Concrete coefficient of thermal expansion (1/°C)	15
16. Concrete shrinkage (%)	Concrete shrinkage (%)	16	Concrete shrinkage (%)	0	Concrete shrinkage (%)	16
17. Concrete creep (1/1000)	Concrete creep (1/1000)	17	Concrete creep (1/1000)	0	Concrete creep (1/1000)	17
18. Concrete permeability (10⁻¹² cm²/s)	Concrete permeability (10⁻¹² cm²/s)	18	Concrete permeability (10⁻¹² cm²/s)	0	Concrete permeability (10⁻¹² cm²/s)	18
19. Concrete carbonation depth (mm)	Concrete carbonation depth (mm)	19	Concrete carbonation depth (mm)	0	Concrete carbonation depth (mm)	19
20. Concrete chloride ion penetration (C)	Concrete chloride ion penetration (C)	20	Concrete chloride ion penetration (C)	0	Concrete chloride ion penetration (C)	20
21. Concrete sulfate ion penetration (S)	Concrete sulfate ion penetration (S)	21	Concrete sulfate ion penetration (S)	0	Concrete sulfate ion penetration (S)	21
22. Concrete freeze-thaw cycles	Concrete freeze-thaw cycles	22	Concrete freeze-thaw cycles	0	Concrete freeze-thaw cycles	22
23. Concrete freeze-thaw damage (%)	Concrete freeze-thaw damage (%)	23	Concrete freeze-thaw damage (%)	0	Concrete freeze-thaw damage (%)	23
24. Concrete freeze-thaw scaling (mm)	Concrete freeze-thaw scaling (mm)	24	Concrete freeze-thaw scaling (mm)	0	Concrete freeze-thaw scaling (mm)	24
25. Concrete freeze-thaw spalling (mm)	Concrete freeze-thaw spalling (mm)	25	Concrete freeze-thaw spalling (mm)	0	Concrete freeze-thaw spalling (mm)	25
26. Concrete freeze-thaw delamination (mm)	Concrete freeze-thaw delamination (mm)	26	Concrete freeze-thaw delamination (mm)	0	Concrete freeze-thaw delamination (mm)	26
27. Concrete freeze-thaw surface cracking (mm)	Concrete freeze-thaw surface cracking (mm)	27	Concrete freeze-thaw surface cracking (mm)	0	Concrete freeze-thaw surface cracking (mm)	27
28. Concrete freeze-thaw internal cracking (mm)	Concrete freeze-thaw internal cracking (mm)	28	Concrete freeze-thaw internal cracking (mm)	0	Concrete freeze-thaw internal cracking (mm)	28
29. Concrete freeze-thaw reinforcement corrosion (%)	Concrete freeze-thaw reinforcement corrosion (%)	29	Concrete freeze-thaw reinforcement corrosion (%)	0	Concrete freeze-thaw reinforcement corrosion (%)	29
30. Concrete freeze-thaw reinforcement rusting (mm)	Concrete freeze-thaw reinforcement rusting (mm)	30	Concrete freeze-thaw reinforcement rusting (mm)	0	Concrete freeze-thaw reinforcement rusting (mm)	30
31. Concrete freeze-thaw reinforcement spalling (mm)	Concrete freeze-thaw reinforcement spalling (mm)	31	Concrete freeze-thaw reinforcement spalling (mm)	0	Concrete freeze-thaw reinforcement spalling (mm)	31
32. Concrete freeze-thaw reinforcement delamination (mm)	Concrete freeze-thaw reinforcement delamination (mm)	32	Concrete freeze-thaw reinforcement delamination (mm)	0	Concrete freeze-thaw reinforcement delamination (mm)	32
33. Concrete freeze-thaw reinforcement surface cracking (mm)	Concrete freeze-thaw reinforcement surface cracking (mm)	33	Concrete freeze-thaw reinforcement surface cracking (mm)	0	Concrete freeze-thaw reinforcement surface cracking (mm)	33
34. Concrete freeze-thaw reinforcement internal cracking (mm)	Concrete freeze-thaw reinforcement internal cracking (mm)	34	Concrete freeze-thaw reinforcement internal cracking (mm)	0	Concrete freeze-thaw reinforcement internal cracking (mm)	34
35. Concrete freeze-thaw reinforcement corrosion product (mm)	Concrete freeze-thaw reinforcement corrosion product (mm)	35	Concrete freeze-thaw reinforcement corrosion product (mm)	0	Concrete freeze-thaw reinforcement corrosion product (mm)	35
36. Concrete freeze-thaw reinforcement rusting product (mm)	Concrete freeze-thaw reinforcement rusting product (mm)	36	Concrete freeze-thaw reinforcement rusting product (mm)	0	Concrete freeze-thaw reinforcement rusting product (mm)	36
37. Concrete freeze-thaw reinforcement spalling product (mm)	Concrete freeze-thaw reinforcement spalling product (mm)	37	Concrete freeze-thaw reinforcement spalling product (mm)	0	Concrete freeze-thaw reinforcement spalling product (mm)	37
38. Concrete freeze-thaw reinforcement delamination product (mm)	Concrete freeze-thaw reinforcement delamination product (mm)	38	Concrete freeze-thaw reinforcement delamination product (mm)	0	Concrete freeze-thaw reinforcement delamination product (mm)	38
39. Concrete freeze-thaw reinforcement surface cracking product (mm)	Concrete freeze-thaw reinforcement surface cracking product (mm)	39	Concrete freeze-thaw reinforcement surface cracking product (mm)	0	Concrete freeze-thaw reinforcement surface cracking product (mm)	39
40. Concrete freeze-thaw reinforcement internal cracking product (mm)	Concrete freeze-thaw reinforcement internal cracking product (mm)	40	Concrete freeze-thaw reinforcement internal cracking product (mm)	0	Concrete freeze-thaw reinforcement internal cracking product (mm)	40

Table 1(b) Continued

Table 1 (a) Deterioration indicator, factor, calculation, and grading for items quantified

Quantifying Item	Selected Indicator		Factor ([] variable)		Calculation of Deterioration Indicator (durability) at Lapse of Year	Grading
	Indicator	Phenomenon	External Factor	Internal Factor		
a. Neutralization	Depth of neutralization x (mm)	① Neutralization	[t : Service life (year)]	W/C : Water cement ratio (%) R : Type of cement Type of AE agent Type of aggregate	$W/C \geq 60\%$: $x = 10 \times \sqrt{\frac{R^2(0.01W/C - 0.25)^2}{0.3(1.15 + 0.01W/C) t}}$ $W/C < 60\%$: $x = 10 \times \sqrt{\frac{R^2(0.046W/C - 1.76)^2}{7.2} t}$	$1: 20 \leq X < 40$ $2: 40 \leq X < 80$ $3: 80 \leq X < 100$ $4: 100 \leq X$ Depth of neutralization X : Depth of cover (%)
b. Reinforcing steel ctmission	Ratio of corrosion surface P (%)	① Corrosion of penetrating chloride	[f : Service life (year)] L : Distance from sea (m) Co : Amount of chloride from sea (wt%)	Dc : Diffusivities of concrete (cm ² /s) D : Depth of cover (mm) UC : Unit weight of cement (kg/m ³) W/C : Water cement ratio (%) γ : Index of workability: $\gamma = 1$	$Co = 0.48 - 0.07 \ln L$ (Pacific side) $Co = 0.45 - 0.06 \ln L$ (Japan sea side) $C = Co \left(1 - e^{-\gamma} \sqrt{\frac{2 \sqrt{Dc \cdot t + 3.1536 \times 10^4}}{D/10}} \right)$ $erfx = \int_0^x \delta \exp(+\mu^2) d\mu$, $m = 0.094t + 0.245 - 0.029D$ $P = \frac{2000 C}{\gamma UC} \cdot \frac{1}{2} \cdot (0.01 W/C - 0.3) \cdot 10^m$	0: $P < 10$ 1: $10 \leq P < 20$ 2: $20 \leq P < 30$ 3: $30 \leq P < 50$ 4: $50 \leq P$
		② Corrosion of neutralization	D : Depth of cover (mm) X : Depth of neutralization (mm)		$P = (1 - \phi) (d - X/0.41X) \times 100$ where: ϕ (a): Normal distribution function	
		③ Corrosion of crack	D : Depth of cover (mm) W_{max} : Maximum width of crack (mm)		$W_{mean} = \frac{W_{max} + 0.03}{1.91}$ where: W_{mean} : Average width of crack $P = 0.167 (W_{mean}/D)^2 \times 10^4 - 20$	
c. Crack	Maximum width of crack (mm)	① Crack of steel stress	f_s : Stress of reinforcing steel (kgf/cm ²) D : Depth of cover (mm) β : Note 1) A : Note 2)		$W_{max} = 0.0108 \beta f_s \sqrt{D/10} \times A \times 10^{-3}$	0: $W_{max} < 0.05$ 1: $0.05 \leq W_{max} < 0.2$ 2: $0.2 \leq W_{max} < 0.3$ 3: $0.3 \leq W_{max} < 0.5$ 4: $0.5 \leq W_{max}$
		② Crap of dry and temperature shrinkage	TC : Change of temperature (°C) b : (m) h : (m) NH : fct : (kgf/cm ²) β : (m) ϕ : (m) ϵ_{es} : shrinkage strain ϵ_{es} : Note 3)		$W_{max} = \frac{2b \cdot h \cdot fct}{\pi \cdot NH \cdot \phi \cdot 1b} (\epsilon_{es} + \epsilon_{es} - 100 \times 10^{-4}) \times 1000$	
		③ Crack of alkali silica reaction		RC : Content of reaction-able aggregate (%) Rw : Amount of Na ₂ O in aggregate by cement (%)	The expansion (EX) is estimated by RC and RW	

Note(a) b: Width of the section(m); h: Depth of the member(m)

NH: Number of steel members

Table 1(b) Continued

Quantifying Item	Selected Indicator		Factor ([] variable)		Calculation of Deterioration Indicator (durability) at Lapse of Year	Grading
	Indicator	Phenomenon	External Factor	Internal Factor		
d. Deterioration of strength	Notes 2) Ratio of compressive strength SV (%)	① Deterioration of penetrating sulfate	[t: Service life (year)]	W/C: Water/cement ratio	Linear Regression of experimental data W/C = 55% (H ₂ SO ₄ : 0.3%, SN = -40.15t + 100 H ₂ SO ₄ : 2.0%, SN = -233.6t + 100 H ₂ SO ₄ : 5.0%, SN = -244.55t + 100 DN of f. ① is converted to by the equation SN = $\frac{DN - 25}{0.75}$ AE (W/C = 40% SN = -0.04N · t + 100 W/C = 50% SN = -0.07N · t + 100 W/C = 55% SN = -0.11N · t + 100 W/C = 60% SN = -0.12N · t + 100 Non AE (W/C = 30% SN = -0.49N · t + 100 W/C = 60% SN = -0.69N · t + 100	0: 95 < SN 1: 90 < SN ≤ 95 2: 80 < SN ≤ 90 3: 70 < SN ≤ 80 4: SN ≤ 70
		② Deterioration of frost damage	[t: Service life (year)] N: Cycles of freeze-thaw a year	W/C: Water/cement ratio AE or Non AE: Whether there is AE agent	DN of f. ① is converted to by the equation SN = $\frac{DN - 25}{0.75}$ AE (W/C = 40% SN = -0.04N · t + 100 W/C = 50% SN = -0.07N · t + 100 W/C = 55% SN = -0.11N · t + 100 W/C = 60% SN = -0.12N · t + 100 Non AE (W/C = 30% SN = -0.49N · t + 100 W/C = 60% SN = -0.69N · t + 100	0: 95 < SN 1: 90 < SN ≤ 95 2: 80 < SN ≤ 90 3: 70 < SN ≤ 80 4: SN ≤ 70
	Strain ε (%)	③ Deterioration of alkali silica reaction aggregate		RC: Content of reaction-able aggregate RU: Amount of Na ₂ O in aggregate (%)	SN (f(EX) is estimated with the expansion EX of c ③	0: ε < 420 (× 10 ⁻⁶) 1: 420 ≤ ε < 670 2: 670 ≤ ε < 1033 3: 1033 ≤ ε < 2290 4: 2290 ≤ ε
Change rate in relative dynamic modulus of elasticity DN (%)		① Deformation of creep strain	σ: Stress of concrete loading (kgf/cm ²)	Ec: Young's modulus φc: Coefficient of creep	$\epsilon = \frac{\sigma}{E_c} \cdot \phi_c$ (outdoor φc = 2.0)	0: ε < 420 (× 10 ⁻⁶) 1: 420 ≤ ε < 670 2: 670 ≤ ε < 1033 3: 1033 ≤ ε < 2290 4: 2290 ≤ ε
	Average depth of degree H (mm)	② Deformation of dry and temperature	Tc: Change of temperature (°C)	Uc: Unit weight of cement W/C: Water cement ratio	$\epsilon_{as} = 0.00148 \frac{W}{C} + 0.000301 U_c - 0.131$ $\epsilon_{st} = 10 \times 10^{-4} \times T_c$	0: 96 < DN 1: 96 < DN ≤ 96 2: 96 < DN ≤ 93 3: 78 < DN ≤ 85 4: DN ≤ 78
Surface deterioration		① Frost damage	[t: Service life (year)] N: Cycles of freeze-thaw a year	W/C: Water cement ratio AE or Non AE: Whether there is AE agent	Linear Regression of experimental data AE (W/C = 40% DN = -0.028N · t + 100 W/C = 50% DN = -0.053N · t + 100 W/C = 55% DN = -0.080N · t + 100 W/C = 60% DN = -0.085N · t + 100 Non AE (W/C = 40% DN = -0.36N · t + 100 W/C = 60% DN = -0.51N · t + 100	0: H < 1 1: 1 ≤ H < 2 2: 2 ≤ H < 3 3: 3 ≤ H < 4 4: 4 ≤ H
	Surface deterioration	① Surface deterioration of frost damage	[t: Service life (year)] N: Cycles of freeze-thaw a year W: Coefficient of supplying seawater	W/C: Water cement ratio α: Coefficient of type of cement and curing condition f _c : Compressive strength of concrete R: Rate of decreasing surface strength	$H = W \cdot \alpha \left(\frac{W}{55} \right)^3 - (0.001195 f_c^2 \cdot f_c^2)^{\left(\frac{W}{55} \right)}$ W = 0.5 where; α = 0.0129	0: H < 1 1: 1 ≤ H < 2 2: 2 ≤ H < 3 3: 3 ≤ H < 4 4: 4 ≤ H

Note 1) β: The ratio of distance from axial of neutrality to center of reinforcing steel to distance from axial of neutrality to tensile side in the case of beam 12.
A: The area of tensile side concrete of symmetry with steel number of reinforcing steel.
Note 2) Superpose the development strength at the age
(SN = -55.32 + 16.60ln(365t))
(DN = -41.49 + 12.54ln(365t))
Note 3) ε_{st}: Total contraction after peak temperature due to heat of hydration

2.2 Case Study of Prediction and Evaluation Method

As shown in Fig. 2, a case-study in which the technique was applied to a tentative design model was called out. The subject of the case-study was a storage pit to be built half underground which was required to have a long service life. It was designed at a tentatively designed model.

These internal and external factors are shown in Table 2.

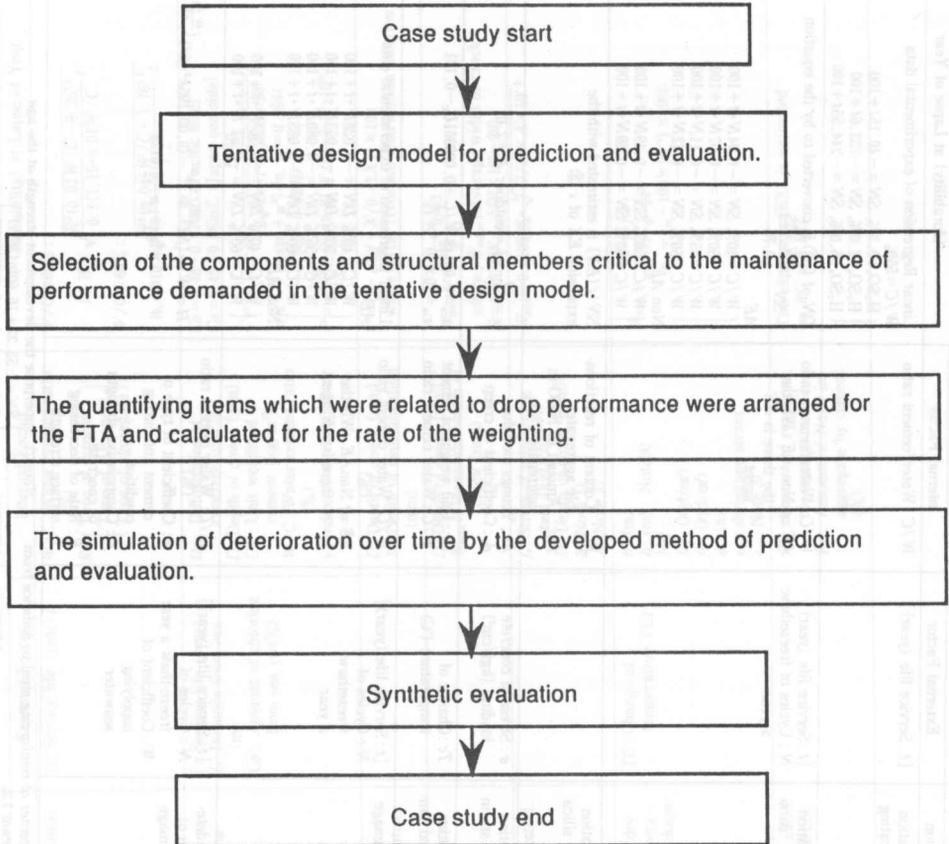


Fig. 2 The case-study procedure

In accordance with our method of examination, the component and structural members important to the maintenance of the performance called for in the tentative design model were selected and the changes over time in deterioration were evaluated and predicted using the developed method. Two conditions were studied: (1) unrepaired initial hydration cracks and (2) damage due to freezing and thawing for a year. For condition (1), the water-cement ratio and the concrete cover selected from internal factor of deterioration were varied. For condition (2), weighting the rates of the synthetic degree of deterioration were varied. The characteristics demanded of the tentative design model were assumed to be bearing load and watertightness. The criteria used to quantify the performance loss were arranged by FTA (Fault Tree Analysis) and given for the appropriate weighting.

Table 1 The main external and internal factors of the tentative design model

Factor		Mark : Parameter (unit)	Datum
External factors	External factors of deterioration	t : Service life (years)	100
		L : Distance from sea (m)	3000
		N : Cycles of freeze-thaw per year (cycles)	110
Internal factors	Material	f_{ct} : Tensile strength of concrete (kgf/cm^2)	25
		f_b : Average bond strength of concrete and steel (kgf/cm^2)	48
		D_c : Diffusivities of concrete	1.8×10^{-8}
		ϕ_c : Coefficient of creep	2
		R : Type of cement	Normal
		Type of agent	AE agent
		Type of aggregate	river sand and river gravel
		AB : Absorption rate of aggregate (%)	2

3. CASE-STUDY RESULTS

The simulation results, shown in Fig. 3, show that the water-cement ratio and the depth of cover over reinforcing steel influenced deterioration over time due to neutralization and steel corrosion. As a result of varying the water cement ratio and the Concrete cover, several fold variations in the neutralization depth and 2 fold variation in the time until the onset of steel corrosion were observed. As shown in Fig. 4, it was determined that the difference in synthetic degree of deterioration was 20% at 50 years after construction.

4. CONCLUSION

A method has been proposed for prediction and evaluation of the service life of RC structures in cold regions. According to the proposal that deterioration over time be predicted on the basis of the construction factors and the internal environmental factors. The most suitable parameters and rate parameters were selected according to external factors, at the location of the RC structure. The simulation was able to predict the deterioration during the expected life of the RC structure and to evaluate the service life of the structure. We were able to check the durability design considered the most suitable and acquire the technical information needed to judge the most suitable time for repair. The possibility of prediction and evaluation of service life of RC structures in cold climates was proved.

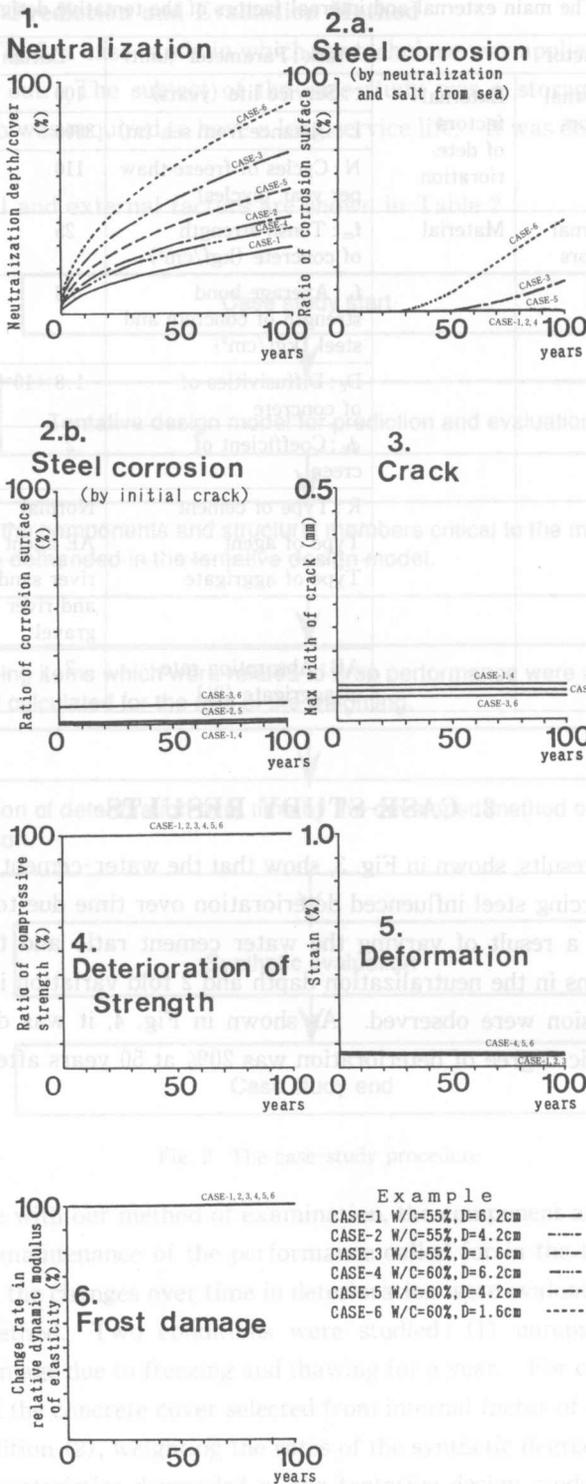
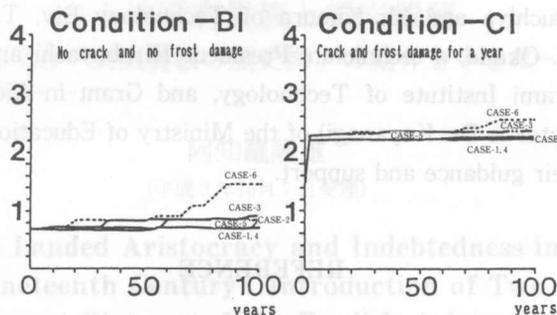
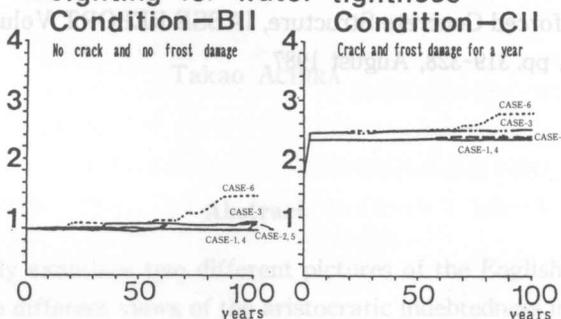


Fig. 3 Change in deterioration indicator with variation in deterioration factor (unrepaired initial cracks and frost damage for a year)

Weighting for load



Weighting for water tightness



Example

CASE-1	W/C=55%, D=6.2cm	—
CASE-2	W/C=55%, D=4.2cm	- - - -
CASE-3	W/C=55%, D=1.6cm	- · - · -
CASE-4	W/C=60%, D=6.2cm	- · - · -
CASE-5	W/C=60%, D=4.2cm	- - - -
CASE-6	W/C=60%, D=1.6cm	- · - · -

Fig. 4 Change in synthetic degree of deterioration with variation in deterioration factor and type of weighting

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4. CONCLUSION

A method has been proposed for the prediction and evaluation of the service life of RC structures in cold regions. Basic study led to the proposal that deterioration over time be predicted on the basis of internal design and construction factors and external environmental factors. The most suitable parameters and rates of internal factors were selected according to external factors, at the location of the RC structure. The simulation was able to predict the deterioration during the expected life of the RC structure and to evaluate the service life of the structure. We were able to check the durability design considered the most suitable and acquire the technical information needed to judge the most suitable time for repair. The possibility of prediction and evaluation of service life of RC structures in cold climates was proved.

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4. CONCLUSION

A method has been proposed for the prediction and evaluation of the service life of RC structures in cold regions. Basic studies led to the proposal that deterioration over time be predicted on the basis of material design and construction factors and external environmental factors. The most suitable parameters and rates of deterioration were selected according to external factors at the location of the RC structure. The simulation was able to predict the deterioration during the expected life of the RC structure and to evaluate the service life of the structure. We were able to check the turnshifting design considered the most suitable and acquire the technical information needed to judge the most suitable time for repair. The possibility of prediction and evaluation of service life of RC structures in cold climates was proved.